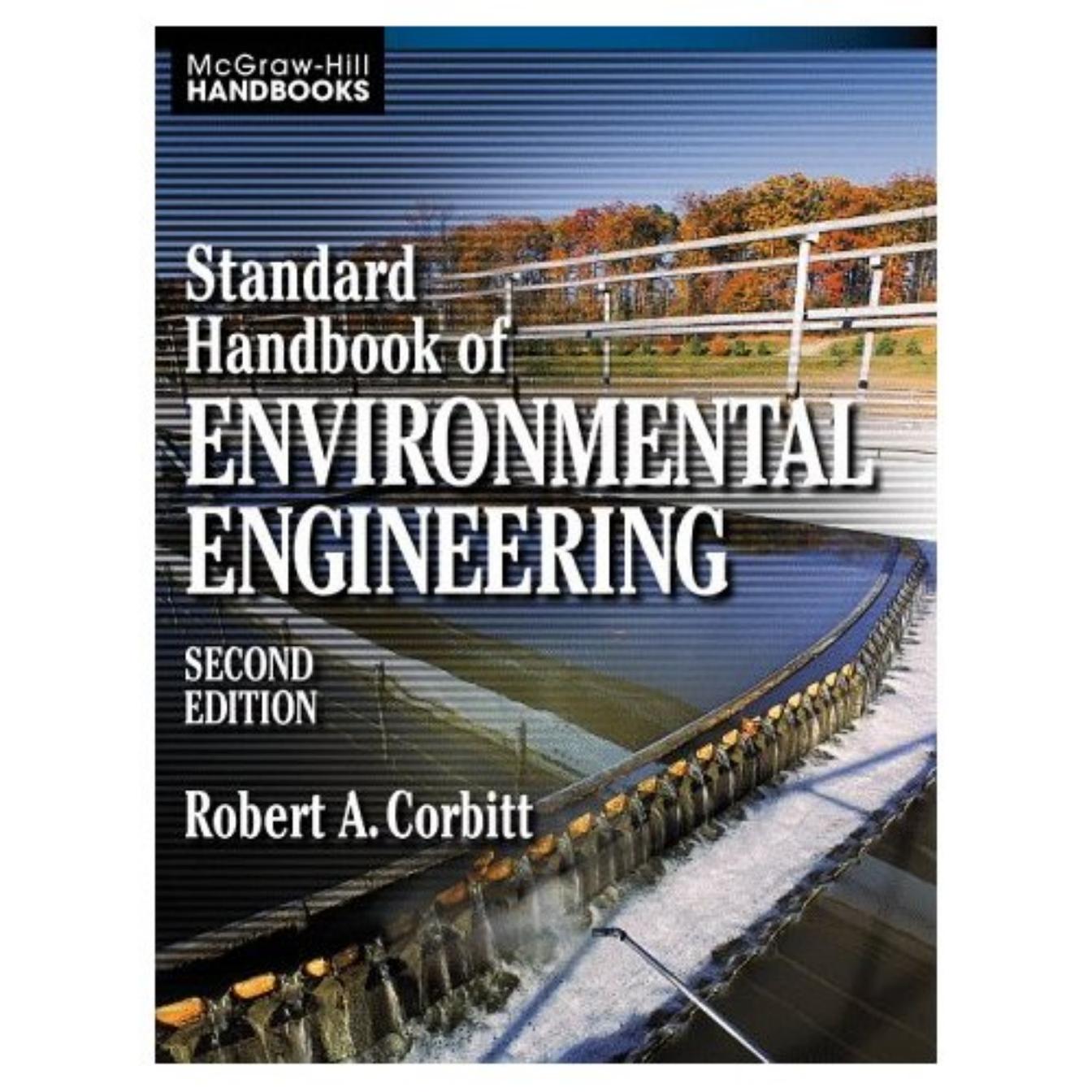


McGraw-Hill
HANDBOOKS



Standard
Handbook of
**ENVIRONMENTAL
ENGINEERING**

SECOND
EDITION

Robert A. Corbitt

CHAPTER 1

ENVIRONMENTAL ENGINEERING

David Burstein, P.E.*

THE ENVIRONMENTAL ENGINEER

Environmental engineering is essential for development of facilities for protection of the environment and for the proper management of natural resources. The environmental engineer places special attention on the biological, chemical, and physical reactions in the air, land, and water environments and on improved technology for integrated management systems, including reuse, recycling, and recovery measures.

Environmental engineering began with society's need for safe drinking water and management of liquid and solid wastes. Urbanization and industrialization significantly contributed to the formation of unsanitary conditions in many areas. The terms "public health" and "sanitary" were first applied to those engineers seeking solutions to the elimination of waterborne disease in the 1800s. More recently, abatement of air and land contamination became new challenges for the environmental engineer. Today, management of toxic and hazardous wastes are additional focus areas.

Traditionally, environmental engineers drew their basic education and training from civil engineering programs. In order to broaden their perspective and capabilities, contemporary environmental engineers pursue course work and postgraduate training in professional areas including biology, chemical engineering, chemistry, and hydrology. Since the environmental engineer is now dealing with sensitive public issues, training in public education, public policy, and other social sciences is desirable.

The principal environmental engineering specialties are well established: air quality control, water supply management, wastewater disposal, storm water management, solid waste management, and hazardous waste management. Other specialties include industrial hygiene, noise control, oceanography, and radiology.

Principal areas of employment for practicing environmental engineers include consulting, industry, and government. Other environmental engineers work in the academic community or direct the development and production of equipment.

After satisfying experience and testing requirements, the environmental engineer obtains professional engineering registration. Professional associations of interest to the environmental engineer include the American Academy of Environmental Engineers, the Air and Waste Management Association, the American Water Works Association, the Water Environment Federation, and the Solid Waste Association of North America.

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Consulting

Consulting engineers are professionals experienced in applying their knowledge and experience to the benefit of those who have retained their services. The services to be provided by an environmental engineering consultant may range from studies to preliminary design, final design, construction, and operation and management services. Additional major areas of service include site evaluations, environmental impact studies and assessments, assistance in obtaining permits, and expert witness services. Consulting environmental engineers typically work for local, state, and regional government, the federal government, industry, and trade and professional organizations.

Industry

The industrial sector of the economy has a substantial demand for environmental engineering professionals due to the requirement to comply with statutes and regulations of federal, regional, and state pollution control agencies. Typically, industry may employ environmental engineers at their headquarter, division, and plant levels, as well as in liaison roles directly with legislative and executive agencies.

Typical of responsibilities of environmental engineers at all levels are assistance in planning and development. For example, wetlands may exhibit environmental impacts from industrial expansion, which could slow or halt necessary construction. Other typical environmental engineering activities are preparation and negotiation of permits from regulatory agencies, responsibility for operation of and reporting associated with existing pollution control systems, and development of modifications to existing pollution control systems consistent with production changes.

Government

Environmental engineers in public service provide technical expertise in all levels of government.

Local Government. The environmental engineer's role in local government may include such tasks as assistance in the development of local ordinances; administration of a pretreatment program, including inspection and compliance monitoring; responsibility for the municipal or local wastewater treatment plant operation; responsibility for the operation and administration of the solid waste collection and disposal operations; administration of the local air pollution ordinances; and interfacing with state and federal officials as required on environmental matters.

State Government. Considering the current trend in government, it is reasonable to expect that the environmental engineering staff at the state level will increase, in some cases dramatically. This is due primarily to the federal government's stated intent to have the states perform the primary role in executing the environmental laws of the nation. This will increase the employment of environmental engineers in a variety of roles, including field inspectors, regional/divisional engineers, and administrative staff.

A tabulation of addresses and telephone numbers for state and territorial environmental protection agencies is presented in Table 1.1.

Regional Government. The role of the environmental engineer in regional agencies is similar to that of the state's role except that there may also be significant interactions with other states. A list of major interstate environmental agencies is presented in Table 1.2.

Federal Government. The environmental engineer in the federal government is involved primarily with research and development, development of regulations, and enforcement of regulations. The most widely known agency that affects environmental engineers is the U.S. Environmental Protection Agency (EPA). The

TABLE 1.1 State and Territorial Environmental Protection Agencies (*1, 2, 3*)

Alabama Dept. of Environmental Management 1751 Federal Dr. Montgomery, AL 36130 205-271-7700	Georgia Dept. of Natural Resources 205 Butter St. Atlanta, GA 30334 404-656-4713
Alaska Dept. of Environmental Conservation PO Box 'O' Juneau, AK 99811 907-465-2600	Guam Environmental Protection Agency Box 2999 Agana, GU 96910 671-646-8863
Arizona Dept. of Environmental Quality 2005 North Central Ave. Phoenix, AZ 85004 602-257-2300	Hawaii Dept. of Land and Natural Resources Box 621 Honolulu, HI 96809 808-548-6550
Arkansas Dept. of Pollution Control and Ecology 8001 National Drive, PO Box 9583 Little Rock, AR 72219 501-562-7444	Idaho Dept. of Health and Welfare 450 West State Boise, ID 83720 208-334-4107
California Air Resources Board 1102 Q Street, PO Box 2815 Sacramento, CA 95812 916-322-2990	Illinois Environmental Protection Agency 2200 Churchill Road Springfield, IL 62706 217-782-3397
California Dept. of Water Resources 1416 Ninth St Sacramento, CA 94236 916-445-9248	Indiana State Board of Health PO Box 1964 Indianapolis, IN 46206 317-633-8400
Colorado Dept. of Health 4210 E. 11th Ave Denver, CO 80220 303-320-8333	Iowa Dept. of Natural Resources 900 E Grand Ave Des Moines, IA 50319 515-281-8666
Connecticut Dept. of Environmental Protection 165 Capitol Ave Hartford, CT 06115 203-566-5524	Kansas Dept. of Health and Environment Forbes Field, Building 740 Topeka, KA 66620 913-862-9360
Delaware Dept. of Natural Resources and Environmental Control Tatnall Building PO Box 1401 Dover, DE 19901 302-736-4403	Kentucky Dept. for Natural Resources and Environmental Protection Capital Plaza Tower Frankfort, KY 40601 502-564-2041
District of Columbia Dept. of Environmental Services 5010 Overlook Avenue, SW Washington, DC 20032 202-767-7486	Louisiana Dept. of Environmental Quality 625 North Fourth St Baton Rouge, LA 70804 504-342-1265
Florida Dept. of Environmental Regulation 2600 Blair Stone Rd Tallahassee, FL 32301 904-488-1344	Maine Dept. of Environmental Protection State House Augusta, ME 04333 207-289-2811

(continues)

TABLE 1.1 State and Territorial Environmental Protection Agencies (*continued*)

<p>Maryland Dept. of Environment 201 W Preston St Baltimore, MD 21202 301-225-5750</p>	<p>New Mexico Environment Improvement Division PO Box 968 Santa Fe, NM 87503 505-827-5271</p>
<p>Massachusetts Division of Environmental Quality Control 1-11 Winter St Boston, MA 02110 617-292-5856</p>	<p>New York Dept. of Environmental Conservation 50 Wolf Road Albany, NY 12223 518-457-3446</p>
<p>Michigan Dept. of Natural Resources PO Box 30028 Lansing, MI 48909 517-373-1220</p>	<p>North Carolina Division of Environmental Management PO Box 27687 Raleigh, NC 27611 919-733-7015</p>
<p>Minnesota Pollution Control Agency 520 Lafayette Rd St. Paul, MN 55155 612-296-6300</p>	<p>North Dakota State Dept. of Health 1200 Missouri Ave Bismarck, ND 58505 701-224-2348</p>
<p>Mississippi Dept. of Natural Resources 2380 Highway 80 West Jackson, MS 39209 601-961-5171</p>	<p>Ohio Environmental Protection Agency PO Box 1049 Columbus, OH 43266 614-481-7000</p>
<p>Missouri Dept. of Natural Resources 2010 Missouri Blvd Jefferson City, MO 65101 314-751-3332</p>	<p>Oklahoma Dept. of Health PG Box 53551 Oklahoma City, OK 73152 405-271-5600</p>
<p>Montana Dept. of Health and Environmental Sciences Cogswell Building Helena, MT 59601 406-449-2544</p>	<p>Oregon Dept. of Environmental Quality PO Box 1760 Portland, OR 97207 503-229-5696</p>
<p>Nebraska Dept. of Environmental Control 301 Centennial Mall South PO Box 94877 Lincoln, NB 68509 402-471-2186</p>	<p>Pennsylvania Dept. of Environmental Resources PO Box 2063 Harrisburg, PA 17120 717-787-2814</p>
<p>Nevada Division of Environmental Protection 201 South Fall St Carson City, NV 89710 702-8854670</p>	<p>Puerto Rico Dept. of Health PG Box 70184 San Juan, PR 00922 809-722-2050</p>
<p>New Hampshire Dept. of Environmental Services Hazen Drive Concord, NH 03301 603-2714582</p>	<p>Rhode Island Dept. of Environmental Management 9 Hayes Street Providence, RI 02908 401-277-6800</p>
<p>New Jersey Dept. of Environmental Protection 401 E. State Street Trenton, NJ 08625 609-292-2885</p>	<p>South Carolina Dept. of Health and Environmental Control 2600 Bull Street Columbia, SC 29201 803-758-5654</p>

TABLE 1.1 State and Territorial Environmental Protection Agencies (*continued*)

South Dakota Dept. of Water and Natural Resources Joe Foss Building Pierre, SD 57501 605-773-3329	Virginia Council on the Environment Ninth Street Office Building Richmond, VA 23219 804-786-4500
Tennessee Water Quality Control Board 150 North Ave Nashville, TN 37203 615-741-3657	Virginia State Water Control Board 2111 N. Hamilton St Richmond, VA 23230 804-257-0056
Texas Dept. of Health 1100 W. 49th St Austin, TX 78756 512-458-7111	Washington State Dept. of Ecology M.S. PV-11 Olympia, WA 98504 206-753-2821
Texas Water Development Board PO Box 13231 Austin, TX 78711 512-463-7847	West Virginia Air Pollution Control Commission 1558 Washington St, East Charleston, WV 25311 304-348-3286
Utah Dept. of Health 150 W North Temple PO Box 16700 Salt Lake City, UT 84116 801-538-6111	West Virginia Dept. of Natural Resources Capital Complex Charleston, WV 25311 304-348-2107
Vermont Agency of Environmental Conservation State Office Building Montpelier, VT 05602 802-828-3130	Wisconsin Dept. of Natural Resources Box 7921 Madison, WI 53707 608-266-2621
Virgin Islands Dept. of Conservation PO Box 4340 St. Thomas, VI 00801	Wyoming Dept. of Environmental Quality 401 W. 19th St Cheyenne, WY 82002 307-777-7937

headquarters' organization is presented in Fig. 1.1, and a listing of EPA regional office addresses is given in Table 1.3. The laboratories and research centers operated by EPA are listed in Table 1.4, along with a description of their functions and activities.

This agency's mandate is to act as the chief representative of the executive department in protecting the nation's environment. This responsibility includes surface water, groundwater, land and geological resources, and air resources.

There are environmental engineers employed in virtually every major organization within the federal government, including the military and the Departments of Commerce, Energy, Interior, and Agriculture. The environmental engineers' roles in these agencies parallel those in the EPA, with perhaps more emphasis on nonenforcement activities.

International Agencies. There are many international agencies, organizations, and institutions that provide project assistance to foreign companies and governments. Typical of these organizations are the Agency for International Development (Department of State), the World Bank Group of International Financial Agencies, the Inter-American Development Bank, and the Export-Import Bank. Although the degree of interest in environmental protection varies considerably from country to country, there are usually significant efforts in environmental planning; environmental impact analysis; and the water, air, and waste management development phases of projects that require significant environmental impact analysis.

TABLE 1.2 Major Interstate Environmental Agencies

Great Lakes Commission

(New York, Pennsylvania, Ohio, Michigan, Indiana, Illinois, Wisconsin, Minnesota)

5104 IST Bldg.
2200 Bonisteel Blvd.
Ann Arbor, MI 48109

Interstate Commission on the Potomac River Basin

(District of Columbia, Maryland, Pennsylvania, Virginia, West Virginia, U.S. federal government)

6110 Executive Blvd.
Rockville, MD 20852

New England Interstate Water Pollution Control Commission

(Connecticut, Maine, Massachusetts, New Hampshire, New York, Rhode Island, Vermont)

85 Merrimac St.
Boston, MA 02114

Ohio River Valley Water Sanitation Commission

(Illinois, Indiana, Kentucky, New York, Ohio, Pennsylvania, Virginia, West Virginia)

49 E. Fourth St.
Cincinnati, OH 45202

Susquehanna River Basin Commission

(New York, Pennsylvania, Maryland)

Interior Bldg.
Washington, D.C. 20240

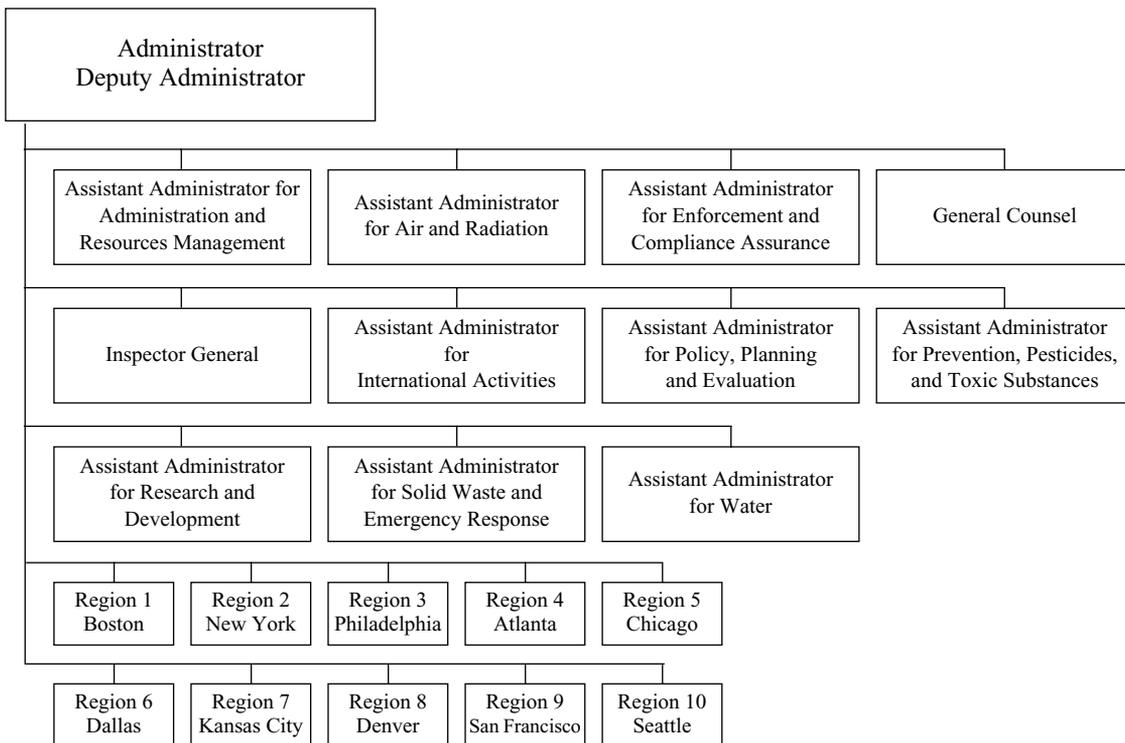


FIG. 1.1 U.S. Environmental Protection Agency Organization

TABLE 1.3 U.S. Environmental Protection Agency Offices

Address and Telephone Number	States and Territories
<i>EPA Headquarters</i> 401 M Street, SW Washington, DC 20460 202-382-4700	
<i>United States EPA, Region 1</i> John F. Kennedy Federal Building Boston, MA 02203 617-565-4502	Connecticut, Maine, Massachusetts, New Hampshire, Rhode Island, Vermont
<i>United States EPA, Region 2</i> 290 Broadway New York, NY 10007-1866 212-637-4013	New Jersey, New York, Puerto Rico, Virgin Islands
<i>United States EPA, Region 3</i> 841 Chestnut Building Philadelphia, PA 19107 215-566-5000	Delaware, District of Columbia, Maryland, Pennsylvania, Virginia, West Virginia
<i>United States EPA, Region 4</i> 61 Forsyth Street, SW Atlanta, GA 30303 404-562-9900	Alabama, Florida, Georgia, Kentucky, Mississippi, North Carolina, South Carolina, Tennessee
<i>United States EPA, Region 5</i> 77 W. Jackson Blvd. Chicago, IL 60604 312-353-2205	Illinois, Indiana, Michigan, Minnesota, Ohio, Wisconsin
<i>United States EPA, Region 6</i> First Interstate Bank Tower 1445 Ross Avenue Dallas, TX 75202 214-665-7223	Arkansas, Louisiana, New Mexico, Oklahoma, Texas
<i>United States EPA, Region 7</i> 726 Minnesota Avenue Kansas City, KS 66101 913-551-7020	Iowa, Kansas, Missouri, Nebraska
<i>United States EPA, Region 8</i> 999 18th Street, Suite 500 Denver, CO 80202-2466 303-312-6312	Colorado, Montana, North Dakota, South Dakota, Wyoming, Utah
<i>United States EPA, Region 9</i> 75 Hawthorne Street San Francisco, CA 94105 415-744-1500	Arizona, California, Hawaii, Nevada, American Samoa, Guam, Trust Territories of the Pacific
<i>United States EPA Region 10</i> 1200 6th Avenue Seattle, WA 98101 206-553-1200	Alaska, Idaho, Oregon, Washington

TABLE 1.4 U. S. Environmental Protection Agency Laboratories and Research Centers

National Air and Radiation Environmental Laboratory (NAREL) (Montgomery, Alabama)—comprehensive environmental laboratory for measuring environmental radioactivity and evaluating its risk to the public.

National Center for Environmental Assessment—serves as the national resource center for the overall process of human health and ecological risk assessments; the integration of hazard, dose-response, and exposure data and models to produce risk characterizations. Divisions that include:

- *Washington, DC Office*—provides exposure/risk characterization, hazard identification, dose-response, and administrative and budgetary support.
- *Cincinnati Ohio Office*—agent-specific risk assessment and technical assistance.
- *Research Triangle Park, North Carolina Office*—primarily develops and publishes air quality criteria documents for major air pollutants, provides health and ecological assessments of air toxics and assessments and scientific assistance on fuels/fuel additives.

National Center for Environmental Research and Quality Assurance (NCERQA) (Washington, DC)—has primary responsibility to issue and manage research grant and fellowship programs, as well as issues of quality assurance, and peer review.

National Enforcement Investigations Center—Laboratory Branch (Denver, Colorado) provides facilities, equipment, personnel, and expertise needed for measurement activities, data evaluations, and investigations conducted to support civil and criminal environmental enforcement efforts.

National Exposure Research Lab (NERL) (Research Triangle Park, North Carolina)—provides scientific understanding, information and assessment tools to reduce and quantify the uncertainty in the Agency's exposure and risk assessments for all environmental stressors. Divisions include:

- *Atmospheric Sciences Modeling Division* (Research Triangle Park, North Carolina)—performs much of the air pollution meteorology research for the Agency, conducts in-house research, and manages numerous cooperative agreements and contracts.
- *Microbiological and Chemical Exposure Assessment Research Division (MCEARD)* (Cincinnati, Ohio)—conducts research to measure, characterize, and predict the exposure of humans to chemical and microbial hazards.
- *Ecosystems Research Division* (Athens, Georgia)—conducts research on organic and inorganic chemicals, greenhouse gas biogeochemical cycles, and land use perturbations that create direct and indirect, chemical and nonchemical stressor exposures and potential risks to humans and ecosystems.
- *Environmental Sciences Division (ESD)* (Las Vegas, Nevada)—conducts research, development, and technology transfer programs on environmental exposures to ecological and human receptors. ESD develops methods for characterizing chemical and physical stressors, with special emphasis on ecological exposure.

National Health and Environmental Effects Research Lab (NHEERL) (Research Triangle Park, North Carolina) the agency's focal point for scientific research on the effects of contaminants and environmental stressors on human health and ecosystem integrity. NHEERL divisions include:

- *Environmental Carcinogenesis Division*—conducts mechanistically based research to explain the association between environmental pollution and cancer.
- *Experimental Toxicology Division*—responsible for pharmacokinetics research; examines pulmonary, immunological, hepatic, cardiovascular, and renal toxicity resulting from environmental contaminants.
- *Human Studies Division*—conducts clinical and epidemiological investigations to better understand human response to pollution.
- *Neurotoxicology Division*—studies the effects of chemical and/or physical agents on the nervous system.
- *Reproductive Toxicology Division*—evaluates the effects of environmental pollutants on human reproductive potential/competence and development.
- *Atlantic Ecology Division* (Narragansett, RI)—studies the effects of contaminants and other stressors on the coastal waters and watersheds of the Atlantic seaboard.
- *Gulf Ecology Division* (Gulf Breeze, FL)—assesses the condition of coastal ecosystems (wetlands, bays, estuaries, and coral reefs) in the Gulf of Mexico and analyzes causes of change to ecological status.

TABLE 1.4 U. S. Environmental Protection Agency Laboratories and Research Centers (*continued*)

- *Mid-Continent Ecology Division* (Duluth, MN)—performs research to protect freshwater ecosystems and wildlife and to understand the basic processes and mechanisms involved in aquatic toxicity.
- *Western Ecology Division* (Corvallis, OR)—evaluates the effects of chemical contaminants, land use, and global climate change on terrestrial ecosystems and on watershed ecology along the Pacific coast.
- *Environmental Monitoring and Assessment Program*—a comprehensive, long-term research program designed to assess natural resources and identify causes for damaged environments.

National Risk Management Research Lab (NRMRL) (Cincinnati, Ohio)—advances the scientific understanding and the development and application of technological solutions to prevent, control, or remediate important environmental problems that threaten human health and the environment. NRMRL divisions include:

- *Air Pollution Protection and Control Division* (Research Triangle Park, North Carolina)—this division is responsible for research, development, and evaluation of air pollution control technologies.
- *Subsurface Protection and Remediation Division*—(formerly the Robert S. Kerr Environmental Research Laboratory) (Ada Oklahoma)—conducts research and engages in technical assistance and technology transfer on the chemical, physical, and biological structure and processes of the subsurface environment, the biogeochemical interactions in that environment, and fluxes to other environmental media.
- *Water Supply and Water Resources Division's Urban Watershed Management Branch (UWMB)* (Edison, New Jersey)—is responsible for researching, developing, and demonstrating technologies, systems, and methods required to manage the risk to public health, ecologically sensitive areas, and property from wet-weather flows (WWFs) and petroleum and chemical storage system (PCSS) sources.

National Vehicle and Fuel Emissions Laboratory (Ann Arbor, Michigan)—responsible for developing national regulatory programs to reduce mobile source related air pollution; evaluating emission control technology; testing vehicles, engines, and fuels; and determining compliance with Federal emissions and fuel economy standards.

Radiation and Indoor Environments National Laboratory (Las Vegas, NV)—provides technical support for numerous radiation protection and control activities, conducts site investigations, radon assessments and evaluations, health assessment modeling, and indoor air studies.

Academia

The environmental engineer's role in the academic community is generally in teaching and/or research. The role played by these individuals is instruction in both basic and advanced environmental engineering courses at the undergraduate and graduate levels. Many colleges and universities that offer an environmental engineering course also perform basic and applied research on a contract or grant basis for both the private and governmental sectors.

Professional Societies

There are many professional societies that have as a major emphasis of their program a section or division that focuses on environmental engineering. Principal societies in the United States include the American Society of Civil Engineers, American Institute of Chemical Engineers, Water Pollution Control Federation, Air and Waste Management Association, and American Water Works Association. These and other societies provide an indispensable service through the transfer of information via their periodicals, which provide a forum to discuss experiences and an opportunity to exchange information through scheduled meetings at local, regional, and national levels. There are many other organizations at both the national and international levels that are useful sources of environmental information. A representative sampling of these organizations is presented in Table 1.5.

TABLE 1.5 National and International Environmental Associations (*I, 2*)

Acoustical Society of America 335 E 45 St. New York, NY 10017 212-661-9404	American Society of Civil Engineers 345 E 47th St. New York, NY 10017 212-644-7494
Air and Waste Management Association PO Box 2861 Pittsburgh, PA 15230 412-621-1090	American Water Works Association 6666 W Quincy Ave. Denver, CO 80235 303-794-7711
American Academy of Environmental Engineers 132 Holiday Court Annapolis, MD 21401 301-266-3311	Center for Environmental Education 624 9th St., NW Washington, DC 20001 202-737-3600
American Consulting Engineers Council 1010 15th St, NW, #802 Washington, DC 20005 202-347-7474	Chamber of Commerce of the U.S. 1615 H St., NW Washington, DC 20062 202-659-6000
American Enterprise Institute for Public Policy Research 1150 17th St, NW Washington, DC 20036 202-466-8225	The Conservation Foundation 1717 Massachusetts Ave., NW Washington, DC 20036 202-797-4300
American Industrial Hygiene Association 475 Wolf Ledges Pkwy. Akron, OH 216-762-7294	Council on Economic Priorities 84 Fifth Ave. New York, NY 10011 212-691-8550
American Institute of Chemical Engineers 345 E 47th St. New York, NY 10017 212-705-7338	Edison Electric Institute 1111 19th St., NW Washington, DC 20036 202-828-7400
American Medical Association 535 N. Dearborn St. Chicago, IL 60610 312-751-6000	Electric Power Research Institute PO Box 10412 Palo Alto, CA 94303 415-855-2000
American Nuclear Society 555 N Kensington Ave. La Grange Park, IL 60525 312-352-6611	Environmental Studies Board National Academy of Sciences 2101 Constitution Ave. Washington, DC 20418
American Public Health Association 1015 15th St., NW Washington, DC 20005 202-789-5600	Industrial Gas Cleaning Institute 700 N Fairfax St. Alexandria, VA 22314 703-836-0480
American Public Power Association 2301 M St., NW Washington, DC 20037 202-775-8300	Institute of Environmental Science 940 E Northwest Highway Mount Prospect, IL 60056 312-255-1561
American Public Works Association 1313 E 60th St. Chicago, IL 60637 312-947-2520	

TABLE 1.5 National and International Environmental Associations (1, 2) (continued)

Institute of Noise Control Engineering PO Box 3206 Arlington Branch Poughkeepsie, NY 12603	National Solid Wastes Management Association 1120 Connecticut Ave., NW Washington, DC 20036 202-659-4613
Institute of Scrap Iron and Steel 1627 K St., NW Suite 700 Washington, DC 20006 202-466-4050	National Wildlife Federation 1412 16th St, NW Washington, DC 20036 202-797-6853
Inda—Association of the Nonwoven Fabrics Industry 1700 Broadway, 25th Floor New York, NY 10019 212-582-8401	Natural Resources Defense Council 917 15th St, NW Washington, DC 20005 202-737-5000
International Ozone Association 14701 Detroit Ave. Lakewood, OH 44107 216-228-2166	Noise Control Products and Materials Association Suite 201, Park Place 655 Irving Park Chicago, IL 60613 312-525-2644
National Association for Environmental Education PO Box 400 Troy, OH 45373	Public Citizen, Inc. PO Box 19404 Washington, DC 20036 202-293-9142
National Association of Environmental Professionals PO Box 9400 Washington, DC 20016 301-229-7171	Sierra Club 530 Bush St San Francisco, CA 94108 415-981-8634
National Association of Recycling Industries, Inc. 330 Madison Ave. New York, NY 10017 212-867-7330	Solar Energy Industries Association 1001 Connecticut Ave., NW Suite 800 Washington, DC 20036 202-293-2981
National Center for Resource Recovery 1211 Connecticut Ave., NW Washington, DC 20036 202-223-6154	Water Environment Federation 601 Wythe Street Alexandria, VA 22314 703-684-2400
National Coal Association 1130 17th St., NW Washington, DC 20036 202-628-4322	Water Quality Association 477 E Butterfield Rd. Lombard, IL 60148 312-969-6400
National Safety Council 444 N Michigan Ave. Chicago, IL 60611 312-527-4800	Water and Wastewater Equipment Manufacturers Association 7900 Westpark Dr., Suite 102 McLean, VA 22102 703-893-1520

Engineering Registration

The furtherance and prestige of the environmental engineering profession, like other engineering disciplines, depends upon maintenance of the quality of professional services provided. One of the principal means of assuring professionalism is through the registration of engineers in accredited programs. The National Council of Engineering Examiners, headquartered in Clemson, South Carolina, works with member and affiliate boards throughout the United States in administering a standardized test for engineers and land surveyors. The names, mailing addresses, and telephone numbers of member and affiliate member boards are presented in Table 1.6.

CONTRACTING FOR CONSULTANT SERVICES

Proper selection of and contracting with an environmental engineering consultant is a crucial first step in any project requiring the services of a consulting engineering firm. This section addresses the key issues that are common to both the client and the engineer in order to assure a harmonious relationship and a successful project.

Consultant Services

Consulting environmental engineers can provide clients with a wide variety of services, including:

- Air quality and air pollution control
- Construction management
- Energy development, conservation, and recovery
- Environmental and ecological studies
- Environmental impact analyses
- Facility operation and management
- Hazardous and toxic waste management
- Human settlements
- Industrial waste control and treatment
- Irrigation and agriculture
- Laboratory services
- Marine waste disposal and nearshore oceanography
- Regional water pollution control planning
- Sewage treatment and disposal
- Sludge handling and disposal
- Solid waste management
- Storm drainage and flood control
- Water reclamation and reuse
- Water resources and hydrology
- Water supply, treatment, and distribution

Most of the above project types can be categorized as engineering studies, design services, construction services, or startup and training services. Each of these categories is described later in this chapter.

Consultant Selection

Clients should study the proposed project and their engineering needs so that they understand what is required from and expected of the consulting engineer. If clients have previous experience with consulting engineers who have in the past rendered satisfactory services, they may consider it unnecessary to go through the procedure outlined herein. Otherwise, they should follow a formal selection procedure similar to the one described below.

1. Consider the qualifications of consulting engineers who appear to be capable of meeting the requirements of the project.
2. Consider at least three firms that appear to be best qualified for the particular project. Write separate letters to each of them, describing briefly the proposed project and inquiring as to interest in it. On receipt

TABLE 1.6 Boards of Engineering Registration

<p>National Council of Examiners for Engineering and Surveying (NCEES) PO Box 1686 Clemson, SC 29633-1686 (803) 654-6824</p> <p>Alabama Board of Registration for Professional Engineers and Surveyors 501 Adams Avenue Montgomery, AL 36109 (205) 242-5568</p> <p>Alaska Board of Registration for Architects, Engineers and Land Surveyors PO Box 110806 Juneau, AL 99811-0806 (907) 465-2540</p> <p>Arizona Board of Technical Registration 1951 W Camelback Road, Suite 250 Phoenix, AZ 85015 (602) 255-4053</p> <p>Arkansas Board of Registration for Professional Engineers and Land Surveyors PO Box 2541 Little Rock, AR 72203 (501) 324-9085</p> <p>California Board of Registration for Professional Engineers and Land Surveyors P.O Box 349002 Sacramento, CA 95834-9002 (916) 263-2221</p> <p>Colorado Board of Registration for Professional Engineers and Professional Land Surveyors 1560 Broadway, Suite 1370 Denver, CO 80202 (303) 894-7788</p> <p>Connecticut Board of Examiners for Professional Engineers and Land Surveyors The State Office Building, Room G-3A, 165 Capitol Avenue Hartford, CT 06106 (203) 566-3290</p> <p>Delaware Association of Professional Engineers 2005 Concord Pike Wilmington, DE 19803 (302) 577-6500</p> <p>District of Columbia Board of Registration for Professional Engineers 614 H Street, NW, Room 923 Washington, DC 20001 (202) 727-7454</p>	<p>Florida Board of Professional Engineers 1940 North Monroe Street Tallahassee, FL 32399-0755 (904) 488-9912</p> <p>Georgia Board of Registration for Professional Engineers and Land Surveyors 166 Pryor Street, SW, Room 504 Atlanta, GA 30303 (404) 651-9532</p> <p>Hawaii Board of Registration for Professional Engineers, Architects, Surveyors and Landscape Architects PO Box 3469 Honolulu, HI 96801 (808) 586-2702</p> <p>Idaho Board of Professional Engineers and Land Surveyors 600 S Orchard Street, Suite A Boise, ID 83705 (208) 334-3860</p> <p>Illinois Department of Professional Regulation, State Board of Professional Engineers 320 West Washington Street, 3rd Floor Springfield, IL 62786 (217) 785-0820</p> <p>Indiana Board of Registration for Professional Engineers 302 W Washington St, EO34 Indianapolis, IN 46204 (317) 232-2980</p> <p>Iowa Engineering and Land Surveying Examining Board 1918 S E Hulsizer Ankeny, IA 50021 (515) 281-5602</p> <p>Kansas Board of Technical Professions Landon State Office Building 900 Jackson, Suite 507 Topeka, KS 66612-1214 (913) 296-3053</p> <p>Kentucky Board of Registration for Professional Engineers and Land Surveyors Kentucky Engineering Center 160 Democrat Drive Frankfort, KY 40601 (502) 573-2680</p>
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(continues)

TABLE 1.6 Boards of Engineering Registration (*continued*)**Louisiana Board of Registration for Professional Engineers and Land Surveyors**

1055 St Charles Avenue, Suite 415
New Orleans, LA 70130-3997
(504) 568-8450

Maine Board of Registration for Professional Engineers

State House, Station 92
Augusta, ME 04345
(207) 287-3236

Maryland Board of Registration for Professional Engineers

501 St Paul Place, Room 902
Baltimore, MD 21202
(410) 333-6322

Massachusetts Board of Registration of Professional Engineers and Land Surveyors

Room 1512, Leverett Saltonstall Bldg.
100 Cambridge Street
Boston, MA 02202
(617) 727-3055

Michigan Board of Professional Engineers

PO Box 30018
Lansing, MI 48909
(517) 335-1669

Minnesota Board of Registration for Architects, Engineers, Land Surveyors and Landscape Architects

85 East 7th Street, Suite 160
St Paul, MN 55101
(612) 296-2388

Mississippi Board of Registration for Professional Engineers and Land Surveyors

PO Box 3
Jackson, MS 39205
(601) 359-6160

Missouri Board of Architects, Professional Engineers and Land Surveyors

PO Box 184
Jefferson City, MO 65102
(314) 751-0047

Montana Board of Professional Engineers and Land Surveyors

PO Box 200513
Helena, MT 59620-0513
(406) 444-4285

Nebraska Board of Examiners for Professional Engineers and Architects

PO Box 94751
South Lincoln, NE 68509
(402) 471-2021

Nevada Board of Registered Professional Engineers and Land Surveyors

1755 East Plumb Land, Suite 135
Reno, NV 89502
(702) 688-1231

New Hampshire Board of Professional Engineers

57 Regional Drive
Concord, NH 03301
(603) 271-2219

New Jersey Board of Professional Engineers and Land Surveyors

PO Box 45015
Newark, NJ 07101
(201) 504-6460

New Mexico Board of Registration for Professional Engineers and Surveyors

1010 Marques Place
Santa Fe, NM 87501
(505) 827-7561

New York Board of Engineering and Land Surveying

State Education Department
Cultural Education Center
Madison Avenue
Albany, NY 12230
(518) 474-3846

North Carolina Board of Registration for Professional Engineers and Land Surveyors

3620 Six Forks Road
Raleigh, NC 27609
(919) 781-9499

North Dakota Board of Registration for Professional Engineers and Land Surveyors

PO Box 1357
Bismarck, ND 58502
(701) 258-0786

Ohio Board of Registration for Professional Engineers and Surveyors

77 South High St, 16th Floor
Columbus, OH 43266-0314
(614) 466-3650

TABLE 1.6 Boards of Engineering Registration (*continued*)**Oklahoma Board of Registration for Professional Engineers and Land Surveyors**

Oklahoma Engineering Center, Room 120
201 NE 27th Street
Oklahoma City, OK 73105
(405) 521-2874

Oregon Board of Engineering Examiners

750 Front Street, NE, Suite 240
Salem, OR 97310
(503) 3784180

Pennsylvania Registration Board for Professional Engineers

P.O. Box 2649
Harrisburg, PA 17105-2649
(717) 783~7049

Rhode Island Board of Registration for Professional Engineers

Charles Orms Bldg.
10 Orms Street, Suite 324
Providence, RI 02904
(401) 277-2565

South Carolina Board of Registration for Professional Engineers and Land Surveyors

PO Box 50408
Columbia, SC 29250
(803) 734-9166

South Dakota Commission of Engineering and Architectural and Land Surveying Examiners

2040 West Main Street, Suite 304
Rapid City, SD 57702-2447
(605) 394-2510

Tennessee Board of Architectural and Engineering Examiners

Dept of Commerce and Insurance
3rd Floor, Volunteer Plaza
500 James Robertson Parkway
Nashville, TN 37243-1142
(615) 741-3221

Texas Board of Registration for Professional Engineers

PO Drawer 18329
Austin, TX 78760
(512) 440-7723

Utah Committee for Professional Engineers and Land Surveyors

PO Box 45805
Salt Lake City, UT 84145
(801) 530-6628

Vermont Board of Registration for Professional Engineers

Office of Professional Regulation
109 State Street
Montpelier, VT 05609-1106
(802) 828-2875

Virginia Board of Architects, Professional Engineers, Land Surveyors and Landscape Architects

3600 W. Broad Street
Richmond, VA 23230-4917
(804) 367-8514

Washington Board of Registration for Professional Engineers and Land Surveying

PO Box 9649
Olympia, WA 98504
(206) 753-6966

West Virginia Board of Registration for Professional Engineers

608 Union Bldg
Charleston, WV 25301
(304) 558-8554

Wisconsin Examining Board of Architects, Professional Engineers, Designers, and Land Surveyors

PO Box 8935
Madison, WI 53708-8935
(608) 266-1397

Wyoming Board for Professional Engineers and Professional Land Surveyors

Herschler Building, Room 4135
E Cheyenne, WY 82002
(307) 777-6155

of an affirmative answer, request the firm to appear for a separate personal interview. In the case of design projects, give the consultant an opportunity to inspect the site, explaining the proposed services required. At the interviews, go over the qualifications and records of each firm, its capability to complete the work within the time allotted, and the specific key personnel assignable to the project.

3. Check carefully with recent clients of each firm and determine the quality of performance. Do not limit this check to references specified by the consulting firm.
4. Request technical proposals and/or presentations from those firms considered to be most qualified for the work. Rate structures and/or estimates of engineering costs may be requested by the client as an aid to evaluating probable costs. However, firms that are better qualified will almost always be able to complete a project at a lower total cost than less-qualified firms, even if the less-qualified firms have lower rates.
5. List the firms in the order of their desirability, taking into account their location, reputation, experience, financial standing, size, personnel available, quality of references, work load, and any other factors peculiar to the project being considered.
6. Invite the firm that is considered to be the best qualified to appear for a second interview to discuss the project further and to negotiate the questions of compensation and other contractual aspects. This procedure is similar to that now used by federal agencies and to that recommended to state and local governments by the American Bar Association.

Teaming Agreement

Some environmental projects require the talents and experience of a multidisciplinary project team which may not be available from a single firm. The most common way of overcoming this problem is for the client to engage a team consisting of personnel, facilities, and equipment from two or more firms with complementary capabilities. Such teaming agreements may be in the form of consulting agreements, subcontracts, or joint ventures.

The client should always be aware of any such agreements to be used on the project. In most cases, the client should be involved in the selection and should approve the final choices. Some criteria for establishing successful subcontracts include the following (5):

- Have the firms involved worked together before? If so, what were the results?
- How are they generally regarded by other engineering consultants?
- Have they ever worked for this client before? With what results?
- Who are the individuals that will be working on the project? How qualified are they?
- What is the track record of each firm in meeting commitments?
- What logistical problems will be encountered between the various firms, e.g., distance, differences in accounting procedures, or differences in organizational structure?

In all teaming arrangements, a prime consultant should be responsible for the performance of all subcontractors and consultants just as much as any member of the prime consultant's own staff. Each firm must also have the highest level of commitment to meeting the project objectives.

Contract Types

A variety of contract types have been used between clients and consultants. These agreements can be generally categorized as fixed price, percent of construction cost, time and expense, cost plus fixed fee, and basic. Specific aspects and applications for each of these contract types are described below (6).

Fixed Price. Fixed-price contracts may be of several types, depending upon the particular circumstances. In general, fixed-price contracts provide for a firm price under which the contractor bears the full responsibility for profit or loss. However, fixed-price contracts may provide for adjustable prices under appropriate circumstances by providing ceiling prices or target prices (including target costs). Fixed-price contracts are generally used where reasonably definite design or performance specifications are available and fair and reasonable prices can be estimated and established.

Percent of Construction Cost. This type of contract establishes the consultant's compensation as a specified percentage of the construction cost, defined either as the low bid or the final construction cost (including change orders). The concept of percent of construction cost contracts is that the engineering effort is generally proportional to construction costs.

Percent of construction cost contracts are not acceptable on U.S. government projects because (1) the compensation is not tied directly to the services being provided; (2) the contractor has no incentive to keep construction costs low; and (3) the construction cost can be affected to a significant degree by unpredictable factors such as number of contractors bidding, availability of local construction labor, and general economic conditions. These problems can be overcome by basing the amount of compensation on the estimated construction cost rather than the actual construction cost. Fee changes are then negotiated only if the scope of work changes.

Time and Expense. The time and expense type of contract provides for payment to the consultant on the basis of

1. Specified fixed hourly rates for direct labor hours (rates to include wages, overhead, general and administrative expenses, and profit)
2. Material at cost (plus material handling charges and indirect costs, including general and administrative costs allocated to direct materials to the extent that they are not included in the fixed hourly rate)

The use of this type of contract is commonly used in the procurement of

- Engineering design services in connection with the procurement of supplies
- Engineering design of equipment components
- Work to be performed in emergencies

Cost Plus Fixed Fee (CPFF). The CPFF contract is a cost-reimbursement type of contract that provides for the payment of a fixed fee to the consultant irrespective of the allowable and allocable costs incurred. The fee (profit) dollars can change only when the scope of the work under the contract changes. Notice must be given by the consultant of any expected overrun of estimated costs.

This type of contract is suitable for the performance of research or preliminary exploration or study where the level of contractor effort required is unknown, or for development and test work where the use of a time and expense contract is not practicable. CPFF contracts are used when the work specifications cannot be defined exactly, and the uncertainties involved in performance are so great that neither a firm price nor an incentive arrangement can be established during the life of the contract.

The contract may be either for "completion" of a clearly defined task or goal or for a specific "term" of time and level of effort. In the completion type of CPFF contract, if the scope of work under the contract cannot be completed within the estimated costs, the client may elect to provide for additional costs and require further performance without any increase in the fee. In the term type of CPFF contract, the consultant is obligated to provide a specific level of effort within a definite period of time.

Basic Agreements. A basic agreement is not a contract. It is a written instrument of understanding between a consultant and a client which sets forth the negotiated contract clauses which will apply to all procurement contracts entered into between the parties during the term of the basic agreement. Basic agreements are only negotiated when a particular future procurement contract is contemplated between the parties. The basic agreement contains all of the “boilerplate,” and each individual formal contract will provide for the scope of work, price, delivery, and other items necessary for the specific procurement contract.

Contract Provisions

The following checklist contains the essential provisions to be considered in preparing any agreement for engineering services, whether executed in precise legal form or by exchange of letters (7):

1. Effective date of agreement.
2. Names and descriptions of the parties to the agreement with their addresses and, in the case of a corporate body, the legal description of the incorporation. If the client is a commission or public body, the authority under which it acts and the source of available funds should be identified.
3. Acknowledgment of the engineer’s knowledge and understanding of the project and interview with the client.
4. Nature, extent, and character of the project; the location thereof; and the time limitations.
5. Services to be rendered by the consulting engineer.
6. Services to be rendered by the client.
7. Provisions for the termination of the engineer’s services before final completion of the work.
8. Compensation for services rendered by the consulting engineer, including times and methods of payments on account, interim payments, and final payment in full settlement.
9. Provisions for penalties for delayed payments.
10. Additional compensation: for redesign, after approval of preliminary phase documents; for change in scope of project; for preparation of alternate designs, drawings, and construction documents; for delays causing expense to the consulting engineer, etc.
11. Time schedule for execution of engineering services.
12. Specifications regarding what constitutes approval of the engineer’s work by the client.

Partnering

Partnering is a process that mitigates confrontational and litigious relationships in the design/construction industry by building a focused and cooperative project work environment among project stakeholders. Stakeholders may become partners when they exhibit mutual desire to make the project work and share commitment to common goals. (14–19)

The partnering concept is based on principles of honesty, integrity, fairness, respect for others and their needs, trust, personal accountability, and the pursuit of excellence. The U.S. Army Corps of Engineers, the Associated General Contractors of America, and others have developed materials describing critical elements of effective partnering programs. (20–25)

A win–win outcome for all stakeholders is the ultimate goal of partnering. Stakeholders may be owners, designers, and contractors or other individuals or organizations with vested interests (tangible or intangible) in the success of the project. When win–lose strategies and tactics are used by one or more stakeholders to gain an advantage over the other stakeholders, experience has shown that a lose–lose situation results for everyone involved. Common results are projects with quality and functional degradation, unreasonable cost growth, and delayed completions.

Effective teams created by partnering will accomplish shared goals by exhibiting mutual trust and respect

TABLE 1.7 Basic Steps in the Project Partnering Process

Step 1—Preparation for Partnering
<ul style="list-style-type: none"> • Invite other stakeholders to participate • Choose a facilitator for the initial workshop • Plan workshop logistics
Step 2—Partnering at Initial Workshop
<ul style="list-style-type: none"> • Develop team-building concepts • Create means and methods to resolve conflict • Discuss and reach a consensus on solutions to critical issues • Create process to evaluate team's progress • Develop a shared mission and goals (partnering charter) • Commit individually by signing partnering charter
Step 3—Continuous Improvement after Workshop
<ul style="list-style-type: none"> • Meet periodically at various team levels to discuss issues leading to win–win solutions • Evaluate team achievements • Recognize and reward individual and team successes • Identify lessons learned and evaluate the project and the partnering process

and by exercising effective communication to ensure an understanding of expectations and values. The shared goals should be “smart” goals: specific *measurable*, *achievable*, *reasonable*, and *timely*. Expected stakeholder benefits include improved efficiency and cost effectiveness, increased opportunity for innovation in problem solving, reduced decision making time, decreased stakeholder risks, and the continuous improvement of quality products and services for all project shareholders.

A key component of effective partnering is a well facilitated workshop. The facilitator must be neutral and objective while serving as the catalyst to cultivate stakeholder partnering. Also, the facilitator may assist or be required as part of partnership maintenance activities, including follow-up workshops or other stakeholder exercises.

The three basic steps in the project partnering process (Table 1.7) are:

1. Preparation for partnering, focusing on organization of stakeholders for the initial facilitated workshop
2. Partnering at initial workshop providing stakeholders all the tools for a mutually successful project
3. Continuous improvement after the workshop, recognizing achievements and providing continued support for win–win solutions

Partnering has demonstrated success for a wide variety of project undertakings and should be considered as an integral part of every organization's quality improvement program. The partnering process may be applied on a small scale to find team solutions to specific problems or to develop a systematic approach to achieving personal and professional goals.

PROJECT MANAGEMENT

Project management is the utilization of all available resources to accomplish the project's objectives. Unlike the medical and legal professions, which rely primarily on personal service, engineering firms usually perform projects requiring a mix of skills from a variety of design professionals. This has resulted in a trend among engineering firms to increase the authority and responsibility of the project manager. While the internal organization of each engineering firm is unique, most firms generally conform to the principles presented in this section.

The Project Manager

The project manager is, by this definition, the individual with day-to-day control of the project. This includes supervision of the technical aspects, control of the budget and schedule, and relationship with the client. In some firms, these responsibilities are divided among two or more persons. However, the trend among most engineering firms today is to assign all these responsibilities to a single individual. This is known as the strong project manager organization and is the form that will be discussed in the remainder of this section.

A project manager in a strong project manager organization has seven basic roles (5):

1. Technical supervision
2. Planning the project
3. Organizing the project team
4. Directing the project team
5. Controlling technical quality, costs, and schedule
6. Financial management
7. Marketing assistance

Position descriptions for project managers also vary from firm to firm according to their particular organizational structure. Table 1.8 is a typical position description for a project manager in a strong project manager organization (5). The following is a description of some common approaches used to plan and monitor a variety of environmental projects.

Planning the Project

The longer and more complex the project, the more time should be spent planning it. This is because of the difficulty in realizing loosely defined objectives that extend over a long period of time.

The major purpose of planning, therefore, is to divide broad contractual goals into manageable tasks that can be performed in relatively short periods of time. Proper planning also provides the project manager with a yardstick to use in measuring progress and controlling the project, as well as avoiding, or at least mitigating, crises that may occur during the execution of the project.

The Work Plan. A good work plan is not a report. It is a compilation of tables and figures, with little or no text, that can be used as a ready reference document throughout the project. The work plan should define:

- What is to be done.
- When it is to be done.
- How much it will cost.
- Who will do it.

Every project should have a work plan; the level of detail is proportional to the size and complexity of the project. For small, simple projects, a single-page work plan, such as that shown in Table 1.9, should be sufficient, while a comprehensive document, such as the one outlined in Table 1.10, may be required for a major design project.

The Task Outline. A good task outline consists of a brief summary of the scope of work required for fulfillment of the contract. The following criteria can be used to evaluate the suitability of a task outline:

1. Does the task outline contain all the deliverables required by the contract?
2. Can the same task outline be used to establish the project schedule?

TABLE 1.8 Position Description for a Project Manager in a Strong Project Manager Organization (5)

POSITION: Project Manager

ACCOUNTABLE TO: Project Director

RESPONSIBLE FOR: Communication with client's representative and team members

Major Responsibility

Directs the project throughout its various phases by continuing and timely communication with client's representative and the team members.

Additional Minimum Duties

1. Assists during promotional phase as required.
2. Assists in organization of project providing input to the project director.
3. Works with all involved disciplines: prepares estimated manpower requirements, schedules, and other pertinent data, participates with project director in preparing fee proposals, conducting fee negotiations, preparing contractual agreements; and is sufficiently familiar with all agreements between firm and client to effectively manage the project in a professional and economic manner.
4. Ascertains that code checks are made and zoning status is acceptable and coordinates with proper building officials.
5. Arranges for timely submission of documents to Quality Control Board for in-house and client milestone progress reviews. Submittal documents should be dated and initialed by project manager signifying his or her review and the coordination of all disciplines.
6. Coordinates and conducts reviews of the project with discipline directors or their assigned representatives on a timely basis to ensure proper coordination and beneficial input from team members.
7. Interviews and contracts with required consultants, subject to review and approval of the project director.
8. Obtains approvals and decisions from client in a timely manner that allows the project to flow smoothly and quickly through the office and alerts project director to any changes of scope, lack of information, or decisions from client that affect schedule or production costs. Negotiates or assists in negotiations with client to account for such changes.
9. Mediates disagreements between disciplines and makes decisions always in regard to the best interests of the project and the firm.
10. Participates in periodic reviews of the project with respect to schedule, construction cost, and profit plan; prepares progress report for biweekly job status meeting.
11. At the closeout of the project, prepares project history data, for example, construction cost analysis, special design features, and problem feedback for future jobs.
12. Turns all documents over to librarian for entry into the firms record storage, including project files, accounting files, and original construction documents.

Description

This is a 100-employee firm in the South offering architectural, structural, civil, electrical, mechanical, and interior design services.

General Responsibility

A project manager is an experienced analyst who can define a project, develop a set of tasks to accomplish the project, coordinate and monitor the work involved in the tasks, and deliver a final product on time and within prescribed cost constraints.

Project managers are designated by the president upon the initiating recommendation of the group manager and approving recommendation of the department manager. This designation, while resulting from specific qualifications the individual has attained, will always contain some amount of subjective judgment. To be designated as a project manager, an analyst should generally possess the qualities of maturity, judgment, leadership, experience, and project success.

(continues)

TABLE 1.8 Position Description for a Project Manager in a Strong Project Manager Organization (*continued*)**Guidelines**

The following guidelines should be used as an aid in selecting individuals for designation as project managers. They can also be used by junior and midlevel analysts as a qualification *guide* to achieving the project manager position. These guidelines are *not* intended to be rigid requirements that must be met without fail.

1. Experience in work field: at least 3 years.
2. Experience of at least 1 year in the company.
3. Experience coordinating technical efforts of others.
4. Project management experience: manage from conception to completion at least one significant project, that is, of more than 6 months in duration and dollar value exceeding \$30,000, specifically.
5. Ability to work on and manage a variety of technical tasks.
6. Ability to make formal presentations to company management and customers.
7. Ability to estimate costs associated with technical tasks.
8. Sales success.

Specific Responsibilities

1. Defining the scope of individual projects.
2. Developing integrated tasks to accomplish individual projects.
3. Coordinating/monitoring the technical work of each task.
4. Assigning personnel within own group.
5. Recommending salaries for group personnel for higher authority approval.
6. Reviewing (for group manager approval) all reports prepared for each project.
7. Monitoring progress toward deliverable schedules.
8. Working closely with finance and contracts personnel to ensure fulfillment of contract requirements.

Personnel

1. Planning/evaluating staffing requirements for each project.
2. Coordinating technical efforts of all project members.

Marketing

1. Developing and maintaining trust and confidence of customers.
2. Seeking follow-on business for each project.
3. Making formal presentations to company management and customers.

TABLE 1.9 Work Plan for a Simple Project (a Biological Wastewater Treatability Study)

Task identification	Responsibility	Budget, (labor-hours)	Target date	Actual date
1. Prepare experimental plan	R. Shaw	24	4/1	4/2
2. Acclimate seed organisms	J. Baker	32	4/21	4/21
3. Operate reactor under steady-state conditions	J. Baker	100	6/15	6/12
4. Conduct solids settling tests	J. Mackey	16	6/20	6/22
5. Reduce/evaluate treatability data	R. Shaw	32	7/4	7/4
6. Prepare draft report	R. Shaw	60	7/22	
7. Review draft report	G. Patrick Client	16	8/15	
8. Finalize report	R. Shaw	24	9/1	

TABLE 1.10 Typical Work Plan Outline for a Major Design Project (5)

Section 1.	Project Definition
	1.1 Copy of contract scope
	1.2 Task outline
	1.3 Preliminary list of drawings
	1.4 Preliminary list of specifications
Section 2.	Schedule
	2.1 CPM diagram
	2.2 Bar chart
	2.3 Milestone chart
Section 3.	Budget
	3.1 Task budgets
	3.2 Projected expenditure curve
Section 4.	Project Organization
	4.1 Project organization chart
	4.2 Task responsibilities
	4.3 Projected labor requirements
	4.4 Addresses and phone numbers of key participants

3. Can the same task outline be used to establish the project budget?
4. Does each item meet the three criteria required for inclusion in the task outline, i.e., scope, duration, and level of effort?
5. Will the task outline require revision only if the contract is modified?
6. Is the task outline general enough to accommodate routine changes in approach without being modified?

If the answer to any of the above questions is “no,” then more work is needed to finalize the task outline.

A “first cut” and final task outline for a hypothetical project is presented in Table 1.11. Although this project is fictional, it is based on an actual project conducted for the U.S. Department of Energy (DOE). The objective of the project was to evaluate the impact of newly promulgated solid waste disposal regulations on the DOE’s goal of promoting greater use of coal.

A comparison of the first cut and final outlines shows how the six task evaluation criteria are used to simplify the outline. This kind of critical analysis can greatly simplify the task outline, thus reducing the amount of time required to monitor and control the project.

Scheduling. Over the years, hundreds of scheduling systems have been devised to control all types of projects. All have advantages and disadvantages, and each project must be scheduled using a method that is suitable for that particular project’s scope and complexity. The most common scheduling methods used for environmental engineering projects are milestone charts, bar charts, and network diagrams. Examples of these methods are illustrated in Table 1.12 and Figures 1.2 through 1.4, based on the example project defined in Table 1.11.

In recent years, several progressive firms have begun using a technique known as *wall scheduling*. Wall scheduling uses a full wall on which vertical lines are drawn 5” apart, with the space between each pair of lines representing one work week. Horizontal lines spaced 3” apart are used to separate the key individuals on the project team (see Fig. 1.5).

To assemble the full wall schedule, the project manager must develop a list of project tasks, identify who is primarily responsible for each task, and prepare a preliminary milestone chart. Then each task is written

TABLE 1.11 Task Outline for an Environmental Study (5)

First Cut Outline	Final Outline
A. Develop background data	A. Develop background data
1. Coal use data	B1. Select case study sites
2. Physical characteristics	B2. Prepare briefing documents
3. Regulatory considerations	B3. Develop data management plan
4. Other relevant cost impacts	B4. Visit case study sites
B. Conduct case studies	B5. Analyze waste samples
1. Select case study sites	C1. Develop computer cost models
2. Prepare briefing documents	C2. Perform preliminary case study site designs
3. Develop data management plan	C3. Estimate case study disposal costs
4. Visit case study sites	D. Evaluate treatment, recovery, and reuse
a. Compile engineering information	E. Assess cost impacts
b. Identify jurisdictional constraints	F. Evaluate cost impact models
c. Collect and preserve waste samples	G1a. Prepare background data reports
5. Analyze waste samples	G1b. Prepare site visit report
C. Estimate disposal costs for case studies	G1c. Prepare sampling and analyses report
1. Develop computer cost models	G2. Prepare draft report
2. Perform preliminary designs	G3. Prepare final report
3. Estimate costs	H. Project management
D. Evaluate treatment, recovery, and reuse	
1. Waste treatment technologies	
2. Direct waste reuse	
3. Potential for waste recovery	
E. Assess cost impacts	
1. Regional impacts	
2. National impacts	
F. Evaluate cost impact models	
G. Project reporting	
1. Topical reports	
a. Background data	
b. Case study site visits	
c. Waste sampling and analyses	
2. Draft report	
3. Final report	
H. Project management	
1. Budget and schedule control	
2. Liaison with principal-in-charge	
3. Client liaison	
a. Meetings	
b. Periodic progress reports	
c. e-mail	
d. Telephone conversations	
4. Maintenance of project records	
a. Files	
b. Calculations	
c. Meeting minutes	
5. Coordination of activities	
a. Defining manpower requirements	
(1) Type of expertise	
(2) Number of man-hours	
(3) Schedule	
b. Maximizing productivity	
6. Technical quality control	

TABLE 1.12 Milestone Chart for an Environmental Study Project (5)

Project Activity	Responsibility	Target Date	Actual Date
A. Develop background data	PJS	5/1	9/12
B1. Select case study sites	BEB	4/1	3/22
B2. Prepare briefing documents	DSF	2/1	3/6
B3. Develop data management plan	DSF	4/1	3/22
B4. Visit case study sites	DSF/ WEW	10/1	9/26
B5. Analyze waste samples	WGC	12/1	
C1. Develop computer cost models	DSF	4/1	6/12
C2. Perform preliminary case study designs	FRT	12/1	
C3. Estimate case study disposal costs	FRT	1/1	
D. Evaluate treatment, recovery, and reuse	WEW	6/1	
B. Assess cost impacts	JAW	9/1	
F. Evaluate cost impact models	DSF/JAW	6/1	
G1a. Prepare background data report	PJS	8/1	
G1b. Prepare site visit report	DSF	12/1	
G1c. Prepare sampling/analysis report	WGC	2/1	
G2. Prepare draft report	DB	6/1	
G3. Prepare final report	DB	8/1	
H. Project management	DB	6/1	

on two index cards. One card is labeled “start” and the other “finish.” The cards are then divided into piles, one for each person responsible for performing the tasks.

After all cards are prepared, the project manager writes the name of each key individual on a card and pins the cards along the left column of the divided wall. The project manager then calls a meeting in the room and asks that all persons responsible for carrying out the tasks be present, including the client, any third-party agency representatives, and, if possible, the contractor. Each person is handed his or her cards and asked to tack each one onto the vertical division on the wall in the work week during which each task will be started and finished (see Fig. 1.5). Tasks may be divided into subtasks or better defined as required. When all cards are up, additional tasks not anticipated may be added. Others may be modified.

The key feature of the wall-scheduling procedure is the high degree of interaction that takes place during the scheduling meeting. Conflicts are identified early, discussed, and resolved. This process enables the key project team members to make firm commitments to the project management as to their assigned activities. Furthermore, these commitments are made in front of the entire group, thereby increasing the probability that they will indeed be accomplished on time.

The successful application of the wall-scheduling system relies on the project manager’s skills in planning the project, establishing a preliminary schedule for use by the group, and directing the proceedings during the scheduling meeting. If the project manager does these things well, total meeting time can be cut to a minimum while producing a realistic schedule to meet the client’s needs.

No single scheduling method is applicable for all projects. Table 1.13 summarizes some of the most important criteria for selecting the best scheduling method. These criteria should be reviewed in order to select the scheduling method that is best suited to the criteria with the highest priorities.

Budgeting. There are usually four ways that budgets can be computed for a given project. The first method is *zero-based* budgeting, in which the project manager starts with a list of tasks and estimates the worker-hours and corresponding costs to perform the work. The second is *downward* budgeting, which in-

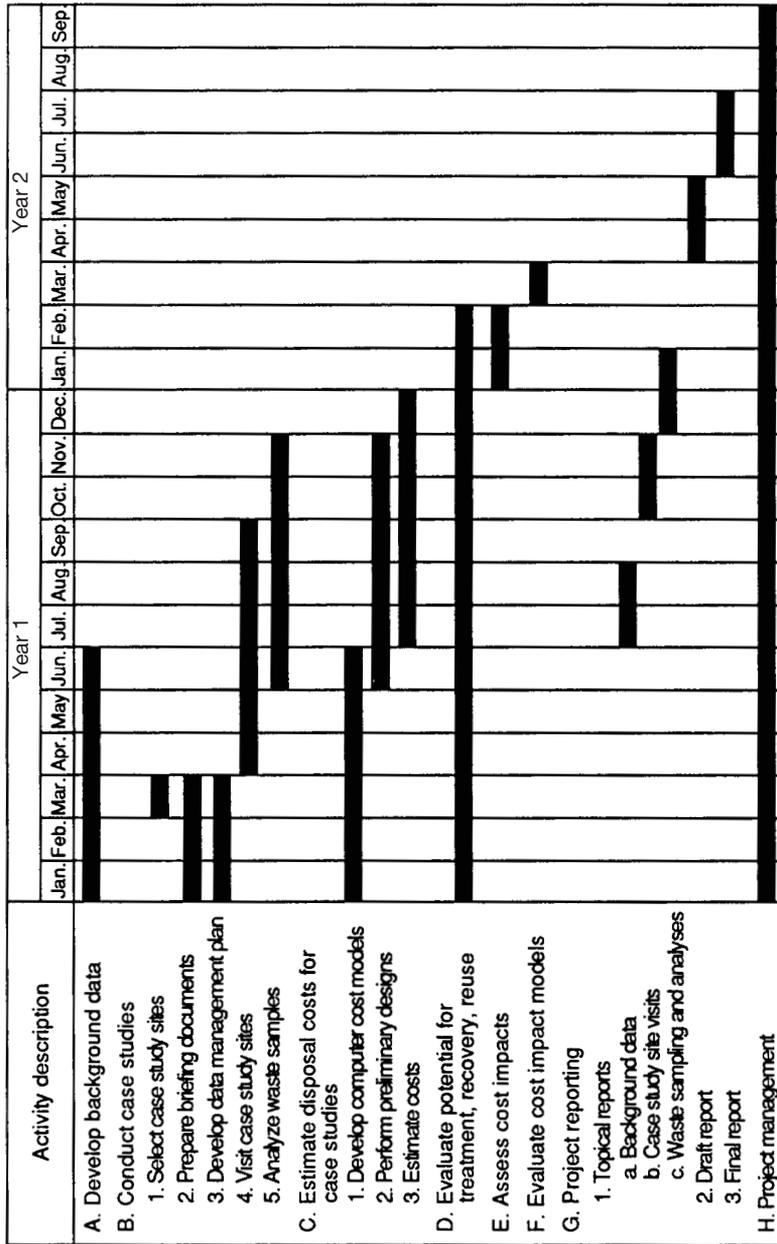


FIG. 1.2 Bar chart for an environmental study project (5).

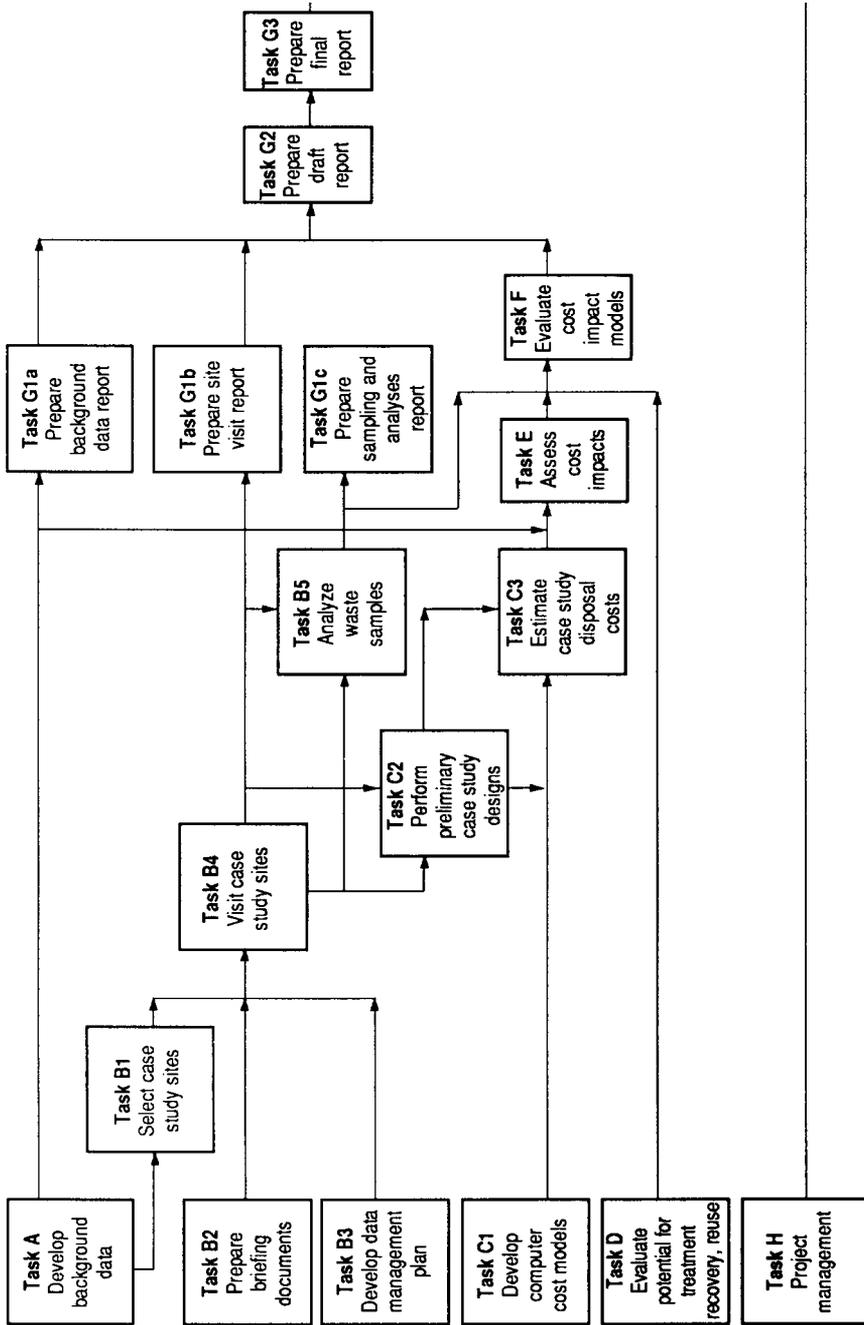


FIG. 1.3 Precedence diagram for an environmental study project (5).

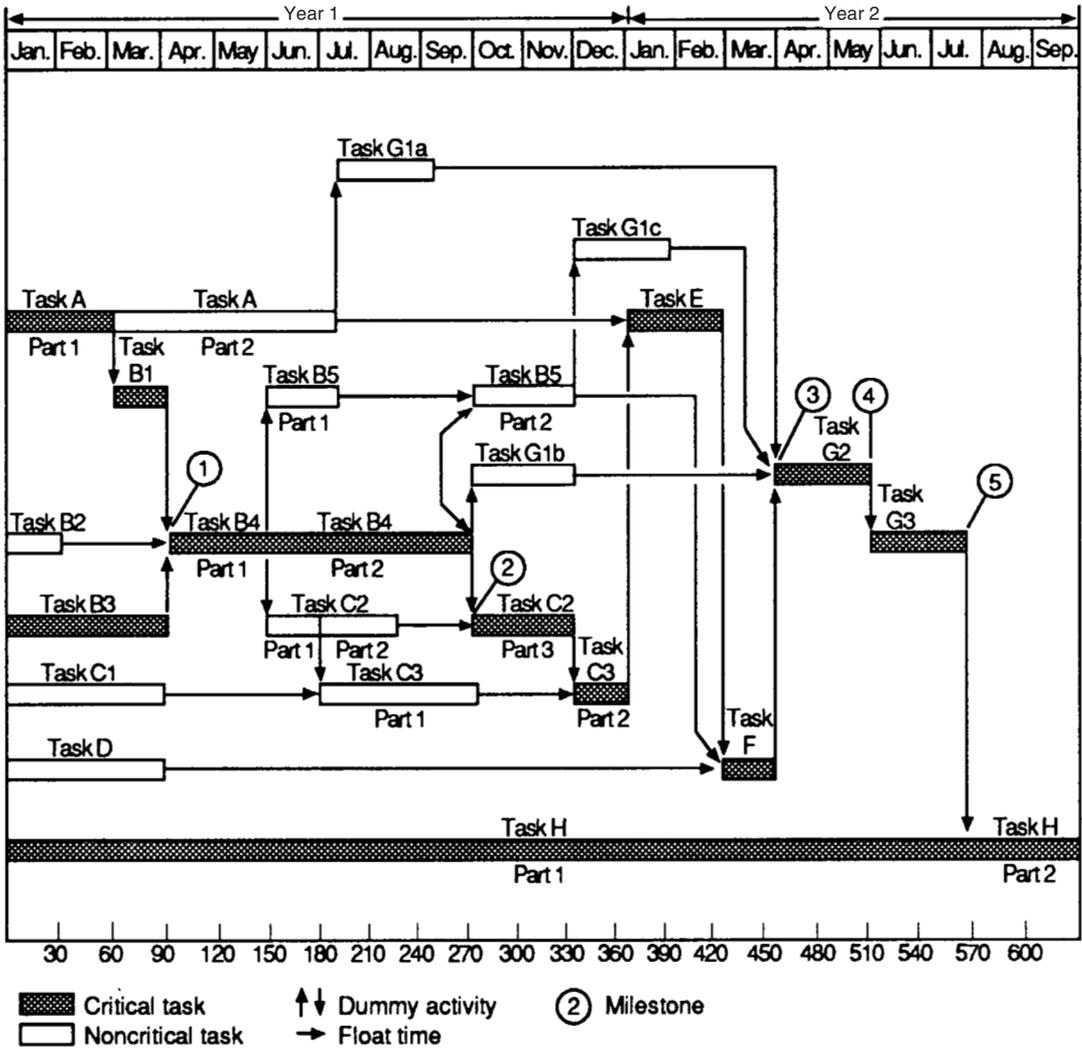


FIG. 1.4 Critical path diagram for an environmental study project (5).

involves starting with the total amount of compensation and breaking out the various cost components to establish the number of worker-hours that can be allocated. The third method of budgeting is the *unit cost* budget, or the use of “historical” cost data from a previous similar project, e.g., cost per sheet of drawings. The fourth method is the *staffing level* budget, which considers the number of people assigned to a job for a certain time period.

Each of these four budgeting methods has distinct advantages and disadvantages. Used together they provide a sound basis to derive the final project budget. Once the project budget has been determined, the result must be put into the zero-based budgeting format to permit proper monitoring and control of the project.

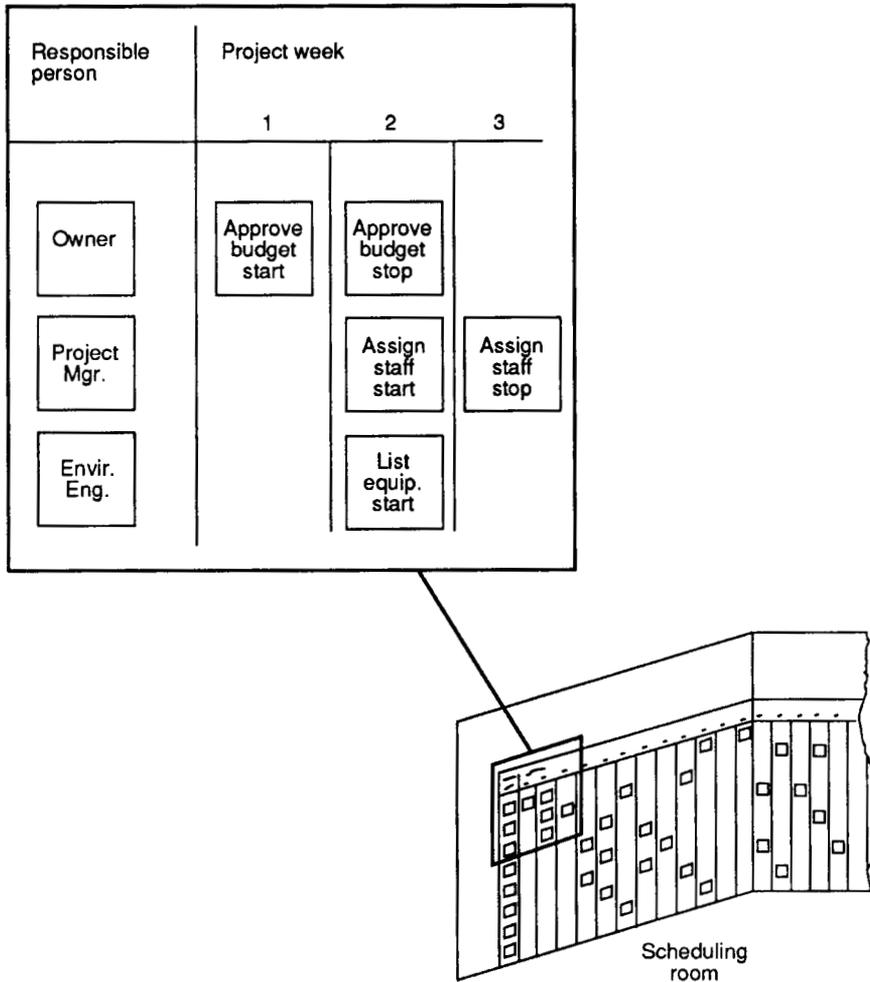


FIG. 1.5 Full wall scheduling room.

TABLE 1.13 Criteria for Selecting the Proper Scheduling Method (5)

Evaluation criteria	Milestone	Bar chart	CPM network diagram	Full wall schedule
1. Ease of communication	Good	Good	Poor	Excellent
2. Cost to prepare	Minimal	Minimal	Extensive	Moderate
3. Cost to update	Minimal	Minimal	Extensive	Moderate
4. Degree of control	Fair	Good	Excellent	Good
5. Applicability to large projects	Poor	Fair	Excellent	Good
6. Applicability to small projects	Excellent	Good	Poor	Good
7. Commitment from project team	Fair	Fair	Fair	Excellent
8. Client appeal	Fair	Good	Excellent	Excellent

An example of a completed budget for the solid waste study described in Table 1.11 is shown in Table 1.14. An example of a budget for a waste treatment facility design project is presented in Fig. 1.6.

Once the project budget has been completed, the next step is to develop a projected expenditure curve that will serve as the basis for projecting labor requirements and monitoring schedule and budget status throughout the project. The expenditure projection is derived by apportioning each task budget into the scheduled time frame for the corresponding activity. Values can then be totaled for each project period (monthly in the example) and summed up to estimate the cumulative expenditures throughout the project duration, as illustrated in Table 1.15. The results are then plotted to form a curve such as the one in Fig. 1.7.

Monitoring Schedules and Budgets

Both the client and the consulting engineer must be able to assess their project's budget and schedule status accurately throughout the project. One method to accomplish this is known as *integrated budget and schedule monitoring* (IBSM) (5,8). The IBSM method determines the overall schedule and budget status and consists of six phases:

1. Estimating the progress of each task
2. Computing total project progress
3. Estimating project expenditures
4. Determining overall schedule and budget status
5. Determining the schedule status of each task
6. Determining the budget status of each task

Each of the phases of the IBSM method is discussed below.

Estimating Individual Task Progress. The first step in the IBSM method is to prepare a realistic estimate of the progress on each task in terms of percent completion of that task. "Progress" is defined as work actually accomplished and is independent of either budget expenditures or calendar time spent on the activity. For example, if a task consists of pouring 4000 yd³ of concrete and a total of 3000 yd³ have been poured, then that task is 75% complete regardless of how much time or money it took to accomplish it.

Estimating the progress of each activity is the single most important step in the IBSM method. This step is nonmathematical, relying solely on the experience and judgment of the project manager and his or her staff.

One way to monitor progress on design jobs is to use a preliminary list of drawings to measure the progress of each design discipline. Figure 1.8 shows how this technique could be used to estimate the progress of the mechanical drawings on a waste treatment plant design project.

Computing Total Project Progress. After the percent of completion has been determined for each task, all percentages are multiplied by the corresponding task budgets, as shown in Table 1.16. The products of each multiplication are then totaled to obtain a value that represents the estimated amount of progress made to date on the total project. Dividing this sum by the total project budget provides an estimate of overall project progress. The progress computation is independent of the amount of money that has been spent on the project.

Estimating Project Expenditures. The next step is to estimate how much money has been spent on the project. These data must be obtained for the same time period that was used in developing the estimated progress.

TABLE 1.14 Budget for the Solid Waste Study Shown in Table 1.11

Phase/Item	DIRECT LABOR COSTS (Category of Personnel) A (Rate per Hour)										FEE COMPUTATION							PROJECT VALUE
	A Principal \$ 22.00/hr	A Supv. Mgr. \$ 16.00/hr	A Asst. Mgr. \$ 13.00/hr	A Tech./Eng. \$ 10.00/hr	A Scholarship \$ 8.00/hr	A Deadweight \$ 6.00/hr	A Secretary \$ 6.00/hr	B DIRECT COSTS	C OVERHEAD 18% B+C	D OTHER COSTS	E EST. COST B+C+D	F CONTR. FEE'S	G TOTAL B+C+D+F	H PROFIT	I			
A	20	30	200	40	0	40	30	300	6,300	1,314	13,140	1,314	14,454	1,445	15,899			
B1	8	16	2600	400	0	0	8	3200	1,500	2,920	292	3,212	321	3,533				
B2	176	356	570	0	0	48	1000	1,680	1,680	2,960	296	3,256	326	3,582				
B3	12	370	570	0	0	20	100	4,296	4,296	7,650	765	8,415	842	9,257				
B4	367	640	1560	0	0	160	340	9,900	9,900	19,700	1,970	21,670	2,167	23,837				
B5	0	320	520	400	0	0	40	7,620	4,800	17,500	1,750	19,250	1,925	21,175				
C1	176	320	7560	600	0	20	40	4,584	420	8,060	806	8,866	887	9,753				
C2	440	640	2080	0	0	120	20	6,360	260	10,860	1,086	11,946	1,195	13,141				
C3	176	192	1020	0	0	12	380	5,172	200	8,920	882	9,702	970	10,672				
D	440	640	570	0	0	120	140	2,532	200	4,420	442	4,862	486	5,348				
E	20	40	40	0	0	12	164	2,988	280	5,260	526	5,786	579	6,365				
F	440	640	1020	80	0	96	2388	3,432	520	6,240	624	6,864	686	7,550				
G1a	176	320	1020	300	0	80	130	4,644	1,200	8,940	894	9,834	983	10,817				
G1b	176	320	1020	300	0	80	130	4,644	1,200	8,940	894	9,834	983	10,817				
G1c	176	320	1020	300	0	80	130	4,644	1,200	8,940	894	9,834	983	10,817				
G2	440	1280	2080	800	0	140	700	10,500	600	18,100	1,810	19,910	1,991	21,901				
G3	176	640	570	200	0	160	280	3,084	2,800	7,940	794	8,734	873	9,607				
H	1320	2560	520	0	0	160	40	7,200	1,400	13,400	1,340	14,740	1,474	16,214				
TOTALS	55632	11328	25480	7400	0	652	5664	91080	21990	173790	17379	191169	19116	210285				

ARCHITECT-ENGINEER COST ESTIMATE		ARCHITECT-ENGINEER FIRM NAME AND ADDRESS			DESIGN STAGE		COST OR PRICING DATA REFERENCE
RFP/CONTR. NUMBER DACA87- 81 - R - 0000		ABC Consultants, Inc. 1000 Main Street Anytown, USA			MODIFIED CONCEPT	(OPTION) FINAL	
PREPARED BY: David Smith		PROJ. TITLE: Industrial Waste Treatment Facility			LOCATION Fort Kennedy		
APPROVED BY: A. W. Jones		DATE:	TASK NAME/BUILDING/ OTHER: Intermediate Design		PROJ. NO. 5062.61		
ITEM "A" - TASK EFFORT BREAKDOWN		TASK RECAP: 4,024 MH=\$ 133,674					
SPECIALTIES	JOB TITLES	MAN - HOURS	RATES \$	AMOUNTS \$	TOTALS \$		
PROJECT MANAGEMENT	Supervisor	214	20.50	4,387			
	Typist/Clerical	80	5.89	471	4,858		
CIVIL NO. OF DWGS.(3)	Supervisor	10	20.50	205			
	Journeyman	30	13.89	417			
	Draftsman	90	8.90	801			
	Typist	20	5.89	118	1,541		
ARCHITECTURAL NO. OF DWGS.(4)	Supervisor	14	20.50	287			
	Journeyman	30	13.89	417			
	Draftsman	48	10.95	526			
STRUCTURAL NO. OF DWGS.(7)	Typist	12	5.89	71	1,301		
	Supervisor	34	20.50	697			
	Journeyman	100	13.89	1,389			
MECHANICAL NO. OF DWGS.(36)	Draftsman	280	8.90	2,492			
	Typist	24	5.89	141	4,719		
	Supervisor	232	20.50	4,756			
	Journeyman	928	13.89	12,890			
HV/AC NO. OF DWGS.(2.)	Draftsman	996	10.95	10,906			
	Typist	32	5.89	188	28,740		
	Supervisor	6	20.50	123			
	Journeyman	20	13.89	278			
ENVIRONMENTAL NO. OF DWGS.(13)	Draftsman	30	8.90	267			
	Typist	4	5.89	24	692		
	Supervisor	25	20.50	513			
	Journeyman	98	13.89	1,361			
ELECTRICAL NO. OF DWGS.(11)	Draftsman	100	8.90	890			
	Typist	20	5.89	118	2,882		
	Supervisor	63	20.50	1,292			
	Journeyman	85	13.89	1,181			
	Draftsman	110	10.95	1,205			
SPECIFICATIONS	Typist	16	5.89	256	3,934		
	Supervisor	28	20.50	574			
	Journeyman	79	13.89	1,097			
ESTIMATES	Typist	44	5.89	259	1,930		
	Supervisor	20	20.50	410			
	Journeyman	102	13.89	1,417	1,827		
OTHER							
TOTAL DRAWINGS	71	TOTAL MAN-HOURS	4,024	TOTAL DIRECT SALES	52,424		
ITEM "B" OVERHEAD POOLS							
TITLES		RATES %	BASES \$				
1	Total Allowable Overhead	148.3	52,424		77,745		
2							
3							
ITEM "C" OTHER COST (CONSULTANTS, TELEPHONE, REPRODUCTION, POSTAGE, ATTACH ESTIMATES AS REQUIRED)					1,252		
ITEM "D" TRAVEL COST (ATTACH ESTIMATE SHEETS)					2,253		
ITEM "E" MATERIALS AND SUPPLIES (ATTACH ESTIMATES AS REQUIRED)							

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FIG 1.6 Budget for a waste treatment facility design project (5).

ENVIRONMENTAL ENGINEERING

TABLE 1.15 Calculation of Projected Expenditures (5)

Task Description	Task budget, \$/task	Unit cost, \$/month	Year 1												Year 2										
			Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.		
A. Develop background data	13,140	2,190	2,190	2,190	2,190	2,190	2,190	2,190	2,190	2,190	2,190	2,190	2,190	2,190											
B. Conduct case studies	2,920	2,920		2,920																					
1. Select case study sites	2,960	987	987	987																					
2. Prepare briefing documents	7,650	2,550	2,550	2,550																					
3. Develop data management plan	19,700	3,283		3,283	3,283	3,283	3,283	3,283	3,283	3,283	3,283	3,283	3,283	3,283											
4. Visit case study sites	17,500	2,917			2,917	2,917	2,917	2,917	2,917	2,917	2,917	2,917	2,917	2,917											
5. Analyze waste samples																									
C. Evaluate disposal costs for case studies	8,060	1,343	1,343	1,343	1,343	1,343	1,343	1,343	1,343	1,343	1,343	1,343	1,343	1,343											
1. Develop computer cost models	10,860	1,810	1,810	1,810	1,810	1,810	1,810	1,810	1,810	1,810	1,810	1,810	1,810	1,810											
2. Perform preliminary designs	8,820	1,470	1,470	1,470	1,470	1,470	1,470	1,470	1,470	1,470	1,470	1,470	1,470	1,470											
3. Estimate costs	4,420	316	316	316	316	316	316	316	316	316	316	316	316	316											
D. Evaluate potential for treatment, recovery, and reuse	5,260	2,630																							
E. Assess cost impacts	6,240	6,240																							
F. Evaluate cost impact models																									
G. Project reporting																									
1. Topical reports	8,940	4,470																							
a. Background data	8,940	4,470																							
b. Case study site visits	8,940	4,470																							
c. Waste sampling and analyses	8,940	4,470																							
2. Draft report	18,100	9,050																							
3. Final report	7,940	3,970																							
H. Project management	13,400	638	638	638	638	638	638	638	638	638	638	638	638	638											
Total Monthly Costs	173,790		8,024	8,024	10,944	8,024	16,048	26,992	34,762	42,532	55,029	69,933	84,837	104,334	118,512	125,406	133,460	143,922	153,610	163,298	173,152	182,840	192,528	202,216	211,904
Cumulative costs			8,024	16,048	26,992	34,762	42,532	55,029	69,933	84,837	104,334	118,512	125,406	133,460	143,922	153,610	163,298	173,152	182,840	192,528	202,216	211,904	221,592	231,280	240,968

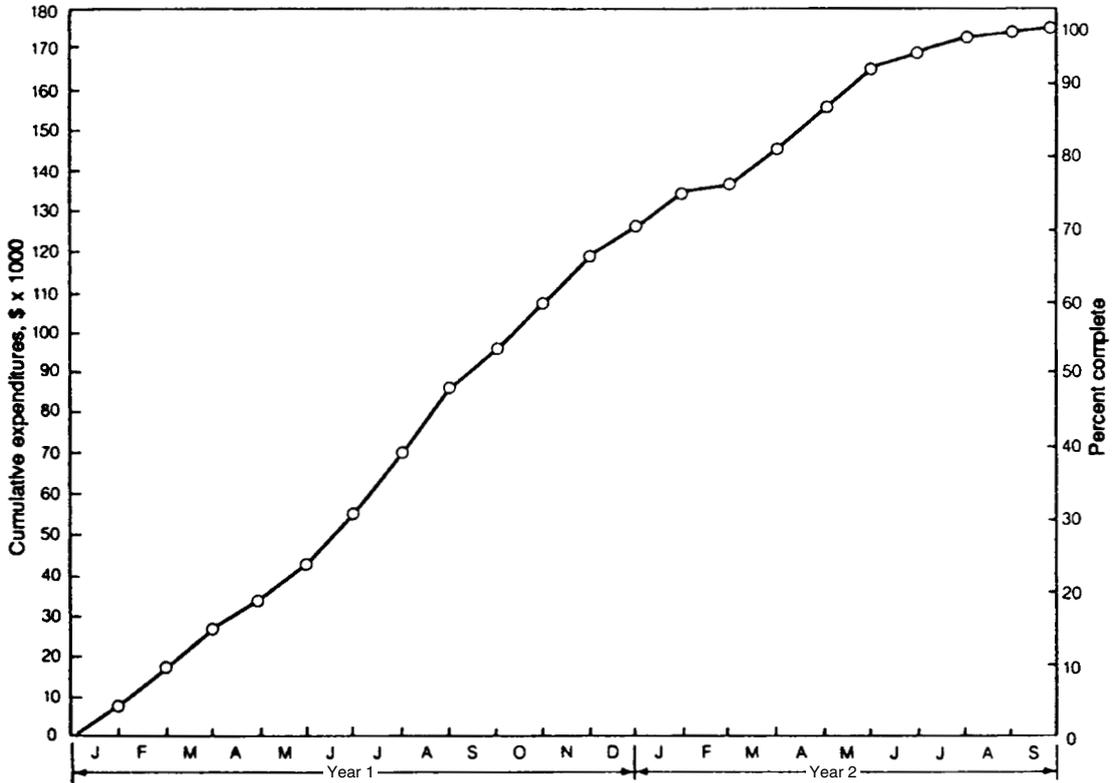


FIG. 1.7 Projected expenditure curve (S).

Schedule and Budget Status. Using the projected expenditure curve as a base, the actual project progress and project expenditures are plotted onto the same graph, as shown in Fig. 1.9. The overall schedule status can be determined graphically by measuring the horizontal distance between the projected expenditure curve and the progress curve, distance *A*. The overall budget status can be determined by measuring the vertical distance between the project progress curve and the actual expenditure curve, distance *B*.

A common mistake is to compare the projected expenditures (budgets) with the actual expenditures (costs) in an attempt to determine overall project status. If this comparison is made in the example project (Fig. 1.9), the actual expenditures appear to be less than the projected expenditures. This leads to the wrong conclusion—that the project is under budget and thus on target—when in reality this is not the case. Comparing projected expenditures with actual expenditures is not a realistic way to determine either budget or schedule status and may tend to lull both the project manager and the client into a false sense of security. A project may be so far behind schedule that it appears under budget.

Task Schedule Status. If the results of this analysis indicate that the project is behind schedule, it is necessary to know which areas are in trouble. By applying the estimated progress percentages to the project schedule, it will become readily apparent which tasks are behind schedule—tasks A, B4, B5, C1, C2, and H in Fig. 1.10.

Task Budget Status. Monitoring the budget status of each individual task is not as simple as monitoring the schedule status. Independent cost accounting must be maintained for each task to be monitored. This re-

TABLE 1.16 Calculations of Estimated Progress for a Solid Waste Study (5)

Task description	Task budget, \$/task	Data base date Estimated progress	
		% Complete	Equivalent progress
A. Develop background data	13,140	× 65	– 8,541
B. Conduct case studies			
1. Select case study sites	2,920	× 100	– 2,920
2. Prepare briefing documents	2,960	× 100	– 2,920
3. Develop data management plan	7,650	× 100	– 7,650
4. Visit case study sites	19,700	× 20	– 3,940
5. Analyze waste samples	17,500	× 0	– 0
C. Estimate disposal costs for case studies			
1. Develop computer cost models	8,060	× 10	– 806
2. Perform preliminary designs	10,860	× 10	– 0
3. Estimate costs	8,820	× 10	– 0
D. Evaluate potential for treatment, recovery, and reuse	4,420	× 0	– 1,326
E. Assess cost impacts	5,260	× 30	– 0
F. Evaluate cost impact models	6,240	× 0	– 0
G. Project reporting			
1. Topical reports			
a. background data	8,940	× 0	– 894
b. case study site visits	8,940	× 10	– 0
c. waste sampling and analyses	8,940	× 0	– 0
2. Draft report	18,100	× 0	– 0
3. Final report	7,940	× 0	– 0
H. Project management	13,400	× 25	– 3,350
Totals	\$173,790		32,387
Total project progress = \$32,387 ÷ \$173,790 = 0.186 = 18.6%			

quires increased paperwork and requires close monitoring of the project team to ensure that everyone is charging their time and expenses to the proper task code. The project manager and client must determine on a case-by-case basis whether the additional information is worth the cost required to maintain it.

Analyzing Trends. While it is essential to know the budget and schedule status of a project at a given point in time, monitoring trends can be even more revealing. A tabulation of project status trends is presented in Table 1.17. The “progress” column indicates a rather steady increase in the progress of the project—nothing particularly unusual. The “schedule status” column shows that the project got behind schedule almost from the outset and has deteriorated steadily since. This steady deterioration in the schedule status indicates a fundamental problem. Possible explanations are

- Insufficient work force has been allocated to the project
- Project staff have not been working productively
- Too much time is being spent on details
- Original schedule was unrealistic
- Too many design alternatives are being examined

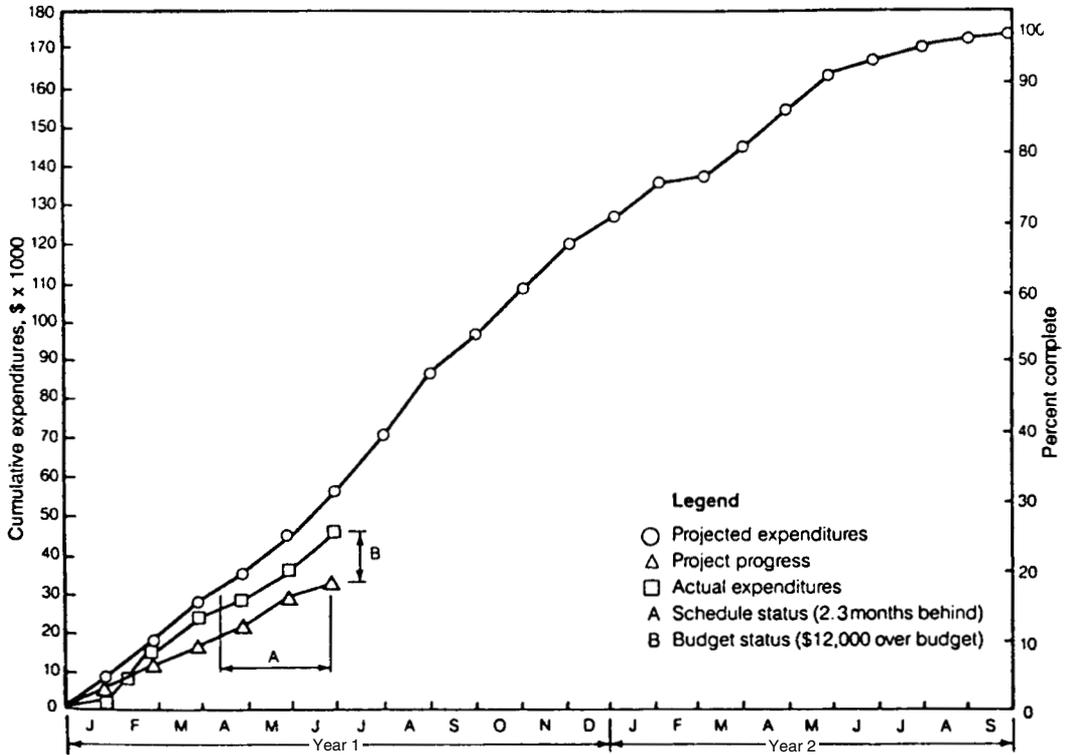


FIG. 1.9 Computing overall schedule and budget status using the IBSM method (5).

The “budget status” column tells a somewhat different story. The project started out well, got into a bit of trouble in February, and then generally stabilized until June, at which time the budget suddenly developed major problems. This trend indicates that something dramatic happened in June. Possible explanations are

- Costs from another project were incorrectly charged to the job
- A major design error was discovered and substantial redesign was required
- The scope of work was increased

Careful study of schedule and budget trends can be most informative. However, such data are available only if the progress of each project is monitored on a regular basis. Frequent monitoring also allows early problem identification while there is still time to resolve problems without jeopardizing the final completion date or going over budget.

QUALITY CONTROL

Quality environmental engineering performance encompasses business, contracting, and professional practices. In the broad sense, quality may be characterized as meeting not only the requirements and expecta-

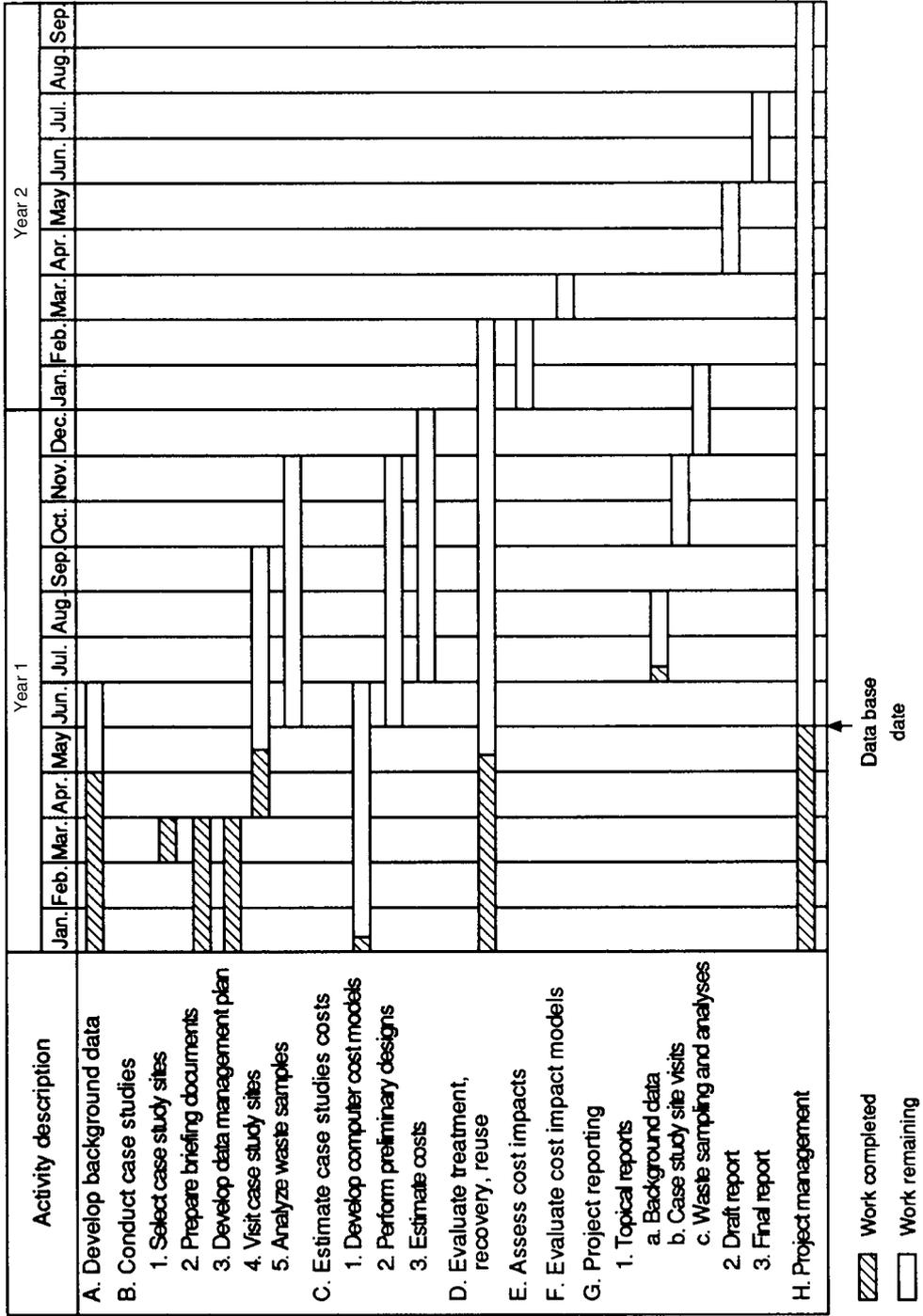


FIG. 1.10 Determining the schedule status of each task (5).

TABLE 1.17 Project Status Trends (5)

Project period	Progress, %	Schedule status	Budget status, \$
Year 1			
January	3.2	0.3 months behind	2,000 under
February	6.7	0.8 months behind	3,000 over
March	9.4	1.1 months behind	8,000 over
April	11.6	1.7 months behind	8,000 over
May	15.8	2.0 months behind	8,500 over
June	18.6	2.3 months behind	12,000 over
July			
August			
September			
October			
November			
December			
Year 2			
January			
February			
March			
April			
May			
June			
July			
August			
September			

tions of the owner but also those of the design engineer and the construction contractor. In addition, applicable insurance, legal, and regulatory requirements must be fulfilled. (26, 27)

The effective application of a quality assurance/quality control program or formal program registration, such as that provided by the International Organization for Standardization (ISO), requires the full commitment of top management and participation by staff throughout the organization. This philosophy also is known as total quality management, in which an organization is driven by client satisfaction with a commitment to quality at the source.

Quality performance is part of an organization's mission and is an endless part of day-to-day operations. Management must recognize that staff is the key to consistent quality and obtain client critiques to assess and improve program effectiveness. Also, a protocol for record keeping and application of modern standards is essential. Frequent program updates and continued employee training are important to keep abreast of industry developments and regulatory changes.

The requirements of the quality control program should be a published procedure. The quality control procedure should begin with a statement of organization policy and proceed into functional areas, such as subcontracting, design procedures, document controls, and training. One approach would be to address each topic in terms of scope, responsibility and authority, quality activity, and reference documents. The reference documents should include the organization's standard operating procedures, project management responsibilities, and design guidelines.

On all projects, a strong quality control program significantly improves client satisfaction with the final facilities. In professional practice, implementation of a quality control program for a design team begins with assignment of senior technical advisors. The technical advisors are chosen project-specifically to ensure that:

- The completed project will effectually meet the client's needs as an operational and cost-effective facility.
- The client's best interest is served in strict accordance with highest technical standards of practice and with the terms of the contract and sound technical practices.

Quality assurance encompasses the identification of standards of practice, which may include public, regulatory, and/or in-house guidelines, and involvement of senior level technical experts, who serve as independent advisors and reviewers. During conceptual and preliminary design projects, the technical advisors are selected to bring design development and big picture experience to the project. During preliminary and final design, technical reviewers are selected principally to observe and scrutinize detailed design activities.

At project initiation, the technical advisor's role is to facilitate technical exchange in the development of design and operation concepts and to assist in the assessment of applicable technology, including participation in selected project work sessions. As the project progresses, technical advisors review design development documentation, ensure that approved design concepts are being applied, and evaluate risk management considerations.

Beginning in preliminary design and continuing through final design, the quality control program should involve senior design discipline engineers to review the adequacy of field data and design computations. Reviews should encompass the technical adequacy from the discipline perspective, check compliance with contract requirements, and identify potential coordination issues.

Quality control comments should be recorded on comment/response forms prescribed by the organization's quality control procedures. Normally, responses should be reconciled at the project team level. Nevertheless, the quality control program should provide a disputes resolution procedure for use if needed.

Major milestone reviews are necessary throughout the project. Early reviews examine major design criteria, preliminary engineering documents, and order-of-magnitude cost estimates that form the basis of the project. Later, detailed and final design criteria, materials of construction and equipment, layouts and other design details, and outline specifications are examined. Near the completion of the design phase, quality control focuses on the bidability and constructability of the approved systems and the overall functions of the project features.

INTERNATIONAL ORGANIZATION FOR STANDARDIZATION

The International Organization for Standardization (ISO) was established in 1947 to set uniform standards for quality control in Europe. It is attracting increased interest and support as the source of true international standards in manufacturing, product specification, and communications. The ISO produces standards through a structure of technical committees with subcommittees, which are further divided into working groups. ISO 9000 and 14000 standards for quality and environmental management, respectively, are of interest to the environmental engineer. (28, 29)

As the sole private sector certifier for United States national standards, the American National Standards Institute (ANSI) is the United States's representative to the ISO. The American Society of Testing and Materials (ASTM), American Society for Quality Control (ASQC), and NSF International are members of ANSI and are participating in ISO standards development and distribution.

ISO 9000

ISO 9000 is a series of standards of general business conduct to achieve quality. It applies to quality management systems and serves as an extension of ongoing total quality management practices. ISO 9001, 9002, and 9003 are contractual standards for design and servicing, production and installation, and final in-

spection and testing, respectively. These standards are tiered in that ISO 9001 includes 9002, which includes 9003.

ISO 14000

More specific to environmental engineering, ISO 14000 is an evolving series of standards for various components of an organization's environmental management plan. Environmental issues and functions are integrated into the organization's business strategy and become a proactive part of doing business. ISO 14000 focuses on management systems rather than performance levels and may be pursued as an add-on to an ISO 9000 registration, which has many similar components, or as a stand-alone registration.

The ISO Technical Committee—(TC) 207: Environmental Management—is developing the ISO 14000 series of standards. The TC 207 subcommittee and working group structure is illustrated in Figure 1.11. The United States has a Technical Advisory Group that works with these groups.

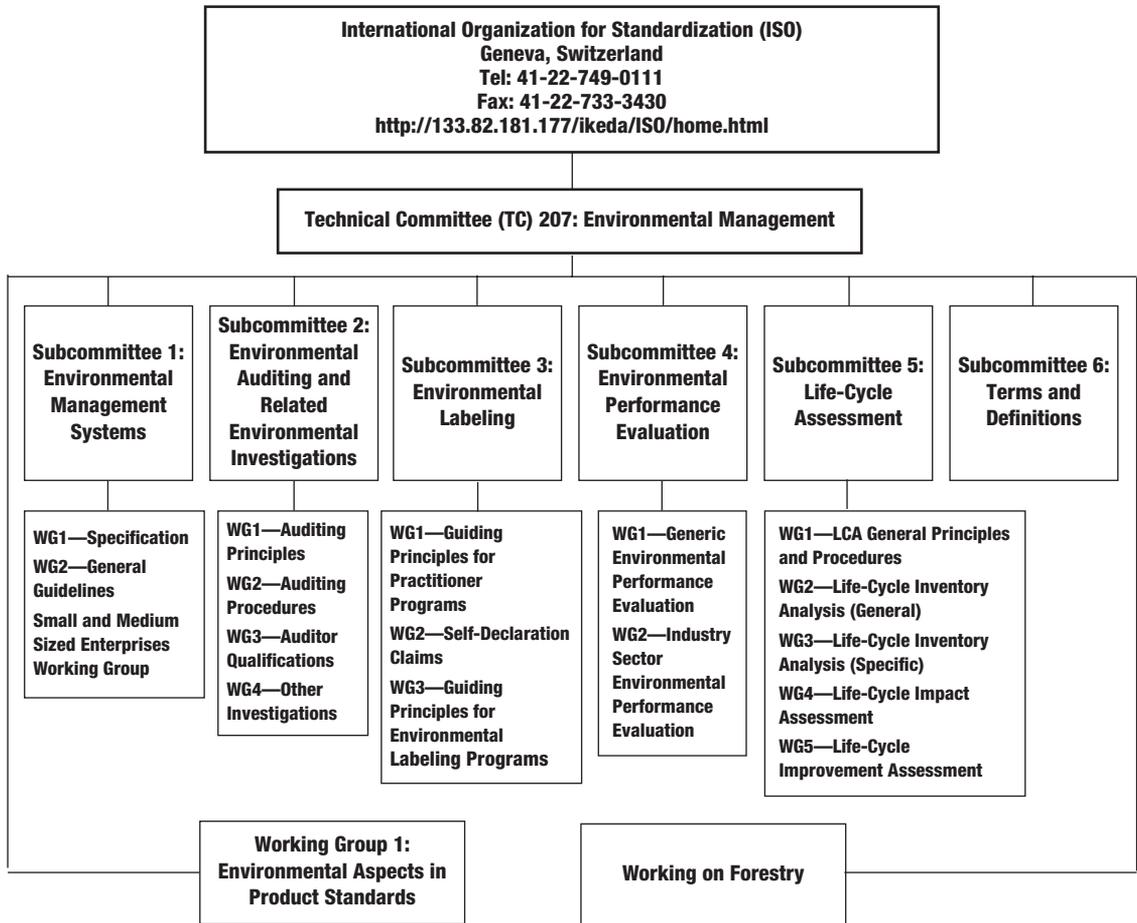


FIG. 1.11 ISO Technical Committee 207 Organization.

The ISO 14001 standard is a specification standard and contains those requirements that may be objectively audited for registration and/or self-declaration purposes. The standard identifies an environmental management system (EMS) designed to address all facets of an organization's operations, products, and services, including environmental policy, resources, training, emergency responses, audits, and management reviews. The model EMS is based on five major elements:

1. Corporate Policy and Commitment
2. Planning
3. Implementation and Operations
4. Measurement and Evaluation
5. Management Review and Improvement.

The standards in the ISO 14000 series are presented in Table 1.18. Other than ISO 14000, the standards provide guidance for development and implementation of management systems and product standards. The ISO 14000 series of environmental management standards may be obtained from:

- The American National Standards Institute (ANSI)
7315 Wisconsin Avenue, Suite 250-E
Bethesda, MD 20814

TABLE 1.18 Standards in the ISO 14000 Series (27)

Organizational Evaluation	
ISO 14001	Environmental Management Systems—Specifications with Guidance for Use
ISO 14004	Environmental Management Systems—General Guidelines on Principles, Systems, and Supporting Techniques
ISO 14010	Guidelines for Environmental Auditing—General Principles on Environmental Auditing
ISO 14011/1	Guidelines for Environmental Auditing—Audit Procedures—Audit of Environmental Management Systems
ISO 14012	Guidelines for Environmental Auditing—Qualification Criteria for Environmental Auditors
ISO 14015	Environmental Site Assessments
ISO 14031	Evaluation of Environmental Performance
Product Evaluation	
ISO 14040	Environmental Management—Life Cycle Analysis—Principles and Framework
ISO 14041	Environmental Management—Life Cycle Analysis—Life Cycle Inventory Analysis
ISO 14042	Environmental Management—Life Cycle Analysis—Impact Assessment
ISO 14043	Environmental Management—Life Cycle Analysis—Interpretation
ISO 14020	Goals and Principles of All Environmental Labeling
ISO 14021	Environmental Labels and Declarations—Self Declaration
	Environmental Claims—Terms and Definitions
ISO 14022	Environmental Labels and Declarations—Self Declaration
	Environmental Claims—Symbols
ISO 14023	Environmental Labels and Declarations—Self Declaration
	Environmental Claims—Testing and Verification
ISO 14024	Environmental Labels and Declarations—Environmental Labeling Type I—Guiding Principles and Procedures
ISO 14025	Environmental Labels and Declarations—Environmental Information Profiles—Type III Guiding Principles and Procedures
ISO Guide	Guide for Inclusion of Environmental Aspects in Product Standards

- American Society for Testing and Materials
100 Bar Harbor Drive
West Conshohocken, PA 19428
- American Society for Quality Control (ASQC)
611 East Wisconsin Avenue P.O. Box 3005
Milwaukee, WI 53201
- NSG International
2100 Commonwealth Boulevard
Ann Arbor, MI 48105

PROJECT ECONOMICS

The proper application of economic principles to environmental problems is essential in order to identify and implement the most cost-effective solutions. In most investment decisions, the investment costs can be compared to the anticipated return in order to establish the viability of a proposed project. Most environmental projects, on the other hand, have a return on investment that is difficult to quantify.

This section describes a variety of techniques that are commonly used to ensure that the most cost-effective solutions are identified and to quantify the costs of implementing these solutions.

Capital Costs

Capital cost estimates may be developed to compare project alternatives, to identify major cost items, or to establish a project budget. The scope of the estimate depends upon the specific project purpose and the degree of project definition.

In general, from the initial feasibility study, through appropriation, and into design, the cost estimate becomes more detailed and more accurate. The most common types of estimates are described in Fig. 1.12.

Order-of-magnitude estimates are used in feasibility studies and provide guidance on basic decision making. Comparative estimates combine an order-of-magnitude estimate with the specific factors of a particular project and are developed for comparing alternative solutions to a particular problem. Predesign estimates require additional process design factors, such as equipment lists and site plans. They generally are used for appropriation of funds and for a preliminary level of budget control.

Preliminary design, intermediate design, and final design cost estimates are based upon increasingly detailed levels of engineering design and are used to revise fund appropriations and for budget control. Contractor estimates, like final design estimates, are based on 100% complete plans and specifications. However, the quantity takeoffs and unit prices are estimated in much greater detail.

Capital cost estimates are typically divided into direct and indirect costs. Direct costs are construction items for the project and include property, equipment, and materials. Indirect costs are project services, such as legal and administrative fees, engineering fees, contractor overhead and profit, and interest during construction.

Direct Costs. The method for determining direct costs depends upon the cost-estimating level. Direct costs for order-of-magnitude estimates are normally obtained from published cost curves and in-house project cost files. Comparative estimates are determined by estimating the cost of only those components of the project which are different from the alternatives being considered.

Typical methods for determining the direct costs for predesign, preliminary design, intermediate design, and definitive cost estimates are presented in Table 1.19. A partial listing of the general categories of estimating aids and some typical sources of cost information are presented in Table 1.20.

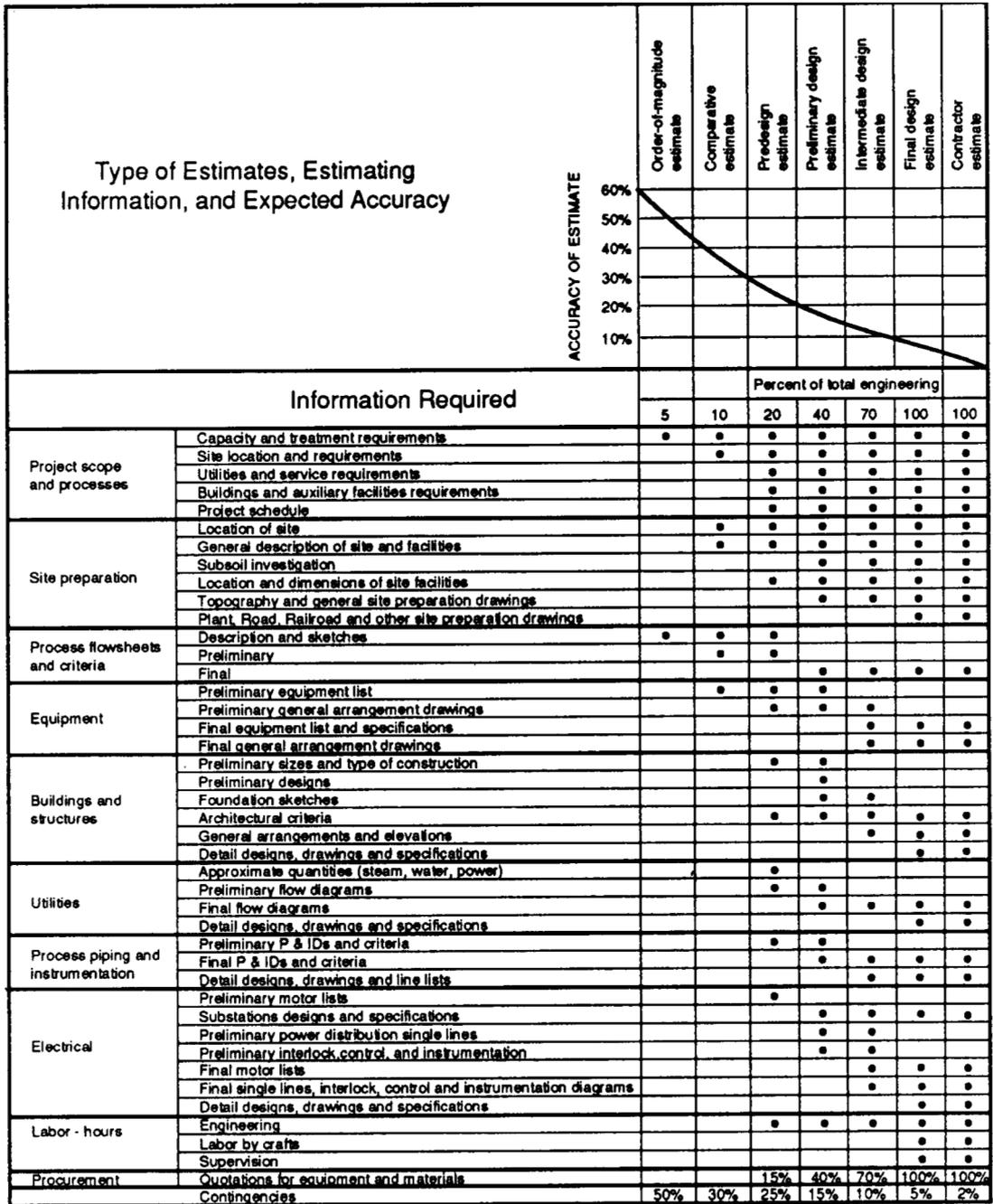


FIG. 1.12 Types of estimates, estimating information and expected accuracy.

TABLE 1.19 Methods of Determining Quantities and Unit Prices

Cost component	Pre-design estimate		Preliminary design estimate		Intermediate design estimate		Final design estimate	
	Quantities	Unit prices	Quantities	Unit prices	Quantities	Unit prices	Quantities	Unit prices
Major equipment	Detailed takeoff	Vendor quotes	Detailed takeoff	Vendor quotes	Detailed takeoff	Vendor quotes	Detailed takeoff	Vendor quotes
Minor equipment	% of major equipment costs		% of major equipment costs		Detailed takeoff	Estimating guides	Detailed takeoff	Vendor quotes
Major piping	Rough takeoff	Estimating guides	Rough takeoff	Estimating guides	Detailed takeoff	Estimating guides	Detailed takeoff	Local contractors
Minor piping	% of major equipment costs		% of major equipment costs		Rough takeoff	Estimating guides	Rough takeoff	Estimating guides
Concrete	Rough takeoff	Estimating guides	Rough takeoff	Estimating guides	Rough takeoff	Estimating guides	Detailed takeoff	Local contractors
Buildings	Rough takeoff	Estimating guides	Rough takeoff	Estimating guides	Rough takeoff	Estimating guides	Detailed takeoff	Local contractors
Structural steel	% of major equipment costs		% of major equipment costs		Rough takeoff	Estimating guides	Detailed takeoff	Local contractors
Miscellaneous metals	% of major equipment costs		% of major equipment costs		% of major equipment costs		Detailed takeoff	Estimating guides
Electrical	% of major equipment costs		% of major equipment costs		Rough takeoff	Estimating guides	Detailed takeoff	Estimating guides
Instrumentation/controls	% of major equipment costs		Rough takeoff	Estimating guides	Detailed takeoff	Vendor quotes	Detailed takeoff	Vendor quotes
Painting and coatings	% of major equipment costs		% of major equipment costs		Rough takeoff	Estimating guides	Detailed takeoff	Vendor quotes

TABLE 1.20 Selected Sources of Capital Cost Information

-
1. Cost Curves
 - a. *Innovative and Alternative Assessment Manual*, US EPA, 430/9-78-009, February, 1980
 - b. *Handbook of Biological Wastewater Treatment—Evaluation, Performance and Cost*, H. H. Benjes, Jr., 1980
 - c. *Capital and Operating Costs of Selected Air Pollution Control Systems*, US EPA, 450/3-76-014, May, 1976
 - d. *Handbook of Sludge Handling Processes—Cost and Performance*, G. L. Culp, Garland STPM Press, New York, 1979
 - e. *State of the Art of Small Water Treatment Systems*, Office of Water Supply, US EPA, August, 1977
 2. Computer Cost Models
 - a. *Computer Cost Models for Potable Water Treatment Plants*, US EPA, 1979, PB-287 744/7BE
 - b. CAPDET Clearinghouse, Mississippi State University (Cost Estimating for Municipal Wastewater Treatment Systems)
 3. Estimating Guides
 - a. *Dodge Guide for Estimating Public Works Construction Costs*
 - b. *Dodge Construction Systems Costs*
 - c. *Dodge Manual for Building Construction Pricing and Scheduling*
 - d. *Construction Estimating Standards*, Richardson Engineering Services, Inc. (4 volumes)
 - e. *Means Building Construction Data*
 4. Handbooks
 - a. *The Building Estimator's Reference Book*, Frank R. Walker Company
 - b. *Data Book for Civil Engineers*, Elwyn E. Seelye
 5. Construction Reporting Services
 - a. *Contractor's Data Report*
 - b. *Construction Digest Reports*
 6. Magazines and Periodicals
 - a. *Engineering News-Record*
 - b. Miscellaneous trade journals
 7. Contractors and active estimators in the industry
 8. Equipment and materials suppliers
 9. Past estimates that have been developed and other data accumulated in the company and estimator's personal files
-

Indirect Costs. Indirect costs, e.g., legal and administrative costs, engineering fees, contractor overhead and profit, interest during construction, and contingencies, must be added to the direct construction costs to obtain an estimate of total capital costs. Table 1.21 presents a summary of the range of these indirect costs and the factors that determine their magnitude.

Cost Factors. The factors that affect the direct and indirect costs in any capital cost estimate are numerous and vary significantly from project to project. In addition to the obvious charges for labor, materials, and equipment required to construct any particular project, the less obvious factors that must be taken into account generally fall into four broad categories. These are design, construction, location, and administrative variables. Typical considerations within each category are listed on Table 1.22. The development of a specific list for each cost estimate is suggested.

Cost Indexes. It is often desirable to compare a cost estimate to an earlier estimate or to project the estimate to a future date. However, because of the historic charges in unit prices, these adjustments cannot be accurately made without keying the cost estimate to a specific date and having some method of measuring the historic charge in costs. The four most common cost indexes used for this purpose are:

TABLE 1.21 Indirect Cost Factors

Indirect costs	Percent of total direct costs	Factors affecting indirect costs
1. Legal and administrative costs	2–5	Permitting difficulties Land acquisition requirements Requirement for outside legal services
2. Engineering fees	5–20	Project complexity Project size Engineering services to be provided
3. Contractor overhead and profit	10–30	Geographic location Degree of competition Project size
4. Interest during construction	*	Project size Project schedule Prevailing interest rates
5. Contingencies	2–50	Uncertainties regarding project definition Level of cost estimate

*Cannot be accurately estimated as a percentage of direct costs.

TABLE 1.22 Project Variables that Affect Cost Estimates

1. Design variables
 - Required quantities of labor, material, and permanent equipment
 - Complexity of design
 - Standard versus nonstandard materials or equipment
 - Conformance to local practice
 - Maintenance of existing facilities or operations
 - Condition of plans and specifications
2. Construction variables
 - Site soils
 - Dewatering requirements
 - Project equipment requirements
 - Access to the work
 - Scheduling or phasing of the work
 - Storage and space requirements
 - Temporary facilities
 - Waste or borrow areas
 - Job indirect charges
3. Location variables
 - Geographic location
 - Wage scales and stability
 - Utility availability
 - Availability of required skilled trades
 - Local taxes, permits, and code requirements
4. Administrative variables
 - Market conditions
 - Political considerations
 - Size of project
 - Terms of payment
 - Inflation

1. U.S. Environmental Protection Agency (EPA) indexes, published in the Municipal Facilities Division.
2. Engineering News-Record (ENR) indexes, published in *Engineering NewsRecord*.
3. Marshall and Stevens (M&S) equipment index, published in *Chemical Engineering*.
4. Chemical Engineering (CE) index, published in *Chemical Engineering*.

These cost indexes are updated by monitoring specific construction material and labor costs, proportioning these costs by a predetermined factor, and thereby deriving an index. Another function of cost indexes is to adjust unit cost data in order to account for differences between different cities.

Operation and Maintenance Costs

Operation and maintenance (O&M) costs comprise the following elements:

1. Labor
2. Energy
3. Renewal and replacement
4. Chemicals
5. Waste disposal
6. Miscellaneous costs (taxes, insurance, etc.)

The bases for estimating each of these elements are presented in Table 1.23 along with a partial list of commonly used sources for collecting the necessary data to prepare the O&M cost estimate.

The factors affecting O&M cost estimates are similar to those that affect capital cost estimates. O&M cost estimates are also impacted by long-term variables such as changes in interest rates, inflation, and com-

TABLE 1.23 Operation and Maintenance Cost Factors

Cost element	Basis of estimating	Data sources
Labor	<ul style="list-style-type: none"> • Published cost curves for various unit processes • Estimates of overall staffing needs • Prevailing local labor rates 	<ul style="list-style-type: none"> • EPA <i>Innovative and Alternative Technology Manual</i> • <i>Benjes Handbook of Biological Treatment</i> • Magazines and periodicals • EPA <i>State of the Art of Small Water Treatment Systems</i>
Energy	<ul style="list-style-type: none"> • Published cost curves for various unit processes • Takeoff of horsepower totals from preliminary motor lists • Calculations of heating/cooling requirements 	<ul style="list-style-type: none"> • EPA <i>Manual on Energy Conservation in Municipal Wastewater Treatment</i> • EPA <i>State of the Art of Small Water Treatment Systems</i> • Magazines and periodicals
Renewal and replacement	<ul style="list-style-type: none"> • Percentage of equipment cost • Estimated dosages times design flow rates 	<ul style="list-style-type: none"> • Equipment cost estimate • <i>Chemical Marketing</i> • Magazines • Chemical vendors
Waste disposal	<ul style="list-style-type: none"> • Estimated cost per unit of waste disposed 	<ul style="list-style-type: none"> • Contract waste disposers
Other miscellaneous costs	<ul style="list-style-type: none"> • Site specific 	

petition for labor, energy, and chemicals. Because of these unknowns, an expected level of accuracy is typically not assigned to annual cost estimates.

Project Cost Analyses

Initially, the financial issues, such as rate of return, and analyses objectives, such as minimum life cycle costs, are defined for the project cost analysis. Economic analysis factors for simple and compound interest are applied during the analysis. The factors presented later in this section are the capital recovery factor for annual cost analyses and the uniform series present-worth factor for present-worth cost analyses. Other common economic factors follow, where “ i ” represents interest rate and “ n ” is the number of periods (years).

The future value of the simple interest payments factor is:

$$\text{Simple Payment Amount} = (1 + ni)$$

For compound interest, the following economic factors apply:

$$\text{Single Payment Compound Amount} = (1 + i)^n$$

$$\text{Single Payment Present Worth} = \frac{1}{(1 + i)^n}$$

$$\text{Uniform Series Compound Amount} = \frac{(1 + i)^n - 1}{i}$$

$$\text{Sinking Fund payment} = \frac{i}{(1 + i)^n - 1}$$

When evaluating alternatives, it is often necessary to combine capital and O&M costs into a single estimate in order to determine which alternative is most cost-effective. The most commonly used methods for such a consolidation are annualized cost analysis and present-worth analysis. Although each method presents the results differently, the selection of the most cost-effective alternative will always be the same, regardless of which method is used.

Annual Cost Analysis. In order to annualize capital costs, an interest rate and a project life must be estimated based on guidelines used by the client and/or funding agencies. A *capital recovery factor* (CRF) is used to convert capital costs to annual costs. The capital recovery factor can be directly calculated as follows:

$$\text{CRF} = \frac{(i)(1 + i)^n}{(1 + i)^n - 1}$$

where CRF = capital recovery factor
 i = interest rate per period
 n = number of periods

A tabulation of commonly used capital recovery factors is presented in Table 1.24. To calculate the annualized capital cost, the capital cost is multiplied by the capital recovery factor.

In addition to annualized capital costs and annual O&M costs, the project may include equipment that

TABLE 1.24 Capital Recovery Factors

Number of periods n	Interest rate per period i									
	2	4	6	8	10	12	15	20	25	30
1	1.020 00	1.040 00	1.060 00	1.080 00	1.100 00	1.120 00	1.150 00	1.200 00	1.250 00	1.300 00
2	0.515 05	0.530 20	0.545 44	0.560 77	0.576 19	0.591 70	0.615 12	0.654 55	0.694 44	0.734 78
3	0.346 75	0.360 35	0.374 11	0.388 03	0.402 11	0.416 35	0.437 98	0.474 73	0.512 30	0.550 63
4	0.262 62	0.275 49	0.288 59	0.301 92	0.315 47	0.329 23	0.350 27	0.386 29	0.423 44	0.461 63
5	0.212 16	0.224 63	0.237 40	0.250 46	0.263 80	0.277 41	0.298 32	0.334 38	0.371 85	0.410 58
6	0.178 53	0.190 76	0.203 36	0.216 32	0.229 61	0.243 23	0.264 24	0.300 71	0.338 82	0.378 39
7	0.154 51	0.166 61	0.179 14	0.192 07	0.205 41	0.219 12	0.240 36	0.277 42	0.316 34	0.356 87
8	0.136 51	0.148 53	0.161 04	0.174 01	0.187 44	0.201 30	0.222 85	0.260 61	0.300 40	0.341 92
9	0.122 52	0.134 49	0.147 02	0.160 08	0.173 64	0.187 68	0.209 57	0.248 08	0.288 76	0.331 24
10	0.111 33	0.123 29	0.135 87	0.149 03	0.162 75	0.176 98	0.199 25	0.238 52	0.280 07	0.323 46
11	0.102 18	0.114 15	0.126 79	0.140 08	0.153 96	0.168 42	0.191 07	0.231 10	0.273 49	0.317 73
12	0.094 56	0.106 55	0.119 28	0.132 70	0.146 76	0.161 44	0.184 48	0.225 26	0.268 45	0.313 45
13	0.088 12	0.100 14	0.112 96	0.126 52	0.140 78	0.155 68	0.179 11	0.220 62	0.264 54	0.310 24
14	0.082 60	0.094 67	0.107 58	0.121 30	0.135 75	0.150 87	0.174 69	0.216 89	0.261 50	0.307 82
15	0.077 83	0.089 94	0.102 96	0.116 83	0.131 47	0.146 82	0.171 02	0.213 88	0.259 12	0.305 98
16	0.073 65	0.085 82	0.098 95	0.112 98	0.127 82	0.143 39	0.16795	0.211 44	0.257 24	0.304 58
17	0.069 97	0.082 20	0.095 44	0.109 63	0.124 66	0.140 46	0.165 37	0.209 44	0.255 76	0.303 51
18	0.066 70	0.078 99	0.092 36	0.106 70	0.121 93	0.137 94	0.163 19	0.207 81	0.254 59	0.302 69
19	0.063 78	0.076 14	0.089 62	0.104 13	0.119 55	0.135 76	0.161 34	0.206 46	0.253 66	0.302 07
20	0.061 16	0.073 58	0.087 18	0.101 85	0.117 46	0.133 88	0.159 76	0.205 36	0.252 92	0.301 59
21	0.058 78	0.071 28	0.085 00	0.099 83	0.115 62	0.132 24	0.158 42	0.204 44	0.252 33	0.301 22
22	0.056 63	0.069 20	0.083 05	0.098 03	0.114 01	0.130 81	0.157 27	0.203 69	0.251 86	0.300 94
23	0.054 67	0.067 31	0.081 28	0.096 42	0.112 57	0.129 56	0.156 28	0.203 07	0.251 48	0.300 72
24	0.052 87	0.065 59	0.079 68	0.094 98	0.111 30	0.128 46	0.155 43	0.202 55	0.251 19	0.300 55
25	0.051 22	0.064 01	0.078 23	0.093 68	0.110 17	0.127 50	0.154 70	0.202 12	0.250 95	0.300 43
26	0.049 70	0.062 57	0.076 90	0.092 51	0.109 16	0.126 65	0.154 07	0.201 76	0.250 76	0.300 33
27	0.048 29	0.061 24	0.075 70	0.091 45	0.108 26	0.125 90	0.153 53	0.201 47	0.250 61	0.300 25
28	0.046 99	0.060 01	0.074 59	0.090 49	0.107 45	0.125 24	0.153 06	0.201 22	0.250 48	0.300 19
29	0.045 78	0.058 88	0.073 58	0.089 62	0.106 73	0.124 66	0.152 65	0.201 02	0.250 39	0.300 15
30	0.044 65	0.057 83	0.072 65	0.088 83	0.106 08	0.124 14	0.152 30	0.200 85	0.250 31	0.300 11

has a salvage value at the end of the project period n . The salvage value costs should be included in project cost analyses. One method to calculate salvage value in an annual cost analysis is as follows:

$$\text{Salvage credit} = [(\text{initial cost} - \text{salvage value}) \times \text{CRF}] + (\text{salvage value} \times i)$$

where CRF = capital recovery factor

i = interest rate per period

Present Worth Cost Analysis. In a present worth cost analysis, all project costs are ultimately presented in terms of dollars spent at a selected date. All O&M costs are converted into present worth dollars by the use of a present-worth factor. As with the capital recovery factor, the calculation of a present worth factor requires that an estimate be made of the interest rate and project life. These variables are then inserted into the following formula:

$$PWF = \frac{(1+i)^n - 1}{i(1+i)^n}$$

where PWF = present worth factor

i = interest rate per period

n = number of periods

The present worth factor is the inverse of the capital recovery factor. A tabulation of present worth factors is presented as Table 1.25.

Cost-Effective Analysis. Either an annual cost estimate or a present worth cost estimate can be made for a given project. Given the same interest rate and the same project life, comparisons of alternatives by annual cost estimates or by present worth cost estimates will always be the same. An example of a cost comparison

TABLE 1.25 Present Worth Factors

Number of periods n	Interest rate per period i									
	2	4	6	8	10	12	15	20	25	30
1	0.980	0.962	0.943	0.926	0.909	0.893	0.870	0.833	0.800	0.769
2	1.942	1.886	1.833	1.783	1.736	1.690	1.626	1.528	1.440	1.361
3	2.884	2.775	2.673	2.577	2.487	2.402	2.283	2.106	1.952	1.816
4	3.808	3.630	3.465	3.312	3.170	3.037	2.855	2.589	2.362	2.166
5	4.713	4.452	4.212	3.993	3.791	3.605	3.352	2.991	2.689	2.436
6	5.601	5.242	4.917	4.623	4.355	4.111	3.784	3.326	2.951	2.643
7	6.472	6.002	5.582	5.206	4.868	4.564	4.160	3.605	3.161	2.802
8	7.325	6.733	6.210	5.747	5.335	4.968	4.487	3.837	3.329	2.925
9	8.162	7.435	6.802	6.247	5.759	5.328	4.772	4.031	3.463	3.019
10	8.983	8.111	7.360	6.710	6.144	5.650	5.019	4.192	3.571	3.092
11	9.787	8.760	7.887	7.139	6.495	5.938	5.234	4.327	3.656	3.147
12	10.575	9.385	8.384	7.536	6.814	6.194	5.421	4.439	3.725	3.190
13	11.348	9.986	8.853	7.904	7.103	6.424	5.583	4.533	3.780	3.223
14	12.106	10.563	9.295	8.244	7.367	6.628	5.724	4.611	3.824	3.249
15	12.849	11.118	9.712	8.559	7.606	6.811	5.847	4.675	3.859	3.268
16	13.578	11.652	10.106	8.851	7.824	6.974	5.954	4.730	3.887	3.283
17	14.292	12.166	10.477	9.122	8.022	7.120	6.047	4.775	3.910	3.295
18	14.992	12.659	10.828	9.372	8.201	7.250	6.128	4.812	3.928	3.304
19	15.678	13.134	11.158	9.604	8.365	7.366	6.198	4.844	3.942	3.311
20	16.351	13.590	11.470	9.818	8.514	7.469	6.259	4.870	3.954	3.316
21	17.011	14.029	11.764	10.017	8.649	7.562	6.312	4.891	3.963	3.320
22	17.658	14.451	12.042	10.201	8.772	7.645	6.359	4.909	3.970	3.323
23	18.292	14.857	12.303	10.371	8.883	7.718	6.399	4.925	3.976	3.325
24	18.914	15.247	12.550	10.529	8.985	7.784	6.434	4.937	3.981	3.327
25	19.523	15.622	12.783	10.675	9.077	7.843	6.464	4.948	3.985	3.329
26	20.121	15.983	13.003	10.810	9.161	7.896	6.491	4.956	3.988	3.330
27	20.707	16.330	13.211	10.935	9.237	7.943	6.514	4.964	3.990	3.331
28	21.281	16.663	13.406	11.051	9.307	7.984	6.534	4.970	3.992	3.331
29	21.844	16.984	13.591	11.158	9.370	8.022	6.551	4.975	3.994	3.332
30	22.396	17.292	13.765	11.258	9.427	8.055	6.566	4.979	3.995	3.332

using both methods is presented in Table 1.26. In this example, the conclusion that Alternative B is some 10 percent less costly than Alternative A is the same regardless of which method is used.

Cost Sensitivity Analysis

Cost sensitivity is the influence of a specific cost element on the results of the overall cost estimate. Thus, if an element can be varied over a wide range of values without significantly affecting the overall estimate, the estimate is said to be insensitive to that particular element. In contrast, if a small change in the estimate of one element will substantially modify the overall estimate, the ranking is said to be highly sensitive to that element.

Since all cost estimates are subject to some uncertainty, the cost sensitivity analysis can be helpful. The factors that must be considered in a sensitivity analysis are (1) the sensitivity of the cost estimate to each major component and (2) the confidence level of the accuracy of each cost component. Thus, a high confidence

TABLE 1.26 Cost-Effectiveness Analysis of Alternatives

Item	Value or cost	
	Annual cost analysis	Present-worth analysis
Basic assumptions		
• Study period n	20 years	20 years
• Interest rate i	10%/yr	10%/yr
• Capital recovery factor (CRF)	$\frac{(i)(1+i)^n}{(1+i)^n - 1} = 0.1175$	
• Present worth factor	—	$\frac{(1+i)^n - 1}{(i)(1+i)^n} = 8.514$
Alternative A		
Capital cost	\$10,000,000	\$10,000,000
Annual operating cost	300,000/yr	300,000/yr
Annualized capital cost	1,175,000/yr	—
Annual operating cost	300,000/yr	—
Total annual cost	\$1,475,000/yr	—
Capital cost	—	\$10,000,000
Present worth of annual operating cost	—	2,554,000
Total present worth cost	—	\$12,554,000
Alternative B		
Capital cost	\$ 8,000,000	\$ 8,000,000
Annual operating cost	400,000/yr	400,000/yr
Annualized capital cost	\$ 940,000/yr	—
Annual operating cost	400,000/yr	—
Total annual cost	\$ 1,340,000/yr	—
Capital cost	—	\$8,000,000
Present worth of annual operating cost	—	3,406,000
Total annual cost	—	\$11,406,000

level can be assured if the cost elements which are most sensitive are also very accurate. A lower confidence level must be considered for an estimate whose most sensitive components cannot be estimated accurately.

As an example, the results of one sensitivity analysis are presented in Table 1.27. This example is from a study that evaluated the potential economic impact of a proposed drinking water standard. Certain design assumptions were made about a treatment method (activated carbon adsorption), which may be required to meet the proposed standard. The sensitivity analysis evaluated the cost impact of changing several of the key design parameters affecting operation and maintenance costs. This analysis revealed the cost estimate to be most sensitive to reactivation frequency, carbon loss per reactivation cycle, and carbon cost.

Project Financing

The potential sources of financing for environmental projects are varied and depend upon such factors as the type of environmental project, the project size, the type of owner, and the current political and economic climate. Table 1.28 presents examples of the more common sources of funding for environmental projects in the United States.

TABLE 1.27 Cost Sensitivity Analysis (9)

	Values	Percent/Change
Parameters affecting O&M costs—additive		
Carbon loss per reactivation cycle, percent	15	29.6
	10	0
	5	-14.7
Carbon cost, ¢/lb	54	16.9
	38	0
	19	-22.0
Fuel cost, \$/mil Btu	1.89	4.0
	1.26	0
	0.63	-3.8
Power cost, ¢/kWh	1.5	0.6
	1.0	0
	1.5	-0.6
Direct hourly wage rate \$/h	7.78	15.9
	5.19	0
	2.60	-15.9
Wholesale price index	267	6.0
	178	0
	89	-5.0
Parameters affecting O&M costs—multiplicative		
Reactivation frequency, weeks between	3	145.9
	6	100.0
	9	72.1
Capacity factor, percent	50	105.2
	70	100.0
	100	92.4

TABLE 1.28 Common Sources of Financing for Pollution Control Facilities in the United States

Common sources of financing	Type of Owner				
	Municipalities	Water and Sewer authorities	Investor-owned utilities	Private industry	Municipal corporations
Corporate bonds				•	•
Direct bank loans			•	•	
Federal grants and loans	•	•			•
State grants and loans	•	•			•
Retained earnings			•	•	
General obligation bonds	•				
Revenue bonds	•	•			•
Industrial revenue bonds			•	•	

In the developing nations, additional funding sources, such as multinational agencies, play a significant role. Examples of these agencies are the World Bank, World Health Organization, and the Agency for International Development.

STUDIES AND DESIGNS

Engineering and planning studies can be categorized into three levels, depending upon their objectives. This section examines common environmental studies within each of these categories. Also, three levels of design development are discussed.

Policy Studies

Environmental studies for overall policy making vary depending upon the entity that is conducting the study. In the federal government, policy studies are undertaken to assist in the development and implementation of environmental regulations. Local, state, and regional governments often undertake comprehensive community planning studies to establish priorities for specific projects. Industries commonly conduct environmental audits to assess the needs for making improvements.

Federal Studies. Environmental policy studies, i.e., those directed at determining a course of action, are commonly performed in the federal sector by the EPA as well as several other agencies with major environmental interests. EPA has the greatest impact on the environment through development of standards, regulations, and guidelines for the regulated community. The direction of federal regulations generally influences agencies at the regional, state, and local levels, and therefore has a “ripple” effect throughout the country. Policy studies are conducted in all media (air, water, and solid and hazardous wastes) and may have as their purpose the development or evaluation of basic scientific data, application or extrapolation of basic information into the “real world,” and evaluation of secondary or induced effects, including economic impacts.

The specific involvement of environmental engineers could be in such activities as wastewater treatability studies to define permissible effluent levels, measurement of emission levels from air pollution control devices to identify the most effective control approach, and evaluation of the compatibility of liner materials with various hazardous wastes to determine the most dependable material for specific applications.

Policy also involves the estimation of secondary impacts, such as the economic impact on an industrial segment or a geographic area. In this activity, the environmental engineer would work closely with other scientific disciplines, e.g., economists, demographers, and planners, in a team approach.

Environmental engineers typically serve at the highest levels within the agencies setting environmental policy, taking advantage of their broad background, in interpreting data similar to that discussed above.

Community Planning. The comprehensive community plan is prepared to take an overall look at a total region or a specific part of a region within the context of the larger region of which it is a part. The plan is based on an extensive review of the natural and socioeconomic resources in the community. Report recommendations lead to policy decisions that establish priorities to implement specific projects. A combination of talents is required to prepare a proper comprehensive plan. These may include architects, geologists, attorneys, ecologists, civil engineers, economists, environmental engineers, geographers, hydrologists, landscape architects, planners, political scientists, sociologists, soils scientists, and systems engineers. The process of comprehensive community planning is illustrated in Figure 1.13. A typical outline for a regional planning report is presented in Table 1.29.

Environmental Audits. An environmental audit is an objective assessment by independent investigators of the extent of compliance of a corporation or governmental facility with applicable environmental laws and regulations. It is based upon a review of pertinent records and technical data. The audit is generally undertaken for one of the following purposes:

1. Assuring compliance with federal, state, and local regulations
2. Reducing environmental risks and liabilities
3. Cost saving or increasing the efficiency of corporate operations
4. Identifying environmental liabilities prior to purchase or sale of property

Coupled with the intense interest on the part of industry, a number of federal agencies have indicated that they feel an auditing program founded on a joint employee–management committee concept would be an effective mechanism to address compliance.

In January, 1982, the Occupational Safety and Health Administration (OSHA) published proposed procedures to establish voluntary programs to supplement enforcement of the OSHA regulations within the workplace. The concept is based on employee–management committees with safety and health responsibilities. The committees would have equal representation, hold regular and documented meetings, and conduct periodic inspections within the workplace. The advantage of the program is that it would specifically lessen the need for more formalized OSHA inspections, thereby minimizing the pressure on the limited resources of the OSHA administration. Many of the concepts as proposed by OSHA are now being extended to environmental auditing as well.

EPA followed the lead of OSHA by publishing an Environmental Auditing Policy Statement in the *Federal Register* on November 8, 1985. The summary stated: “It is EPA policy to encourage the use of environmental auditing by regulated entities to help achieve and maintain compliance with environmental laws and regulations, as well as to help identify and correct unregulated environmental hazards.”

The following four-step approach to conducting an audit has been effectively utilized by many firms.

1. General overview of a company’s operations as they relate to requirements of such federal agencies as OSHA and EPA, as well as corporate policies.
2. An analysis of selected regulations and their potential impact on the plants to be studied, as well as the development of the actual documents needed to perform an audit. This step generally includes the preparation of audit questionnaires, such as the example presented in Table 1.30.

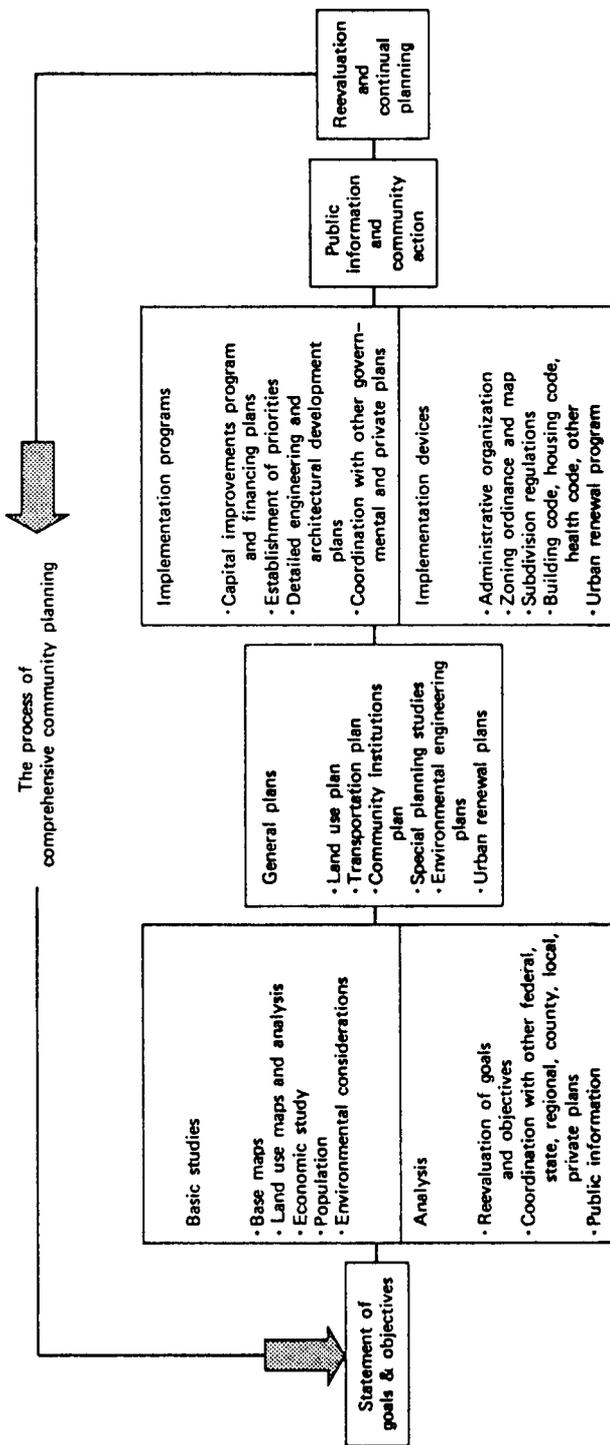


FIG. 1.13 Comprehensive community planning (10).

TABLE 1.29 Typical Regional Planning Report (*II*)

-
1. Letter of transmittal to the contracting agency
 2. Acknowledgments
 3. Table of contents
 - a. List of tables
 - b. List of figures
 4. Findings, conclusions, and recommendations
 5. Purpose and scope
 6. Background data and analysis, as applicable
 - a. Geography, hydrology, meteorology, geology, and groundwater levels
 - b. Population density and characteristics—past, present, future
 - c. Soil characteristics; flora and fauna
 - d. Transportation and mobility; adequacy and effects produced—present and future
 - e. Residential, industrial, commercial, recreational, agricultural, and institutional development and redevelopment
 - f. Land use—present and future; spread of blight and obsolescence; inefficient and desirable land uses
 - g. Drainage, water pollution control, and flood control management
 - h. Water resources, multiuse planning and development with priority to water supply; environmental impact
 - i. Air and water pollution, sewerage, and solid waste management
 - j. Public utilities—electricity, gas, oil, heat—and their adequacy
 - k. Educational and cultural facilities, size, location, effects
 - l. Economic studies—present sources of income, future economic base and balance, labor force, markets, industrial opportunities, retail facilities, stability
 - m. Sociological factors—characteristics, knowledge, attitudes, behavior of the people and their expectations
 - n. Local government, political organizations, and laws, codes, ordinances
 - o. Special problems, previous studies and findings, background data, including tax structure and departmental budgets
 7. Project study. This would be a regional or area-wide in-depth study of one or more projects or functions such as solid waste management, water supply, recreation, vector control, wastewater, or environmental health
 8. The comprehensive regional plan
 - a. Alternative solutions and plans
 - b. Economic, social, and ecologic evaluation of alternatives
 - c. The recommended regional plan.
 - d. Site development and reuse plans
 9. Administration and financing
 - a. Public information
 - b. Administrative arrangements, management, and costs
 - c. Financing methods—general obligation bonds, revenue bonds, special assessment bonds; also grants, incentives, federal and state aid
 - d. Cost distribution, service charges and rates; capital costs—property, equipment, structures, engineering and legal services; annual costs to repay capital costs, principal and interest, taxes. Regular and special service charges and rates
 - e. Legislation, standards, inspection, and enforcement
 - f. Evaluation, research and replanning
 10. Appendices
 - a. Applicable laws
 - b. Special data
 - c. Charts, tables, illustrations
 11. Glossary
 12. References
-

TABLE 1.30 Excerpt from an Environmental Audit Questionnaire

-
1. Have all personnel who handle hazardous waste been trained?
 - a. On-the-job? Yes _____ No _____
 - b. Classroom? Yes _____ No _____
 2. Was the training director formally trained? Yes _____ No _____
 3. Does the training program include:
 - a. Procedures to use, inspect, repair, replace emergency and monitoring equipment? Yes ____ No ____
 - b. Key parameters for automatic waste feed cut-off systems? Yes _____ No _____
 - c. Communications and alarm systems? Yes _____ No _____
 - d. Response to fires or explosions? Yes _____ No _____
 - e. Response to groundwater contamination? Yes _____ No _____
 - f. Operations shutdown? Yes _____ No _____
 - g. Contingency plan? Yes _____ No _____
 4. Are new employees trained within six (6) months? Yes _____ No _____
 - a. Are they continually supervised until trained? Yes _____ No _____
 5. Is there a training review or update annually? Yes _____ No _____
-

3. A regulatory compliance review consisting of 3 to 10 days on the plant site (depending on the size and complexity of the operation).
4. A comprehensive analysis of the company's compliance posture, i.e., how things stand today compared to what they should be.

A guide to topics that should be covered in an environmental audit is presented in Table 1.31. State and local regulations must be reviewed and incorporated into the audit protocol, since often these are more specific than the Federal regulations.

Feasibility Studies. Once a project has been identified, the next step is a preliminary or feasibility study to consider in detail the implementation of alternatives, together with approximate costs. For example, water system study alternatives for a region might include (1) possible service areas and combinations; (2) a well-water supply with water softening and iron removal; or (3) an upland lake or reservoir with multipurpose uses requiring land acquisition, water rights, and a conventional water treatment plant.

At the feasibility stage, no detailed engineering is prepared. However, the engineering, legal, economic, and sociopolitical feasibility of each alternative is presented together with recommendations, cost estimates, and methods of financing each alternative. A summary of some common types of environmental feasibility studies that may be performed is presented in Table 1.32.

The feasibility study report must establish the potential alternative; an evaluation of the engineering, legal, economic, and sociopolitical advantages and disadvantages of each alternative; and final conclusions and recommendations for the preferred alternative. This report is the basis for the design and ultimate construction of a facility. Table 1.33 illustrates a typical feasibility report for a solid waste study.

Design Development

The design development for most pollution control facilities consists of the following three phases of activity, which occur sequentially and must be completed prior to commencing construction.

1. Conceptual design
2. Preliminary design
3. Detailed design

TABLE 1.31 Typical Environmental Audit Topics (12)

Area of compliance	Regulations and permitting/example concerns
1. Facility Description	<i>Regulations:</i> None specific (Good Engineering Practices). <i>Concerns:</i> Develop an understanding of the facility, its boundaries, prior uses, age and history, scope of operations, raw materials, processes, products, services, and any past problems or issues.
2. Water Supply	<i>Regulations:</i> Safe Drinking Water Act (SDWA). <i>Concerns:</i> Contaminants in private wells, lead contamination due to piping, and bacteria.
3. Asbestos Management	<i>Regulations:</i> Clean Air Act (CAA), Occupational Safety and Health Act (OSHA), and National Emission Standards for Hazardous Air Pollutants (NESHAP). <i>Concerns:</i> Presence of Asbestos Containing Materials (ACM), condition of ACM, renovations potentially releasing ACM, and proper disposal of ACM.
4. Polychlorinated Biphenyl (PCB) Management	<i>Regulations:</i> Toxic Substance Control Act (TSCA). <i>Concerns:</i> Presence of equipment containing PCBs, proper marking and maintenance of equipment, and proper disposal.
5. Hazardous Material Management and Underground Storage Tanks (UST)	<i>Regulations:</i> OSHA, Resource Conservation and Recovery Act (RCRA), Comprehensive Response, Compensation and Liability Act (CERCLA), National Contingency Plan (NCP), Superfund Amendments and Reauthorization Act (SARA), and TSCA. <i>Concerns:</i> Potential for release to the environment, proper storage, community emergency response plans, reporting of storage quantities or releases, and proper installation, monitoring, and testing of USTs.
6. Right to Know	<i>Regulations:</i> OSHA and SARA. <i>Concerns:</i> Material Safety Data Sheets (MSDS) complete for chemicals used and posted, SARA Title III reporting, hazardous communications programs, and emergency response planning procedures.
7. Pesticide Usage	<i>Regulations:</i> Federal Insecticide, Fungicide, and Rodenticide Act (FIFRA). <i>Concerns:</i> Use of licensed applicators, use of approved pesticides, and proper disposal of containers and residuals.
8. Air Emissions	<i>Regulations:</i> CAA, OSHA, and SARA. <i>Concerns:</i> Release exceeding National Ambient Air Quality Standards (NAAQS) or NESHAP, Prevention of Significant Deterioration (PSD) permitting, worker exposure levels, release reporting under SARA, and potential for radon exposure.
9. Wastewater	<i>Regulations:</i> Clean Water Act (CWA), National Pollutant Discharge Elimination System (NPDES) permitting, and RCRA. <i>Concerns:</i> Maintaining discharges within NPDES or pretreatment limits, sludges may be listed hazardous wastes, operation properly maintained, use of leach pits or fields, and control of storm water.
10. Solid Waste	<i>Regulations:</i> Solid Waste Disposal Act (SWDA)-primarily administered by States, and RCRA. <i>Concerns:</i> Proper construction and operation of landfills, separate disposal of hazardous waste, past on-site disposal, and former solid waste management units (SWMU).
11. Hazardous Waste	<i>Regulations:</i> RCRA, CERCLA, and OSHA. <i>Concerns:</i> Status as a generator, transporter, and treatment, storage, and/or disposal facility; obtaining proper permits; adequate preparedness and prevention procedures, record keeping, waste analysis, facility design, groundwater monitoring, contingency planning, training, closure plans, and financial assurance for closure and liability coverage; and assessment of contaminant release from former SWMUs.

(continues)

TABLE 1.31 Typical Environmental Audit Topics (*continued*)

Area of compliance	Regulations and permitting/example concerns
12. Spill and Emergency Planning	<i>Regulations:</i> CWA, RCRA, and SARA. <i>Concerns:</i> Adequate Spill Prevention control and Countermeasure (SPCC) Plan for oil and petroleum units, adequate contingency plan for hazardous waste units; adequate emergency response plan for hazardous material units; sufficient on-site spill control materials; agreements with outside contractors and responding agencies such as police, fire and hospitals; and adequate training for response.
13. Due Diligence	<i>Regulations:</i> CERCLA, SARA, and RCRA. <i>Concerns:</i> Current property owner may be responsible for clean up of past environmental release sites, prepurchase review of environmental liability, review of past disposal of practices; sampling for contamination of soil, groundwater, surface water, and air; sampling for hazardous materials, PCB, asbestos, radon, which are potential liabilities, review of previous owners and uses, and impacts of adjacent properties.

Conceptual Design. A conceptual design is the key output of the engineering studies conducted for a new facility. At a minimum, conceptual process design will contain the following elements:

1. Proposed process flow diagram
2. System mass balance
3. Conceptual site layout
4. Medium (water, air, solid waste, liquid waste) characteristics
5. Estimated discharge characteristics

A conceptual process flow diagram and system mass balance for typical wastewater treatment facilities are presented in Figs. 1.14 and 1.15, respectively (pages 1.63 and 1.64).

Preliminary Design. The preliminary design is conducted following the conceptual facilities design to more definitely describe the recommended facilities using preliminary drawings and other supporting information. This phase of design is accomplished prior to beginning work on the detailed design disciplines, i.e., process or mechanical, structural, electrical, etc. Typical preliminary design activities are listed in Table 1.34 (page 1.65).

All of the elements of the preliminary design are typically combined into a basis of design report, which serves the following functions:

- Provides a convenient review point for the client to monitor the progress of the project and determine whether the project is within the capital budget
- Optimizes communications between the various disciplines within the design team
- Provides a document for a preliminary value engineering review
- Serves as a preliminary submittal application to agencies requiring permits
- Provides information for a formal in-house review

The report should be dynamic and lend itself to easy updating as the design proceeds. A table of contents for a typical basis-of-design report is presented in Table 1.35 (page 1.66).

TABLE 1.32 Types of Environmental Engineering Studies

General

Environmental impact study for siting new facilities

Regional planning study

- Epidemiological survey
- Public water supply, treatment, distribution
- Wastewater collection, treatment, and disposal
- Solid waste management, storage, collection, transportation, processing, and disposal
- Air resources management
- Air pollution control

Water Supply

Municipal water supply source studies

Water treatment plant feasibility study

Water treatment plant operations optimization study

Water plant distribution study

Agricultural water resources planning study

Water reclamation and reuse study

Water resources management

Water treatment plant sludge-handling study

Wastewater Treatment

Facilities plan for wastewater management

Waste characterization study

Process waste treatment feasibility study

Pretreatment evaluation

Waste treatment plant operations optimization

Advanced waste treatment study

Granular activated carbon

Powder activated carbon with activated sludge

Ozonation

Reverse osmosis

Marine waste disposal

Solid Waste Sludge-Handling and Disposal

Solid waste collection and routing study

Solid and hazardous waste characterization/classification

Hazardous waste treatment evaluation study

Leachate generation

Leachate treatment

Groundwater monitoring and control studies

Waste Toxicity Studies

Hazardous Waste Disposal Evaluation

Incineration

Secure landfill siting

Land treatment

Air quality and air pollution control

Monitoring

Stack sampling

Emission inventories

Meteorological studies

Dispersion estimates

Diffusion modeling

Water quality

Surface water characterization

Groundwater characterization

Surface water modeling

(continues)

TABLE 1.32 Types of Environmental Engineering Studies (*continued*)

Water quality (<i>continued</i>)
Groundwater modeling
Hydrologic analyses
Assimilative capacity studies
Oceanographic studies
Aquatic ecology studies
Ambient noise and noise pollution control
Traffic and transportation noise survey
Community and industrial noise surveys
Product noise evaluation
Land resources studies
Industrial hygiene survey
Radiological health survey
Ecological studies
Site selection and permitting assistance
Program planning
Identification and comparative assessment of alternate sites
Environmental baseline studies
Socioeconomic and human impact evaluations

TABLE 1.33 Typical Feasibility Report for a Solid Waste Study (*11*)

-
1. Background information and data analysis, including residential, commercial, industrial, and agricultural solid wastes
 - a. Field surveys and investigations
 - b. Existing methods and adequacy of collection, treatment, and disposal and their costs
 - c. Characteristics of the solid wastes
 - d. Quantities, summary tables, and projections
 - e. Waste reduction at source, salvage, and reuse
 2. Solid waste collection, including transportation
 - a. Present collection routes, restrictions, practices, and costs
 - b. Equipment and methods used
 - c. Handling of special wastes
 - d. Recommended collection systems.
 3. Preliminary analyses for solid waste treatment and disposal
 - a. Available treatment and disposal methods (advantages and disadvantages)—compaction, shredding, sanitary landfill, incinerator, high-temperature incinerator, pyrolysis, fluidized bed oxidation, bulky waste incinerator, waste heat recovery, composting, garbage grinders
 - b. Pretreatment devices—shredders, hammer mills, hoppers, compactors—and their applicability
 - c. Disposal of special wastes—automobile, water and wastewater treatment plant sludges, scavenger wastes, industrial sludges and slurries, waste oils, toxic and hazardous wastes, rubber tires, agricultural wastes, pesticides, forestry wastes.
 - d. Treatment and disposal of industrial wastes
 - e. Transfer stations, facilities, and equipment
 - f. Rail haul, barge haul, other
 - g. Alternative solutions and costs.
 4. Review of possible solutions
 - a. Social and political factors
 - b. Existing and potential best land use within 1500 ft of treatment and disposal site and aesthetic considerations
 - c. Site development and reuse plans
 - d. Special inducements needed
 - e. Preliminary public information and education
 - f. Capital and operating costs.
 5. Recommended solution.
-

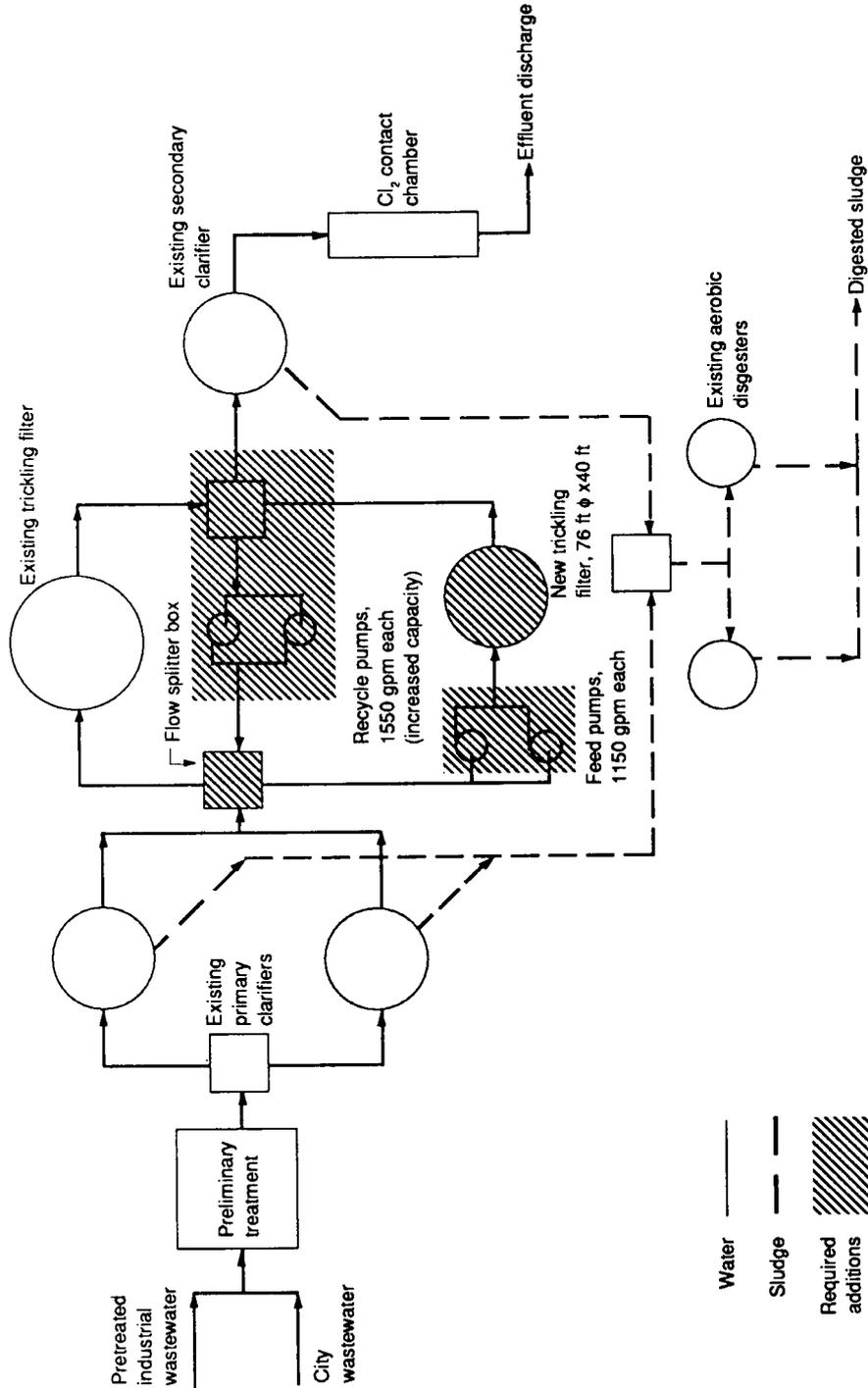


FIG. 1.14 Conceptual process flow diagram for a wastewater treatment facility.

Line #	QAVE	QMAX	QMIN	BOD, lb/day	COD, lb/day	TSS, lb/day	MC, lb/day	F, lb/day	MCF, lb/day	TCE, lb/day	PCE, lb/day	B, lb/day	T, lb/day	X, lb/day	EGME, lb/day	IPA, lb/day
1	50 gpm	75 gpm	0	900	1200	—	73.1	3.6	445.3	1.2	6.6	4.0	6.7	2.0	420	—
2	20.6 gpm	25 gpm	0	665	2351	—	—	—	—	—	—	—	—	14	—	682
3	50 gpm	75 gpm	0	900	1200	—	66.1	3.6	345.3	1.2	3.0	0.6	2.4	0.9	420	—
4	70.6 gpm	100 gpm	0	1565	3551	—	66.1	3.6	345.3	1.2	3.0	0.6	2.4	14.9	420	682
5	70.8 gpm	101.5 gpm	0	1565	3551	—	5.7	<0.07	1.5	<0.04	<0.02	<0.03	<0.03	<0.03	420	682
6	188 gpm	347 gpm	0	—	—	11,970	5.7	<0.07	1.5	<0.04	<0.02	<0.03	<0.03	<0.03	21	34
7	91 gpm	147 gpm	0	27	178	27	5.5	<0.07	1.5	<0.04	<0.02	<0.03	<0.03	<0.03	21	34
8	87 gpm	147 gpm	0	<27	178	178	5.7	<0.07	1.5	<0.04	<0.02	<0.03	<0.03	<0.03	21	34
9	87 gpm	147 gpm	0	11	<178	5.5	<0.055	<0.055	<0.055	<0.055	<0.02	<0.01	<0.01	<0.01	21	34
10	71 gpm	147 gpm	0	8.5	<178	4.3	<0.043	<0.043	<0.043	<0.043	<0.017	<0.009	<0.009	<0.009	21	34
11	210 gpd	—	0	—	—	—	7.13	<0.1	100.7	<0.1	3.6	3.4	4.3	1.1	—	—
12	140 gpd	5 gpm	0	—	—	—	0.13	—	0.7	—	—	—	—	—	—	—
13	12,000 gal/transfer	—	0	—	—	—	—	—	—	—	—	—	—	—	—	—
14	10,000 gal/transfer	—	0	—	—	—	—	—	—	—	—	—	—	—	—	—
15	22,000 gal/transfer	—	0	—	—	—	—	—	—	—	—	—	—	—	—	—
16	12,000 gal/transfer	—	0	—	—	—	—	—	—	—	—	—	—	—	—	—
17	1500 SCFM	1500 SCFM	0	—	—	—	—	—	—	—	—	—	—	—	—	—
18	1500 SCFM	1500 SCFM	0	—	—	—	65.5	3.6	344.7	1.2	3.0	0.6	2.4	14.9	—	157
19	1500 SCFM	1500 SCFM	0	—	—	—	—	—	—	—	—	—	—	—	—	—
20	2400 lb/day	650 lb/hr	0	—	—	—	—	—	—	—	—	—	—	—	—	—
21	2400 lb/day	650 lb/hr	0	—	—	—	65.5	3.6	344.7	1.2	3.0	0.6	2.4	14.9	—	157
22	360 gpd	=1.8 gpm	0	—	—	—	65.5	3.6	344.7	1.2	3.0	0.6	2.4	14.9	—	157
23	70 gpd	=0.3 gpm	0	—	—	—	60.4	3.6	343.8	1.2	3.0	0.6	2.4	14.9	—	157
24	290 gpd	=1.5 gpm	0	—	—	—	5.1	0.04	0.9	0.01	0.01	0.02	0.02	0.02	—	157
25	70 gpd	5 gpm	0	—	—	—	7	<0.1	100	<0.1	3.6	3.4	4.3	1.1	—	—
26	54 gpm	60 gpm	0	—	—	—	—	—	—	—	—	—	—	—	—	—
27	54 gpm	60 gpm	0	—	—	—	—	—	—	—	—	—	—	—	—	—
28	95 gpm	200 gpm	0	—	—	11410	—	—	—	—	—	—	—	—	—	—
29	7,100 gpd	200 gpm	0	—	—	590	—	—	—	—	—	—	—	—	—	—
30	3,170 gpd	20 gpm	0	—	—	—	—	—	—	—	—	—	—	—	—	—
31	3,930 gpd	20 gpm	0	—	—	590	—	—	—	—	—	—	—	—	—	—
32	24,000 gpd	1200 gpm	0	—	—	1.0	—	—	—	—	—	—	—	—	—	—
33	24,000 gpd	1200 gpm	0	—	—	20	—	—	—	—	—	—	—	—	—	—
34	4,000 gpd	75 gpm	0	—	—	21.5	—	—	—	—	—	—	—	—	—	—
35	28,000 gpd	25 gpm	0	—	—	41.5	—	—	—	—	—	—	—	—	—	—
36	1.6 gpd	16 gpd	0	—	—	—	—	—	—	—	—	—	—	—	—	—
37	130 lb/day	—	0	—	—	—	—	—	—	—	—	—	—	—	—	—
38	0.67 gph	—	0	—	—	—	—	—	—	—	—	—	—	—	—	—

FIG. 1.15 Mass balance for an industrial wastewater treatment facility.

TABLE 1.34 Typical Preliminary Design Activities

-
1. Prepare preliminary design drawings
 - a. Process and instrumentation diagrams
 - b. Material balance
 - c. Site plan(s)
 - d. Hydraulic profiles
 - e. Equipment layout drawings
 2. Develop major equipment list
 - a. Size and capacity
 - b. Materials of construction
 - c. Qualified vendors
 3. Obtain soils, foundation, and topographic information
 - a. Review existing data on waste treatment plant site
 - b. Identify requirements for additional field work (if any)
 - c. Perform surveying (if required)
 - d. Perform soils/foundation testing (if required)
 4. Develop final design criteria
 - a. Process and mechanical
 - b. Instrumentation and controls
 - c. Electrical
 - d. Civil, site, foundation
 - e. Structural
 - f. Architectural
 5. Estimate design engineering costs
 - a. Prepare preliminary list of design drawings
 - b. Estimate design engineering costs based on historical records of cost per sheet of drawing for each design discipline
 6. Estimate construction costs
 - a. Update major equipment cost estimates
 - b. Takeoff instrumentation and control items and estimate their costs
 - c. Update other elements of the design
 7. Prepare basis of design report
 - a. Prepare draft report
 - b. Obtain comments from client and in-house reviews
 - c. Finalize report
-

Detailed Design. A complete detailed design package of construction documents is required to enable a competent contractor to build the required facilities. The detailed design also includes equipment specifications that are sufficient to bid and purchase all required equipment. Typical detailed design activities are illustrated in Table 1.36. The sequencing of preliminary and detailed design tasks for a wastewater treatment facility is shown in Fig. 1.16 (page 1.68). A construction specification outline is illustrated in Table 1.37 (page 1.67).

Reliability

Objectives for a facility's performance reliability are site-specific. For example, a water treatment plant with a protected water supply watershed and large volumes of finished water storage must be reliable but can safely operate without the extensive reliability criteria required for a hazardous waste remediation or wastewater treatment facility upstream of a surface water supply. Maximum reliability is achieved when no single component failure at full design conditions can impair the performance of any individual unit or the facility as a whole.

TABLE 1.35 Typical Basis of Design Report Outline

1.	Introduction
2.	Pre-design engineering
3.	Process and mechanical design
a.	Equipment design criteria
b.	Equipment list
c.	HVAC requirements
d.	Fire protection
e.	Corrosion protection
4.	Civil or site design
a.	Rough grading
b.	Finish grading
c.	Dewatering
d.	Drainage
e.	Foundation requirements
f.	Utilities
5.	Architectural design
a.	Preliminary building design
6.	Structural design
a.	Reinforced concrete
b.	Structural steel
7.	Electrical design
a.	Power distribution
b.	Lighting
8.	Instrumentation and controls design
a.	Control philosophy
b.	Hardware requirements
c.	Computer interfaces
9.	Construction considerations
Appendix A	Treatability Study Data
Appendix B	Preliminary design drawings
	Process and instrumentation diagrams
	Material balance
	Site plan(s)
	Hydraulic profiles
	Equipment layout drawings
Appendix C	Soil and foundation report

The U.S. Environmental Protection Agency (EPA) has reliability guidelines (30) for wastewater treatment facilities that describe precautions applicable to a variety of environmental engineering projects. Further enhancements may include design features and operational strategies, including instrumentation and operator interface. During design, failure mode/effects analyses can be used to test the application and acceptability of reliability guidelines for individual components.

Facility reliability is achieved through backup equipment, multiple units, and an overall system providing multiple opportunities to achieve facility performance objectives. Reliability can be designed into a facility by a separate analysis of risks, costs, and benefits or by the use of generally recognized criteria, such as EPA's reliability classification requirements.

The objective of EPA's reliability requirements are to cost-effectively design a system that will consistently protect public health, achieve water quality and pollution control objectives for both surface and

TABLE 1.36 Detailed Design Activities

-
1. Mechanical drawings
 - a. Process and mechanical drawings
 - b. HVAC
 - c. Piping
 2. Civil or site drawings
 - a. Grading and drainage
 - b. Roadways
 - c. Impoundments
 - d. Utilities
 3. Instrumentation and controls drawings
 - a. P&IDs
 - b. Loop diagrams
 - c. Signal paths
 - d. Instrument mounting details
 4. Structural and architectural drawings
 - a. Foundations
 - b. Reinforced concrete
 - c. Structural steel
 - d. Buildings
 5. Electrical drawings
 - a. Power distribution
 - b. Controls
 - c. Lighting
 6. Major equipment specifications
 - a. Process equipment
 - b. Substations and motor control centers
 - c. Control panels
 - d. Buildings (if prefabricated)
 7. Minor equipment specifications
 8. Construction specifications
 - a. Site work and concrete
 - b. Buildings
 - c. Mechanical
 - d. Electrical
 - e. Instrumentation and controls
 9. Final design review and corrections
 - a. In-house review
 - b. Client review
 - c. Final corrections
 10. Construction cost estimate
 - a. Detailed takeoff
 - b. Unit prices
 - c. Risk analysis to establish proper contingency level
 - d. Capital costs
-

TABLE 1.37 Construction Specification Outline (13)

Division 0 Bidding and contract requirements	03180 Form ties and accessories
00010 Prebid information	03200 Concrete reinforcement
00100 Instructions to bidders	03250 Concrete accessories
00200 Information available to bidders	03300 Cast-in-place concrete
00300 Bid/tender forms	03350 Special concrete finishes
00400 Supplements to bid/tender forms	03360 Specially placed concrete
00500 Agreement forms	03370 Concrete curing
00600 Bonds and certificates	03400 Precast concrete
00700 General conditions of the contract	03500 Cementitious decks
00800 Supplementary conditions	03600 Grout
00950 Drawings index	03700 Concrete restoration and cleaning
00900 Addenda and modifications	
Division 1 General requirements	Division 4 Masonry
01010 Summary of work	04050 Masonry procedures
01020 Allowances	04100 Mortar
01030 Special project procedures	04150 Masonry accessories
01040 Coordination	04200 Unit masonry
01050 Field engineering	04400 Stone
01060 Regulatory requirements	04500 Masonry restoration and cleaning
01070 Abbreviations and symbols	04550 Refractories
01080 Identification systems	04600 Corrosion-resistant masonry
01100 Alternates/alternatives	Division 5 Metals
01150 Measurement and payment	05010 Metal materials and methods
01200 Project meetings	05050 Metal fastening
01300 Submittals	05100 Structural metal framing
01400 Quality control	05200 Metal joists
01500 Construction facilities and temporary controls	05300 Metal decking
01600 Material and equipment	05400 Cold-formed metal framing
01650 Starting of systems	05500 Metal fabrications
01660 Testing, adjusting, and balancing of systems	05700 Ornamental metal
01700 Contract closeout	05800 Expansion control
	05900 Metal finishes
Division 2 Sitework	Division 6 Wood and plastics
02010 Subsurface investigation	06050 Fasteners and supports
02050 Demolition	06100 Rough carpentry
02100 Site preparation	06130 Heavy timber construction
02150 Underpinning	06150 Wood-metal systems
02200 Earthwork	06170 Prefabricated structural wood
02300 Tunnelling	06200 Finish carpentry
02350 Piles, caissons, and cofferdams	06300 Wood treatment
02400 Drainage	06400 Architectural woodwork
02440 Site improvements	06500 Prefabricated structural plastics
02480 Landscaping	06600 Plastic fabrications
02500 Paving and surfacing	Division 7 Thermal and moisture protection
02590 Ponds and reservoirs	07300 Shingles and roofing tiles
02600 Piped utility materials and methods	07400 Preformed roofing and siding
02700 Piped utilities	07500 Membrane roofing
02800 Power and communication utilities	07570 Traffic topping
02850 Railroad work	07600 Flashing and sheet metal
02880 Marine work	07800 Roof accessories
	07900 Joint sealants
Division 3 Concrete	Division 8 Doors and windows
03050 Concreting procedures	08100 Metal doors and frames
03100 Concrete formwork	08200 Wood and plastic doors
03150 Forms	

(continues)

TABLE 1.37 Construction Specification Outline (*continued*)

Division 8 Doors and windows (<i>cont.</i>)	10800 Toilet and bath accessories
08250 Door opening assemblies	10900 Wardrobe specialties
08300 Special doors	Division 11 Equipment
08400 Entrances and storefronts	11010 Maintenance equipment
08500 Metal windows	11020 Security and vault equipment
08600 Wood and plastic windows	11030 Checkroom equipment
08650 Special windows	11040 Ecclesiastical equipment
08700 Hardware	11050 Library equipment
08800 Glazing	11060 Theater and stage equipment
08900 Glazed curtain walls	11070 Musical equipment
Division 9 Finishes	11080 Registration equipment
09100 Metal support systems	11100 Mercantile equipment
09200 Lath and plaster	11110 Commercial laundry and dry cleaning equipment
09230 Aggregate coatings	11120 Vending equipment
09250 Gypsum wallboard	11130 Audiovisual equipment
09300 Tile	11140 Service station equipment
09400 Terrazzo	11150 Parking equipment
09500 Acoustical treatment	11160 Loading dock equipment
09550 Wood flooring	11170 Waste handling equipment
09600 Stone and brick flooring	11190 Detention equipment
09650 Resilient flooring	11200 Water supply and treatment equipment
0980 Carpeting	11300 Fluid waste disposal and treatment equipment
09700 Special flooring	11400 Food service equipment
09760 Floor treatment	11450 Residential equipment
09800 Special coatings	11460 Unit kitchens
09900 Painting	11470 Darkroom equipment
09950 Wall covering	11480 Athletic, recreational, and therapeutic equipment
Division 10 Specialities	11500 Industrial and process equipment
10100 Chalkboards and tackboards	11600 Laboratory equipment
10150 Compartments and cubicles	11650 Planetarium and observatory equipment
10200 Louvers and vents	11700 Medical equipment
10240 Grilles and screens	11780 Mortuary equipment
10250 Service wall systems	11800 Telecommunication equipment
10260 Wall and corner guards	11850 Navigation equipment
10270 Access flooring	Division 12 Furnishings
10280 Specialty modules	12100 Artwork
10290 Pest control	12300 Manufactured cabinets and casework
10300 Fireplaces and stoves	12500 Window treatment
10340 Prefabricated steeples, spires, and cupolas	12550 Fabrics
10350 Flagpoles	12600 Furniture and accessories
10400 Identifying devices	12670 Rugs and mats
10450 Pedestrian control devices	12700 Multiple seating
10500 Lockers	12800 Interior plants and plantings
10520 Fire extinguishers, cabinets, and accessories	Division 13 Special construction
10530 Protective covers	13010 Air supported structures
10550 Postal specialties	13020 Integrated assemblies
10600 Partitions	13030 Audiometric rooms
10650 Scales	13040 Clean rooms
10670 Storage shelving	13050 Hyperbaric rooms
10700 Exterior sun control devices	13060 Insulated rooms
10750 Telephone enclosures	13070 Integrated ceilings

TABLE 1.37 Construction Specification Outline (*continued*)

Division 13 Special construction (<i>cont.</i>)	14300 Hoists and cranes
13080 Sound, vibration, and seismic control	14400 Lifts
13090 Radiation protection	14500 Material handling systems
13100 Nuclear reactors	14600 Turntables
13110 Observatories	14700 Moving stairs and walks
13120 Pre-engineered structures	14800 Powered scaffolding
13130 Special purpose rooms and buildings	14900 Transportation systems
13140 Vaults	
13150 Pools	Division 15 Mechanical
13160 Ice rinks	15050 Basic materials and methods
13170 Kennels and animal shelters	15200 Noise, vibration, and seismic control
13200 Seismographic instrumentation	15250 Insulation
13210 Stress recording instrumentation	15300 Special piping systems
13220 Solar and wind instrumentation	15400 Plumbing systems
13410 Liquid and gas storage tanks	15450 Plumbing fixtures and trim
13510 Restoration of underground pipelines	15500 Fire protection
13520 Filter underdrains and media	15600 Power or heat generation
13530 Digestion tank covers and appurtenances	15650 Refrigeration
13540 Oxygenation systems	15700 Liquid heat transfer
13550 Thermal sludge conditioning systems	15800 Air distribution
13560 Site constructed incinerators	15900 Controls and instrumentation
13600 Utility control systems	
13700 Industrial and process control systems	Division 16 Electrical
13800 Oil and gas refining installations and control systems	16050 Basic materials and methods
13900 Transportation instrumentation	16200 Power generation
13940 Building automation systems	16300 Power transmission
13970 Fire suppression and supervisory systems	16400 Service and distribution
13980 Solar energy systems	16500 Lighting
13990 Wind energy systems	16600 Special systems
Division 14 Conveying systems	16700 Communications
14100 Dumbwaiters	16850 Heating and cooling
14200 Elevators	16900 Controls and instrumentation

groundwater resources, and prevent environmental damage. The three reliability classifications designated by EPA are:

1. Class I—Treatment facilities that discharge into navigable waters that could be permanently or unacceptably damaged by effluent that is degraded in quality for only a few hours (e.g., discharges near drinking water sources, into shellfish waters, or in close proximity to areas used for contact sports)
2. Class II—Treatment facilities that discharge into navigable waters that would not be permanently or unacceptably damaged by short-term effluent quality degradation but could be damaged by continued (several days) effluent quality degradation (e.g., discharges into recreational waters)
3. Class III—Treatment facilities not otherwise designated as Class I or II

Reliability requirements are defined for the three major systems within a wastewater treatment facility: the wastewater treatment system; the sludge-handling and disposal system; and the electrical power system. Reliability requirements for each of these three systems are shown in Tables 1.38, 1.39, and 1.40, respectively. (30, 31)

TABLE 1.38 Wastewater Treatment System Reliability

Features Common to Class I, II, and III

Trash removal or comminution

Grit removal—not applicable to treatment works that do not pump or dewater sludge (e.g., stabilization ponds)

Provisions for removal of settled solids—applicable to channels, pump wells, and piping prior to degritting or primary sedimentation

Holding basin—applicable to Class I with adequate capacity for all flows

Unit operation bypass—not applicable where two or more units are provided and operating unit can handle peak flow; applicable to comminution regardless of number of units

Component Backup Features	Class I	Class II	Class III
Backup bar screen for mechanically cleaned bar screen or comminutor	Yes	Yes	Yes
Backup pump	Yes ^a	Yes ^a	Yes ^a
Primary sedimentation basins	Multiple basins ^b	Multiple basins ^b	Minimum, two ^b
Trickling filters	Multiple filters ^c	Multiple filters ^b	No backup
Aeration basin	Minimum of two of equal volume	Minimum of two of equal volume	Single basin permissible
Blowers or mechanical aerators	Multiple units ^d	Multiple units ^d	Minimum, two ^d
Air diffusers	Multiple sections ^e	Multiple sections ^e	Multiple sections ^e
Final sedimentation basins	Multiple basins ^c	Multiple basins ^b	Minimum two ^b
Chemical flash mixer	Minimum of two or backup ^f	No backup	No backup
Chemical sedimentation basins	Multiple basins ^c	No backup	No backup
Filters and activated carbon columns	Multiple units ^c	No backup	No backup
Flocculation basins	Minimum, two	No backup	No backup
Disinfectant contact basins	Multiple basins ^b	Multiple basins ^b	Multiple basins ^b

^aSufficient capacity of remaining pump to handle peak flow with one pump out of service.^bWith largest unit out of service, remaining units have capacity for at least 50% design flow.^cWith largest unit out of service, remaining units have capacity for at least 75% design flow.^dWith largest unit out of service, remaining units able to maintain design oxygen transfer; backup unit may be uninstalled.^eWith largest section out of service, oxygen transfer capability not measurably impaired.^fIf only one basin, backup system provided with at least two mixing devices (one may be uninstalled).**Value Engineering**

The objective of value engineering (VE) is to make a structured review of a project by a carefully selected team of engineers not previously involved with the project in order to identify potential savings in capital and/or operating costs. Studies conducted by the U.S. Naval Facilities Engineering Command, U.S. General Services Administration, and EPA have concluded that proper use of the VE concept can save an average of at least 10% of the construction cost of major projects.

In VE, the objective is to improve the relationship of worth to cost through the study of functions. Worth is the amount that is to be paid for a function, which is a purpose for which an item is identified. With this premise, the VE work plan systematically uses eight phases for analysis, as described below.

1. Information Phase. During the information phase, general data for use in the decision-making process is assembled. These data include basic cost information as well as specific design conditions and criteria. Also included are design history, background, and concepts. More detailed information may be available, such as completed engineering plans and specifications for a construction project.

TABLE 1.39 Sludge-Handling and Disposal System Reliability

Features Common to Class I, II, III

- Alternate methods of sludge disposal and/or treatment—applicable to unit operations without backup capability.
- Provisions for preventing contamination of treated groundwater.

Component Backup Features Common to Class I, II, and III

- Sludge holding tanks—permissible as alternative to backup capability with adequate capacity for estimated time of repair
- Backup pump—sufficient capacity of remaining pumps to handle peak flow with one pump out of service; backup pump may be uninstalled
- Anaerobic sludge digestion
- Digestion tanks—at least two digestion tanks
- Sludge mixing equipment—backup equipment or flexibility of system such that with one piece of equipment out of service, total mixing capability is not lost; backup equipment may be uninstalled
- Air diffusers—with largest selection out of service, oxygen transfer capability is not measurably impaired
- Vacuum filter—multiple filters with capacity to dewater design sludge flow with largest-capacity filter out of service; each filter is serviced by two vacuum pumps and two filtrate pumps
- Centrifuges—multiple centrifuges with capacity to dewater design sludge flow with largest-capacity centrifuge out of service
- Incinerators—backup not required; backup required for critical auxiliary components (e.g., center-shaft cooling fan)

TABLE 1.40 Electric Power System Reliability

Features Common to Class I, II, and III

- Power sources—two separate and independent electric power sources from either two separate utility substations or one substation and one standby generator

Capacity of backup power source	Class I	Class II ^a	Class III ^a
Mechanical bar screen or comminutors	Yes	Yes	Yes
Main pumps	Yes	Yes	Yes
Degritting	Optional	No	No
Primary sedimentation	Yes	Yes	Yes
Secondary treatment	Yes	Optional	No
Final sedimentation	Yes	Optional	No
Advanced waste treatment	Optional	Optional	No
Disinfection	Yes	Yes	Yes
Sludge handling and treatment	Optional	No	No
Critical lighting and ventilation	Yes	Yes	Yes

^aAt least treatment equivalent to sedimentation and disinfection.

2. Functional Phase. The functional phase evaluates each component on the basis of a verb and noun relationship. For example, the relationship for a pipe is to “convey fluid.” The components are then evaluated as primary or secondary needs while assessing their worth versus anticipated cost. For example, the hydraulic system of a wastewater treatment plant may include slide gates. In VE, their function, which is to control flow, may be considered a secondary need and as such will have a very low worth.

3. Creative Phase. In the creative phase, creative thinking is used to generate alternative means to satisfy the function. During this phase, it is important not to evaluate ideas but to identify them for refinement dur-

ing later phases. It is in the creative phase that a series of ideas is used to encourage the identification of alternates that might normally be subconsciously avoided during the judgment phase.

4. Judgment Phase. The merits of each alternative are evaluated during the judgment phase. It is during this phase that potential advantages and disadvantages are enumerated and a ranking of ideas is made. The resulting primary ideas are evaluated further through a judgment matrix. Weighted constraints are used to make a relative evaluation of the alternatives. Some of the more common constraints considered are life-cycle cost, aesthetics, safety, as well as many that are site-specific, such as expandability or environmental impact. A typical format used during this phase is shown in Fig. 1.17.

5. Development Phase. From the judgment phase, ideas are selected and prepared for more in-depth review during the development phase. It is during this phase that preliminary cost estimates are made and a cost-effective analysis is performed for the alternatives. Also, the use of alternative analyses for potential project savings is first identified in the development phase. The final step in the development phase is to use the cost-effectiveness information to update the judgment matrix prior to final selection of an alternative.

6. Presentation Phase. During the presentation phase, the chosen alternative is presented to show its capability to meet the functional requirements and its cost benefits over the necessary service life. The changes are identified for cost savings in terms of dollars and of percent of overall project cost.

(1) Weight of significance of each constraint on a scale from one to a high of ten. See Note 1		(2) Constraints See Note 1										Brief description of alternate					
		Initial Cost	Operating Cost	Maintenance Cost	Reliability	Performance	Service life	Probability of implementation	Aesthetics	*	*		*	*	Redesign cost	Weighted total	Final rank
(3) List original design and your alternatives	Original design																
	Alternate #1																
	Alternate #2																
	Alternate #3																

(4) Performance Ranking
 9-10---Excellent 3-5---Fair
 6-8---Good 1-2---Poor

Team _____ Date _____
 Project _____
 Item _____

Notes:
 1. Use your judgment to assign weight of significance (block 1) to each constraint (block 2)
 2. When comparing original design and alternates, assign performance ranking (see 4) to each constraint.

*Other potential constraints; effect on construction schedule; fire protection; safety; implementation cost; environmental considerations.

FIG. 1.17 Typical format used during the judgment phase of value engineering. (Reproduced Courtesy of Engineering-Science, Inc.)

7. Implementation Phase. The implementation phase is that portion of work in which the recommended alternative is incorporated into the project. In a construction project, this may require a change in engineering plans and/or specifications prepared for the project.

8. Follow-up Phase. Finally, the follow-up phase is necessary to report the actual cost savings of the proposed alternative or change. Also, it is during this phase that the change is evaluated to determine if it performed as required.

To date, the application of VE has a demonstrated savings potential in the construction industry. Moreover, cost savings do not have to be the basic parameter of study. Saving time, improving aesthetics, eliminating critical materials, and other functions can be made the objective of a VE study.

One or more of the VE principles is used in every decision process. For example, a design engineer will evaluate cost, performance, and reliability data on pieces of equipment. The final selection will also be influenced by consultation with others. Thus, VE principles are often best applied by strengthening ongoing decision-making systems rather than solely as a postdesign process to seek cost reductions.

CONSTRUCTION

Following the completion of the project design, the acceptance of that design by the client, and the decision by the appropriate authorities to advertise for bids, the project can be said to have moved to the construction phase. During the construction phase, an environmental engineering firm may be involved in any one or more of several different levels of responsibility for the administration of the contracting procedures. If the engineering firm has an involvement during construction, the level of that involvement, in order of increasing responsibility and authority, would be one of the following:

1. Contract administration
2. Resident representation
3. Resident engineering
4. Construction management
5. Turnkey: design–construction

In general, each of the various levels of involvement contains the elements of the level or levels above it. That is to say, it is unlikely that an engineering firm would have effective resident representation without contract administration, and in fact, experience indicates such a situation would likely result in difficulty and should be avoided unless overriding outside considerations prevail. The activities, responsibilities, and the authority at each level are summarized in Table 1.41 and described below.

Contract Administration

Contract administration responsibilities do not involve the engineering firm in the contract between the client and contractor. The engineer's role is to provide assistance to the client within areas of its expertise so that the client may take such action as is required to advance the construction.

Resident Representation

Resident representation, like contract administration, does not involve the engineer in any way in the contract between the client and the contractor. The thrust of the engineer's effort must always be to provide technical and administrative assistance to the client in his or her efforts to manage the construction.

TABLE 1.41 Responsibilities for Construction Activities^a

Project activities	Level of responsibility				
	Contract administration	Resident representation	Resident engineering	Construction management	Turnkey design–construction
Engineering/design phase					
1. Detailed design, drawings, and specifications	O	O	O	O	X
2. Contract document preparation	O	O	O	X	X
3. Subcontract packaging	O	O	O	X	X
4. Construction cost estimation	O	O	O	X	X
5. Construction scheduling	O	O	O	X	X
Preconstruction phase					
1. Vendor and subcontractor prequalification	O	O	O	X	X
2. Bid solicitation and analysis	O	O	O	X	X
3. Conduct prebid and preconstruction conferences	O	O	O	X	X
4. Detailed construction planning	O	O	X	X	X
Construction phase					
1. Submittal of shop drawings, catalog cuts, laboratory reports, test data, and erection procedures to verify conformance with contract drawings and specifications	O	O	O	O	X
2. Construction observation and documentation, i.e., daily log, of on-site construction activities		X	X	X	X
3. Expediting of equipment deliveries	O	O	O	X	X
4. Subcontractor supervision	X	O	O	X	X
5. Resolution of field problems encountered during construction	O	O	O	X	X
6. Status reporting to owner	x	X	X	X	X
7. Progress payment requests		O	O	X	X
8. Verification of conformance with contract documents		O	X	X	X
9. Change order preparation	O	O	X	X	X
10. Technical decision making	O	O	O	O	X
11. Community relations		O	O	X	X
12. Conduct project meetings		O	O	X	X
13. Safety assurance	O	O	O	O	X
14. Cost and schedule control	O	O	O	X	X
15. Interface with design engineering staff		X	X	X	X
16. Equipment installation, start-up, and testing	O	O	O	X	X

TABLE 1.41 Responsibilities for Construction Activities^a (continued)

Project activities	Level of responsibility				
	Contract administration	Resident representation	Resident engineering	Construction management	Turnkey design–construction
	Postconstruction phase				
1. Final inspection of completed facilities	O	O	X	X	X
2. Demonstration of adequate process performance		O	O	O	X
3. Preparation of record drawings	O	O	O	X	X
4. Development of operation and maintenance manuals	O	O	O	X	X
5. Enforcement of equipment warranties		O	O	O	X

^aX = Primary responsibility; O = Assistance or review responsibility.

The size and complexity of the project, as well as the agreed duties to be performed under resident representation, will dictate the personnel to be assigned to these responsibilities. Generally, there will be at least one full-time person assigned to the project. In the event of extensive administrative duties, additional on-site personnel may be required. From time to time, it is likely that engineers with specific specialties will be required to adequately perform the duties required under resident representation. The contract with the client should provide for these engineers on an as-needed basis and in addition to the full-time representation.

Resident Engineering

It is possible for an engineering firm to become involved in several different levels of responsibility within the general classification of resident engineering. The differences may be quite significant and could require very careful definition in the contract between the engineer and the client. It is also desirable that the responsibilities of the engineer be clearly delineated in the construction specifications, so that the contractor will know the source of instructions or assistance that may be required.

In contrast to resident representation and contract administration, it is likely that, with resident engineering responsibilities, personnel from the engineering firm may become the limited agents of the client. This authority is limited to the responsibilities assumed by the engineer under the contract with the client and will relate to the client's participation in the administration of the project. If the client has personnel who have responsibility for facilities maintenance or construction, or if the client chooses to employ an inspection and testing firm, the responsibilities of resident engineering may be divided. Otherwise, the authority and responsibility of the engineering firm will be quite extensive and the compensation should be commensurate.

The individual serving as the resident engineer should possess several particular qualifications. The resident engineer should:

1. Be a registered engineer for the geographical area of the project, and additionally might possess other certifications appropriate to the project
2. Be a proven capable administrator able to manage his or her staff successfully and to maintain the respect of the contractor and the client

3. Possess the technical competence required by the complexity of the project
4. Be knowledgeable or able to quickly become knowledgeable of the engineering firm's organization and the assistance available to him or her from within the firm and its connections
5. Possess a personality and bearing that will engender a sense of respect and confidence on the part of the people with whom he or she must deal

The balance of the personnel assigned to the project would be subordinate to the resident engineer and would act under his or her direction and control. These individuals would of course vary considerably in numbers, dependent upon the size of the project, and might approximate the following classifications and functions.

1. Project engineer

- Serve as acting resident engineer in the absence of the resident engineer
- Coordinate and check all engineering and surveying on the project
- Maintain the project set of record drawings
- Maintain the files of shop drawings and catalog submittals and ascertain that required approvals are in hand prior to the start of work on items requiring submittals
- Maintain a current record of job status by items of work
- Review the contractor's requests for payment and advise the resident engineer as to correctness prior to approval
- Perform such other duties as may be assigned by the resident engineer

2. Resident inspector

- Maintain complete familiarity with the contract requirements pertinent to the type of work to which the inspector is assigned
- Maintain complete knowledge of the contractor's schedule of operations as it affects the individual inspector in order that he or she may be at the site of work in progress whenever appropriate
- Complete all required testing procedures in a timely manner and with as little interference to the work as possible
- Report to the resident engineer any deviations from the contract documents as soon as they are observed
- Maintain a professional attitude and manner in dealing with contractor personnel in order to foster a spirit of cooperation toward a common effort to secure a high standard of quality in construction

3. Project clerk

- Assist the resident engineer in the administrative responsibilities of the project
- Maintain a complete set of files and logs of all paperwork relative to the project
- Maintain the posting of all required notices on the project
- Be available on the project at all times during working hours to receive telephone messages and correspondence.
- Foster good public relations by means of a good attitude, manner, and level of competence with all individuals coming in contact with the office of the resident engineer
- Assist the resident engineer by maintaining a complete file and log of all material delivery slips for all items delivered to the project by type and data. Insofar as possible, maintain an inventory of delivered materials not yet incorporated in the work in order to aid in the preparation of estimates. As material is incorporated, indicate in the record the location of its use

Construction Management

Unlike contract administration, resident representation, or resident engineering, construction management requires the assumption of direct responsibility for construction as well as for engineering activities. Construction management services can be provided by either the engineering firm that designed the facility or

by an independent construction management firm. In either case, the organization providing construction management services must have personnel with expertise in construction techniques, procurement methods, and cost and schedule control, as well as design engineering.

There are three commonly used methods for contracting between the owner and the construction manager. These are listed below in increasing order of responsibility for the construction manager.

1. Equipment purchase orders and construction contracts are between the owner and the equipment vendor or contractor performing the work. The construction manager advertises for bids, evaluates the bids, and prepares all purchase requisitions and contracts for the owner's signature. The construction manager supervises the activities of the vendors and contractors; the owner makes payments directly to the vendors and contractors.
2. Equipment purchase orders and construction contracts are issued by the owner as above, but the construction manager pays for the services by drawing from a letter of credit established by the owner.
3. The construction manager issues purchase orders and construction contracts directly. As above, payment is made by the construction manager drawing from the owner's letter of credit.

The construction manager's compensation generally consists of agreed upon hourly rates for time spent on the project, plus expenses, plus a percentage of the construction and equipment contracts that are being managed. The percentage is a function of which of the above levels of responsibility is assumed by the construction manager.

An organization chart for construction management of a typical wastewater treatment plant is shown in Fig. 1.18. The resident engineering manager is in charge of the construction activities and reports to the pro-

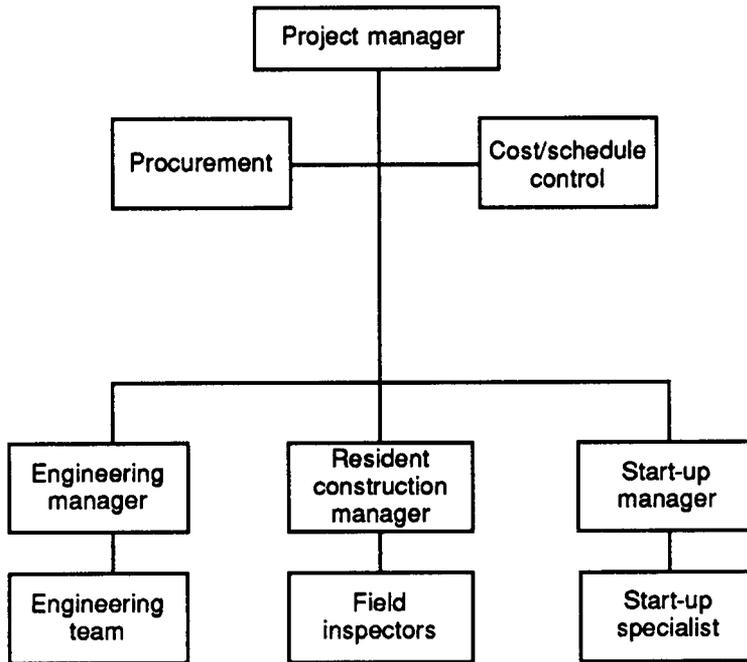


FIG. 1.18 Typical construction management organization.

ject manager. He or she also communicates with the engineering manager and start-up manager in order to assure smooth transitions from phase to phase.

Turnkey: Design–Construction

In all of the construction services described thus far, the organization performing the service is assisting the owner or acting as the owner's agent. A turnkey—design and construction—contract requires a firm to maintain not only responsibility but also ownership until the facility has been constructed, has been started up, and has demonstrated compliance with the performance specifications. Most turnkey contracts also specify a fixed price, which is established at the beginning of the detailed design phase.

Turnkey approaches for wastewater treatment facilities are common in Europe and Asia, where large equipment manufacturers supply most of the equipment themselves, buy what they do not manufacture, and construct the facilities with their own or subcontracted forces. These manufacturers also perform the detailed design based upon a preliminary design and performance specifications prepared by the owner.

In the United States, some of the large oil companies use the turnkey approach for a variety of refinery facilities, including waste treatment facilities. Other examples of common turnkey projects include such things as incinerators, SO₂ scrubbers, and other equipment supplied as a “package,” including detailed design, fabrication, installation supervision, and start-up.

START-UP AND TRAINING

Start-up of new or modified facilities and training of operations personnel to operate the processes is the last critical step in the implementation of any pollution control technology. Start-up procedures and the training of new or experienced personnel must be very carefully planned in order to meet start-up schedules in a safe and effective manner. The essential elements necessary to establish an ongoing successful operation are described below.

Equipment Checkout

The start-up team generally becomes involved in a project when construction is nearly completed and equipment checkout is required. In many cases, the construction contractor will have responsibility for assuring the operability of all mechanical equipment, electric motors, and instruments, and the integrity of all piping, ductwork, and tankage. However, this normally does not include the responsibility to put the process into operation as a system.

Therefore, the first activity of the start-up team is to review the equipment checkout activities, including instrumentation, which is generally the last to be installed and checked, with the construction contractor in preparation for actual process start-up. The start-up team usually consists of process engineers, lead operators, operations specialists in start-up and training, and supervisory and maintenance personnel for mechanical, electrical, and instrumentation systems, as required.

Once the contractor and start-up team agree that the process is ready to run, the start-up supervisor prepares a detailed plan for putting the process into operation. Some of this plan can be prepared prior to the on-site start-up, but it must include detailed instructions concerning set points, valve settings, sampling, analyses, expected performance, and potential signs of trouble. An example of such a start-up procedure is shown in Table 1.42. One other important item that should be taken care of at this point is the ordering of any special tools, chemicals, or equipment that may be needed during the start-up. This will avoid embarrassing delays if key items are difficult or time-consuming to obtain.

TABLE 1.42 Excerpt from a Start-up Procedure for a Sludge Belt Filter Press

Step sequence	Information and details
1. Set HAND-AUTO switch to HAND	Located on control panel.
2. Lock the belt tracking switches ON	Located on control panel
3. Lock the tracking alarm switch to ALARM-SHUTDOWN	Located on control panel
4. Prepare conditioning solution	Prepare proper dilution of sludge conditioning agent
5. Switch components ON in the following order:	Located on control panel
a. Drive	
b. Spray water	
c. Filtrate pump	
d. Fan	
e. Conveyors	
f. Polymer pump	
g. Sludge pump	
6. Adjust pressure roller	See Section 3.10.1 of manufacturer's instructions
7. Observe belt steering under operating conditions	See Section 3.10.1 of manufacturer's instructions if adjustment is required

The equipment checkout activity itself involves operating the entire process as a unit on an intermittent or trial basis. For example, clean water might be used for checkout of a wastewater treatment system. This period should also be used to calibrate all instruments and metering equipment. Calibration curves should be developed where appropriate, and on-line testing of all monitoring equipment should be conducted.

The time required for this activity varies widely according to the complexity of the process, the competence of the construction contractor, and the adequacy of the design. However, a typical range of periods required for most pollution control systems is three days to two weeks. Continuous or long-term operation of the facilities on the material to be treated (gas, liquid, or solid) should not be initiated until the operations supervisor indicates that the process as a whole is safe to put in operation.

Process Start-up

Process start-up on the actual material to be treated is initiated upon instructions from the start-up supervisor according to the detailed start-up plan. This period generally demands considerable time and extra effort by the start-up team, including round-the-clock coverage for critical aspects of some continuous processes. Experience and skill are also needed to make “on the run” decisions.

Key items that enhance the prospects for a successful start-up include a good start-up plan, well-briefed operators, availability of skilled maintenance personnel (particularly for instrumentation and mechanical equipment), and a detailed knowledge of the process in order to solve nonequipment, i.e., process, problems that arise. Process start-up generally proceeds rather rapidly once everything is ready. Most pollution control processes can be started up in one to two days if no major failures occur. However, achieving steady-state operation often requires considerably more time.

Steady-State Operation

Establishing the steady-state operation of a process where the operators are truly in command of the system is the goal of all start-up activities. The time required to achieve that goal can vary widely, ranging from a

few hours for a process with a short turnaround time to several months for processes with long time constants such as biological wastewater treatment systems. In fact, “steady-state” is somewhat of a misnomer when applied to pollution control systems because they generally operate with minimal control over the influent to the process. Therefore, transition from one state or operating point to another is the rule, and a steady state can be said to be achieved when the operators can make these transitions without a process upset. It is the responsibility of the start-up team to explore various operating strategies and control points to identify successful process settings that can accommodate the normal variability in the input.

Achieving steady-state operation is to a large extent dependent on keeping and carefully reviewing operating log sheets and performance data. One of the tasks of the start-up team during this period is to establish the sampling and analytical requirements necessary to control the process. The location, frequency, and type of samples required should be specified, and the types of analyses required on each sample must be noted.

A sample log sheet, including typical process operational parameters and sampling and analytical requirements for a wastewater treatment process, is shown in Table 1.43. In terms of establishing steady-state operation, this information can be used to evaluate the process, determine if it is performing as intended under various input conditions, and identify successful and unsuccessful operating conditions that have been employed.

Operator Training

Operator training can take several different forms and occurs at various times, including prior to start-up, during start-up, and following the establishment of steady-state operation. The type of training conducted is highly dependent on the background and experience of the operating staff to be trained. The range of capabilities and experience may be very broad, ranging from new-hires or trainees to highly experienced supervisors who need to review a new process or type of equipment. For this reason, every training program must be tailored to the needs of the process and the individuals who will be in the training sessions. However, there are some common areas that are generally covered in training courses for pollution control systems.

A typical course outline covering a broad range of topics and objectives is shown in Table 1.44. For any given training course, only portions of this curriculum might be used depending on the class needs.

Key elements that should be incorporated into a training program to make it successful are as follows:

- Pretest to identify students' needs and choose the areas for emphasis in the course.
- Operators (not engineers or scientists) should train the operators.
- Make a few key points; be sure they are understood; do not present too much.
- Use example problems freely; develop them from the actual facility data if possible.
- Combine classroom and on-the-job training to broaden trainees' experience.
- Posttest to evaluate the training, the instructor, and to determine how much progress was made.

Recent surveys have shown that one of the most widespread, significant deficiencies of operators in the pollution control field is the lack of basic process understanding. Therefore, the training should include what a given unit process is intended to do, how it does it, and how to control it when it is in operation. Far fewer deficiencies normally exist in knowledge of mechanical equipment and maintenance than in process control and operating strategies.

Operation and Maintenance Manual

An operations and maintenance (O&M) manual should be a well-worn reference document used by the operators for a variety of day-to-day operations needs. Detailed, valve-by-valve reference manuals cover every aspect of the system. All are useful documents if they meet their objectives, but one must be careful not to

TABLE 1.43 Sample Operating Data Log

	1	2	3		4		5		6					
	Flow,	pH,	pH	Alum	TSS,	Poly	Recycle	Rise	Anti-	H ₃ PO ₄ ,	TSS,	TOC,	COD,	BOD ₅ ,
Date	mgd	units	chart,	rate,	mg/L	rate,	rate,	rate,	foam,	%	mg/L	mg/L	mg/L	mg/L
			units	%		%	gpm	s	%	%				
1														
2														
3														
4														
5														
6														
7														
9														
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24														
25														
26														
27														
28														
29														
30														
31														

Remarks _____

(continues)

Date	10						Discharge			
	TSS, mg/L	TOC, mg/L	COD, mg/L	BOD ₅ , mg/L	pH, units	Phenol, mg/L	Flow, mgd	Temp, °C	Cl ₂ , mg/L	pH, units
1										
2										
3										
4										
5										
6										
7										
9										
10										
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Remarks _____

TABLE 1.44 Sample Training Program Curriculum for a Wastewater Facility

1. Introduction and pretest	
2. Chemistry and biology	Specific to waste treatment operations
3. Wastewater characteristics	Solids, COD, BOD, dissolved oxygen, temperature, pH
4. Wastewater terminology	Definition of key vocabulary items
5. Wastewater calculations	Basic mathematics for treatment plant operators
6. Pretreatment processes	Theory, instrumentation, operating procedures, hands-on operation of equipment
7. Chemical addition and precipitation	pH control, use of polymers, dosages, calculations, operating procedures; hands-on operation of equipment
8. Principles of clarification	Theory and practical application, calculations; hands-on operation of equipment
9. Equalization	Theory and practical application
10. Activated sludge	Theory and biology of activated sludge, control; calculations, field examination of process
11. Final clarification	Theory and principles of control, operation of related equipment, hands-on operation
12. Sludge digestion	Theory and practical application, control calculations, hands-on operation of equipment
13. Sludge removal	Theory of operation of sludge lagoons and drying beds
14. Operation of mechanical equipment	Troubleshooting of equipment, pumps, agitators, instruments, valves, and motors, principles of lubrication
15. Sampling and analytical techniques	Proper sampling techniques, demonstration of jar test, pH test, DO measurement, calibration of all lab instrumentation
16. Data	Collection, interpretation, and use of data, problems in diagnosing plant conditions through use of data
17. Summary and posttest	Review of any areas where questions exist, reexamination

prepare the wrong type of manual for a particular application. Supervisors, operators, and maintenance personnel need different types of information, and it is not always productive to package it all in a single document.

A comprehensive outline for an O&M manual for a wastewater treatment facility is shown in Table 1.45. This includes all of the necessary elements for most O&M manuals and can be used as a reference to choose sections that might be suitable for a particular application.

A critical aspect of the utility of any O&M manual is that it be written for operators, not engineers, in operators' language, preferably by an experienced operator. This will make the document much more useful in the field and more widely accepted by the operators. Also, graphical presentation of information is often more valuable and useful than narrative or tabular information. Finally, it is generally a good idea to bind an O&M manual in a loose-leaf binder to facilitate updating of the manual, thus making it more of a working document and easier to copy for use in the field.

Periodic Operations Reviews

Once a process is in operation and achieving good steady-state performance, one of the most useful operations assistance techniques is to conduct a monthly (or other selected time period) review of the performance of the facility. This review should be done by someone not involved in the day-to-day operation or supervision of that facility and should include a discussion of any problems (process or equipment) encountered, solutions to those problems, review of operating data, adjustment of operating strategy, and identification of any system design modifications that may be required.

TABLE 1.45 Sample O&M Manual Table of Contents

Chapter 1	Introduction
Chapter 2	Description of facilities
Chapter 3	Theory of operation and process control
Chapter 4	Standard operating procedures
Chapter 5	Sampling and analytical requirements
Chapter 6	Maintenance
Chapter 7	Safety
Appendices	Design drawings
	Vendor information
	Reference materials
	Chemical data information

These reviews are an excellent way to short circuit any minor nuisances before they become major problems. The plant supervisor is afforded an opportunity to discuss strategies, performance, new technology, and other operations considerations on a fairly routine basis. This practice often results in improved overall operation and successful long-term performance.

REGULATION OF WETLANDS

The principal federal program for regulating activities that affect wetland areas is provided by Section 404 of the Clean Water Act (CWA) (32). Section 404 of the CWA was added in 1972 when Congress made its first amendments to the Act (33). Under the Section 404 program, the U.S. Army Corps of Engineers (Army Corps) and U.S. Environmental Protection Agency (EPA) share jurisdictional authority over the dredging and filling of waters of the United States, including wetlands. Although the Clean Water Act is essentially silent on which agency has authority to make jurisdictional determinations under the Section 404 Program, the EPA and Army Corps have formulated agreements detailing their respective jurisdictional responsibilities (34). The Army Corps has primary responsibility for administering the Section 404 program, with guidance from the EPA.

Scope of Section 404 Program

On its face, the express language of Section 404 of the CWA is very limited to requiring permits for the “discharge of dredged or fill material” into “navigable waters” (35). However, this “navigable waters” language of Section 404 has been interpreted broadly to mean all “waters of the United States, (36) which may include wetlands (37). The Army Corps’ regulations define “waters of the United States” to mean “[a]ll waters which are currently used, or were used in the past, or may be susceptible to use in interstate or foreign commerce, including all waters which are subject to the ebb and flow of the tide . . .” (38). This definition also includes “[a]ll other waters such as intrastate lakes, rivers, streams (including intermittent streams), mudflats, sandflats, wetlands, sloughs, prairie potholes, wet meadows, playa lakes, or natural ponds, the use, degradation or destruction of which could affect interstate or foreign commerce . . .” (39). The regulations also encompass wetlands “adjacent” to waters associated with interstate commerce, and have been interpreted to include jurisdiction over certain “isolated wetlands” (40).

Army Corps’ regulations define dredged material as “material that is excavated or dredged from waters of the United States (41). Fill material is defined as “any material used for the primary purpose of replacing an aquatic area with dry land or of changing the bottom elevation of a waterbody (42). Although the range of

activities involving dredged and fill material is broad, the Army Corps' regulatory powers do not extend to all activities undertaken in wetland areas. For example, the draining of wetlands, which is a major source of wetland losses, is not expressly regulated or prohibited by Section 404 (43). In addition, when Congress amended the CWA again in 1977, it chose to exempt several activities from Section 404 requirements, including certain farming, silvicultural, and ranching activities (44).

Furthermore, for the first ten or so years of the Section 404 program, the EPA, the Army Corps, and the courts took the position that Section 404 only regulated physical discharges of dredged or fill material into navigable waters (35). As such, many activities that could destroy wetlands were not regulated under the Section 404 program. This narrow "discharge rule" was, however, later eroded by regulatory guidance and judicial decisions. For example, in one case, the Fifth Circuit ruled that certain land-clearing activities, which resulted in redeposits of material taken from the wetland, constituted a discharge of a pollutant and, therefore, required a Section 404 dredge and fill permit (46). Also, in a 1990 Regulatory Guidance Letter (RGL), the Army Corps expressly indicated that all landclearing activities using mechanized equipment are subject to 404 permit requirements (47). Further, in another RGL, the Army Corps stated its intention to regulate projects placed on pilings when the placement of such pilings "is used in a manner essentially equivalent to a discharge of fill material in physical effect or functional use and effect (48).

Section 404 Permit Applications

The Secretary of the Army, acting through the Chief of Engineers, is authorized to issue individual permits for the discharge of dredged or fill material into the waters of the United States, which includes wetlands (49). In some circumstances, the Army Corps may issue "Nationwide Permits" for certain activities in jurisdictional wetlands that are deemed to have minimal environmental impacts (50).

The Army Corps regulations set forth extensive procedures for the permit process (51). Following submission of a permit application (52), the Army Corps must first decide whether the agency has wetland jurisdiction over the proposed project. If the Army Corps has jurisdiction, the next inquiry involves determining whether the activity requires a permit. Then, the Army Corps may determine that the project will not degrade wetlands in a manner that would prohibit issuance of a Section 404 permit. The Army Corps may also issue the permit while requiring project modifications and/or mitigation. An alternative response is the issuance of a general or "Nationwide" permit, or determination that the project meets any of several statutory exemptions to the requirement for a Section 404 permit. Finally, the Army Corps may determine that the project is unacceptable under Section 404, and reject the application.

Section 404(b)(1) Guidelines

When reviewing an application for a Section 404 permit, the Army Corps must consider Section 404(b)(1) Guidelines issued by the EPA (53). Under these Guidelines, the Army Corps cannot issue a permit where "a practicable alternative to the proposed discharge [exists] which would have less adverse impact on the aquatic ecosystem, so long as the alternative does not have other significant adverse environmental consequences" (54). The Guidelines also prohibit the Army Corps from issuing Section 404 permits if the proposed discharge would result in a violation of other environmental laws or regulations or cause or contribute, either individually or collectively, to significant degradation of wetlands or other waters of the United States (55).

Consideration must be given to whether the proposed discharge is the least environmentally damaging "practicable" alternative. An alternative is considered "practicable" if it is available and capable of being performed after taking into consideration cost, existing technology, and logistics in light of overall project goals (56). The Guidelines require that an alternative site be considered "practicable" even if it is not currently owned by the applicant (57). Where the proposed activity is not water-dependent, the Guidelines fur-

ther provide that practicable alternatives not involving wetlands are presumed to exist, unless clearly demonstrated otherwise (58).

The standard for evaluating practicable alternatives has been the subject of considerable strife between permit applicants, the Army Corps, and the EPA. The scope and degree of analysis necessary to fulfill the practicable alternatives requirement is far from clear. As a result, several courts have been called upon to interpret the meaning of the “practicable alternatives” language (59). In addition, the agencies have found it necessary to issue further clarifications on the proper method for analyzing practicable alternatives (60).

Wetland Mitigation Requirements

An important focus of the Section 404(b)(1) Guidelines is a requirement that permit applicants perform some form of “mitigation” to restore or replace wetland values that will be adversely impacted by the proposed activity (61). When reviewing applications for Section 404 dredge and fill permits, the Army Corps evaluates permit applications according to a sequencing of mitigation options. The preferred mitigation option is avoidance of impacts, followed by minimization of impacts, and finally, appropriate and practicable compensation for unavoidable impacts. The EPA and Army Corps further clarified this three-step sequencing methodology for determining appropriate mitigation of project impacts with issuance of a joint Memorandum of Agreement (MOA) (62).

According to the MOA, during the evaluation process of a Section 404 permit application, “[t]he Corps . . . first makes a determination that potential impacts have been avoided to the maximum extent practicable; remaining unavoidable impacts will then be mitigated to the extent appropriate and practicable by requiring steps to minimize impacts and, finally, compensate for aquatic resource values.” The first step, avoidance, is synonymous with the “practicable alternatives” analysis of the Section 404(b)(1) Guidelines. The second step, minimization, requires that permit applicants make changes to project design that will reduce impacts to wetlands. The third step, compensation, requires replacement of wetlands that are degraded or destroyed by unavoidable impacts.

This sequencing methodology applies to all individual permits, regardless of the type or ecological value of the wetlands adversely affected by project impacts. The MOA does, however, provide for some deviations from the sequencing requirement where the requirements have been incorporated in an Army Corps or EPA comprehensive plan, such as a Special Area Management Plan, or where necessary to avoid environmental harm, or where the EPA and Army Corps agree that a proposed discharge “can reasonably be expected to result in environmental gain or insignificant environmental losses.”

CONTINGENCY PLANNING

Natural disasters, such as floods, earthquakes, hurricanes and tornadoes, and mechanical equipment malfunctions and structural failures, such as pipe or dike breaks, can cause accidental spills of harmful or damaging contaminants. Contingency planning develops a course of action for dealing with emergency situations when response time is critical. Of primary importance is minimization of potential adverse effects on public health and safety. Protection of surface and groundwater water quality and preservation of environmentally sensitive areas also are concerns (63–66).

Contingency planning is a systematic method of assessing potential emergencies, developing an organized response to these emergency situations, and providing a working document to guide personnel dealing with the situation. Contingency plans, also known as disaster preparedness plans, can be as simple or as complex as potential situations warrant. Some plans deal with the required response to specific events, such as the rupture of a storage tank in an environmentally sensitive area or a landfill fire. Others may be very complex and cover response to a wide variety of emergency situations.

Common elements of a contingency plan are definition of the emergency situations, delineation of responsibilities, communications plan, emergency response, and preparedness training. Tables, lists decision trees activity flow charts, containment procedures, and other illustrations should be used to quickly and accurately display information and guidelines. After an event or at least annually, the contingency plan should be updated from lessons learned and reviewed for changes in responsibilities, personnel, contact telephone numbers, and other vital information. The contents of a contingency plan for a water and wastewater system is illustrated in Table 1.46.

Emergency Situations

Identifying potential emergency situations and defining response goals are the first steps in contingency planning. An evaluation of the facility's history of emergencies and potential risks is a major part of this task. Typical emergencies requiring a planned response may range from highly visible and devastating natural disasters, such as earthquakes and floods, to fewer visible risks, such as power outages at key pump stations or pipe breaks in critical transmission lines.

In addition to health and environmental protection, goals typically include keeping as much of the facility or system in service as possible and bringing affected portions of the system back in service as soon as possible. Alternative operations plans also are needed. For example, a water utility may require an alternative source of drinking water in the event of an upstream chemical spill or wastewater release.

Responsibilities

Delineation of responsibilities is one of the most critical steps of contingency planning. A clear chain of command must be spelled out. First, the person in charge during the emergency situation should be identified. This is the person who is to be notified in the event of a potential emergency situation and who activates the contingency plan after a problem is verified. Other key personnel are assigned to activate and manage response activities. At least one alternate must be identified for all personnel in key response roles.

The contingency plan should include a list of all response personnel and their home and emergency contact telephone, beeper, and cellular telephone numbers. Also, a list of the personnel's training and response capabilities is desirable. Such a list can serve as a valuable resource in mobilizing the personnel required to deal with a specific emergency situation.

Communications

A communications plan is the decisive tool to diffuse the confusion that surrounds many emergency situations. The communications plan covers all aspects of communication during the situation, both internal and external. A good communications plan will include:

- An internal communications program, which spells out channels of communication so that accurate information about the situation can be obtained
- A public notification program for disseminating information to the local news media
- An agency notification program for notifying regulatory and other agencies
- A public access advisory strategy to alert the public to avoid affected areas or to implement emergency procedures, such as a boil water order

The internal communications plan will specify methods of communication, such as telephones with radio backup and/or digital beepers. This plan may establish a briefing meeting time and location. The plan should

TABLE 1.46 Water and Wastewater System Contingency Plan Contents

Section 1 Introduction	
System Description	Site Liaison Officer
Emergency Incident History	Public Information Officer
System Improvements	Response Action
Contingency Planning	Internal Notification
Plan Implementation	Incident Verification
Section 2 Agency Notification Plan	Emergency Standby and Repair Contract
Notification Criteria	Emergency Repair
Regulatory Regulations	Emergency Permitting
Receiving Water Categories	Appendix—Wastewater Pump Station Data
Notification Protocol	Appendix—Water Reclamation Facility Data
Immediate Notification	Appendix—Water System Booster Pump Station Data
Secondary Notification	Appendix—Water Treatment Facility Data
Agency Contact Information	Section 6 Emergency Flow Control Plan
Appendix—Emergency Actions	Transmission System
Appendix—Agency Notification Contact Information	Reduced Water Supply System Pressure Collection System
Section 3 Public Notification Plan	Flow Diversion
Public Notification	Emergency Water Conservation/Wastewater Flow Control
Media Communications	Government Agencies
Media Interviews	Commercial and Industrial Customers
Prescribed News Releases	General Public
Appendix—Prescribed News Releases	Section 7 Emergency Monitoring Plan
Section 4 Public Access Advisory Plan	Water Quality Monitoring
Advisory Procedures	Sampling Protocol
Posting Procedures	Analysis Protocol
Deployment of Signs	Reporting Procedures
Retrieval of Signs	Section 8 Preparedness Training Program
Section 5 Emergency Operations Plan	Awareness Training
Potential Failure Mode	Implementation Workshop
Recent Investigations	Safety Briefing
Failure Detection	Responsiveness Training
Surveillance Program	
Flow Variation	
Public Report	
Response Administration	
Response Manager	
Operations Manager	
Safety Officer	

consider the establishment of a control center, which may be in existing office space or in mobile units that can be moved to the crisis locations, depending on the type of anticipated emergency and the response needs.

The public notification program should include a list of media contacts along with their telephone and facsimile numbers. Appropriate local officials may be included in this communications plan. The plan must clearly identify who has authority to talk to the media and include guidelines for effective media communi-

cations. Prescribed press releases may be included in the contingency plan, so that only the details about the emergency have to be filled in at the time of release.

Another key aspect of the communications plan is the agency notification program as the reference for personnel assigned responsibility for contacting specific federal, state, and local agencies in the event of an emergency. Guidelines should be included with respect to timing of the contacts and required follow-up, so that federal and state regulations are met.

Emergency Response

The emergency response portion of a contingency plan defines the type of response to be made to a specific crisis. Goals and priorities are established, such as retaining spillage to protect response personnel as well as the public and then focus on repair. The plan should delineate emergency operations to be implemented and define the mobilization and response actions.

Emergency response also includes procedures for emergency environmental monitoring. The plan should define the sampling, analysis, and reporting procedures that should occur during the emergency. Emergency monitoring is the means by which the end of the crisis period is determined, as well as the method for assessing the effects of the crisis situation.

Training

The preparedness training portion of a contingency plan outlines the training required for personnel involved in plan management, oversight, and response. Without proper training, the contingency plan will not be as effective in aiding response personnel in meeting and managing the crisis as it could be. Periodic and unscheduled field exercises should be considered for potential major incidents.

COMPUTER UTILIZATION

The science and technology of computers is a rapidly expanding field. Computer technology is finding increasing utilization in government, business, industry, engineering, and scientific applications. There is hardly an aspect of modern life that is not being significantly affected in some way by advances in computer technology.

The current trend in computer technology advancement is also leading to an increase in computer applications within the environmental engineering field. The day is approaching when virtually all environmental engineers will be interfacing on some level with computers or computer-oriented personnel.

Computer Equipment

There are a number of ways in which the environmental engineer may have access to a computer or computer services. Many companies and institutions own or lease computers, while others make use of contract facilities. There are very few governmental or private institutions that do not have access to computer services of one type or another.

In the past, computers consisted exclusively of large, cumbersome pieces of equipment that required high initial investment, dedicated facilities, special environments, and full-time operating personnel. Though major computer facilities still make use of large, powerful mainframe computers, advancements in the field of

microcircuitry have led to a new breed of computers that are smaller and less costly, yet capable of handling many business, scientific, and engineering applications. It is possible to obtain a desktop computer today with equivalent capabilities to a machine that would have filled a large room only a few years ago.

In the absence of a recognized standard classification system for categorizing computers according to size and ability, the general terminology presented below is often used:

Microcomputer	can be easily moved from place to place
Minicomputer	is fixed in a cabinet
Mainframe	requires special environment (such as false floor, temperature control, etc.)

The CPU (central processing unit) rating of a computer is a measure of the amount of information that is processed at a single time, an indication of computational speed. The RAM (random access memory) is the available space within the computer for storage of operating systems, programs, and data. ROM (read only memory) refers to data storage and retrieval capacity. ROM capacity is generally in the form of storage on media such as tape, floppy disks, hard disk drives, or CDs.

There are many things that must be considered when a company or individual decides to purchase or lease a computer. The type of computer should be suited for the intended applications in data processing, word processing, computer-aided drafting, and communications. Though most computers, with the proper software and peripherals, can perform a wide variety of functions, some are better suited to certain applications than others. Also, the memory size and computational speed should be compatible with the computer's intended short- and long-term use. Additionally, it is important to compare and balance the costs of owning and operating the computer with the economic benefits it will provide in the future.

Programming Logic and Languages

A computer program is, in simple terms, a set of instructions that the computer follows to achieve the desired results. The act of programming a computer is the process of formulating a set of instructions that the computer can understand and, when carried out, will achieve the goals of the user.

Programming Logic. A computer program must be designed so that activities, such as computation, data input, data acquisition, and resultant output, occur in the proper sequence. The pattern behind routing and sequencing of the program activities is referred to as the program's logic. Programmers normally formulate a logic flow diagram as one of the initial steps in program development. The logic diagram serves as a pattern by which the coded program can be written so the computer will follow instructions in the proper sequence to generate the desired results. Logic diagrams are also useful tools for describing a program's structure to persons who are unfamiliar with it.

A logic diagram for a simple program to calculate the pipe diameter necessary to allow a specified flow in a gravity flow situation is shown in Fig. 1.19. The logic diagram is quite simplistic and does not represent the most efficient way to approach this particular problem; however, it does illustrate some important basic concepts of programming logic that are used in almost every program.

Elements 1, 2, 3, 5, 7, and 8 of the logic diagram are referred to as *direct statements*. They give the computer a direct command that, if properly stated, will be carried out.

Elements 4 and 6 of the logic diagram are *conditional statements*. Conditional statements contain a condition, usually an equality or inequality. If the condition is met, program execution takes one route; if not, it takes an alternate route. Conditional statements are key elements of program sequence control, which allows a program to be designed to make internal decisions.

Elements 3, 4, 7, and 6 form a *loop*. In this example, pipe diameter is increased incrementally until a val-

ue is found that is large enough to allow the specified flow rate. One of the major benefits of performing calculations with a computer is that repetitive computations, such as those needed to analyze large amounts of data or perform multiple iterations, can be performed quickly. Looping is a key technique in designing programs for these types of calculations.

The inclusion of element 6 in the 3-4-7 loop is a form of safeguard; a conditional statement that checks the current value for pipe diameter (D) to see if it exceeds a user-specified allowable maximum (DMAX). For example, DMAX might be the largest commercially obtainable size of pipe made of the desired material. Inclusion of element 6 within the loop prevents the calculations from continuing to an unreasonable conclusion if the input data are unrealistic.

Program Languages. A programming language is an artificial language that is readable and understandable by a computer. The rules related to a computer language are referred to as its *syntax*, analogous to the grammar of a spoken language. Syntax pertains to the specifics of statement formulation, word relationships, and punctuation and differs between programming languages.

There are a number of programming languages currently in widespread use and others that are in various stages of implementation. Like spoken languages, most programming languages are continually changing as a result of improvements and expansion. A person who learned a common programming language years ago would find significant changes in its modern form. Some programming languages are also available in different versions (similar to dialects that have arisen in spoken languages).

The following paragraphs contain descriptive information on several of the more significant programming languages in current use.

FORTRAN—Formula Translator. FORTRAN was developed in the 1950s by IBM Corporation and was one of the first high-level programming languages available. It has been a popular language among engineers and scientists and, though not a business-oriented language, has found many applications within the business community. FORTRAN is widely used and understood and is available through most computer installations.

COBOL—Common Business-Oriented Language. COBOL was developed in the late 1950s primarily through the efforts of the U.S. government. COBOL is a business-oriented language and, due to its structure, its applications outside the business community are rare. It is easily transported between different computers and has been regularly updated and standardized. COBOL can be a sophisticated and complex language, and its versatility in business applications has given it widespread use.

BASIC—Beginners All-Purpose Symbolic Instruction Code. BASIC was developed in the early 1960s at Dartmouth College. It is a language that is easily learned and is taught extensively in high schools and universities. BASIC does not have the computational depth of some of the other high-level languages such as FORTRAN, but it is flexible enough to make it usable in many scientific and business applications. BASIC is widely used and understood and is available on most computer systems.

PASCAL. PASCAL was defined by Niklaus Wirth at the Institute for Informatik in Zurich, Switzerland, and its use began when the first compilers become available in the early 1970s. PASCAL is suited to both scientific and business applications. It is an easily readable and highly structured language that is currently increasing in use in learning institutions. PASCAL's structure forces programmers to follow an orderly sequence of steps, which reduces the possibility of errors and can minimize the effort required for program debugging.

APL—A Programming Language. APL was developed by IBM Corporation in the late 1960s and is a language well suited to interactive use. It is a powerful procedure-oriented language that is particularly applicable to scientific programming, though it is also suitable for business and commercial applications. One of APL's key strengths is that it incorporates a number of complex mathematical functions that allow a single command to perform difficult operations that would require a substantial amount of code in many other languages. APL is particularly convenient for performing manipulations of multidimensional arrays of data.

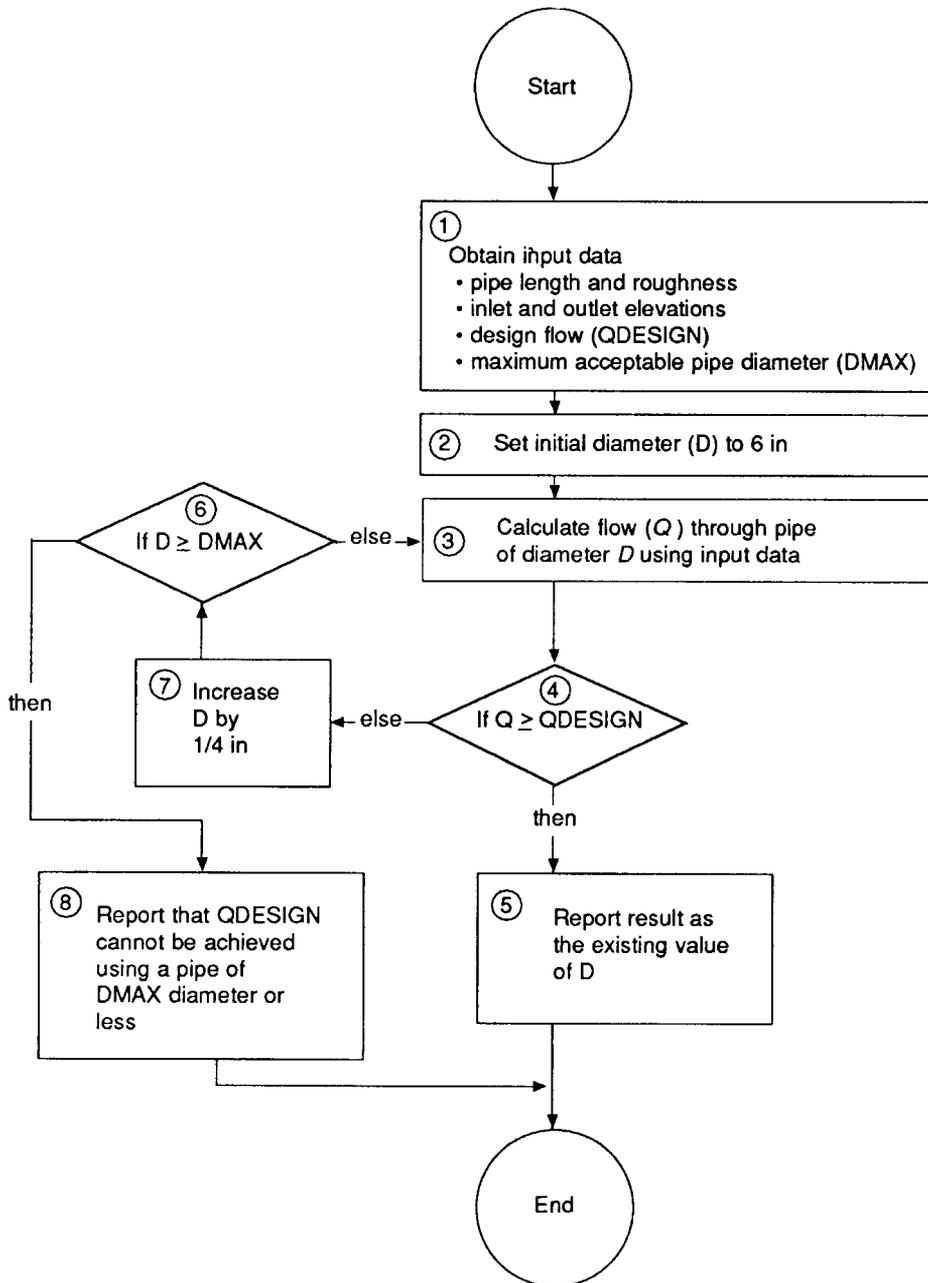


FIG. 1.19 Program logic for pipe diameter calculations.

Unlike other languages such as PASCAL and BASIC, APL uses a cryptic notation that makes programs difficult to follow for persons unfamiliar with the language. In the past, APL implementation had mainly been limited to larger computers, but it is currently becoming available on smaller systems.

PL/1—Programming Language One. PL/1 was developed by IBM Corporation in the mid-1960s for its System/360. It is a powerful language that was designed for both business and scientific applications. PL/1 is more versatile and easier to use than FORTRAN or COBOL because it is a more modular and structured language. At the time PL/1 was introduced, FORTRAN and COBOL were in wide use and meeting the needs of their respective users, so adoption of PL/1 has been relatively slow. However, improvements in computer technology have led to an increasing number of implementations of PL/1.

C. A multifunction general-purpose language developed by Brian Kernigan and Dennis Ritchie at Bell Laboratories in the early 1970s on a Digital Equipment Corporation PDP-11 computer. In its raw state as defined by Kernigan and Ritchie, C provides very few built-in functions; however, current implementations offer rich sets of functions for mathematical, business, statistics, database, graphics, and engineering applications. C is structured such that programs are easily transported from one machine and operating environment to another. As well as the high-level functions provided in current implementations, the language also provides low-level machine support allowing programmers to perform systems programming applications with access to fundamental machine processes (CPU registers, direct memory manipulation, etc.) This range of application is not available from other languages such as BASIC, FORTRAN, COBOL, or PL/1. Because of its flexibility and power, this language has become a mainstay in the business and scientific programming environment.

Computer Program Development

Since there can be significant costs associated with applications program development, they are often written for performing a specific task or solution of a specific problem. When a problem arises or a computational need develops that can most effectively be handled by computer methods, a program is written to perform the task. Though a program may be written for use in a specific task, the foresighted program developer will build flexibility into the computer program so that it can be used in other similar situations by selecting alternate input data.

The general steps in solving problems utilizing computer methods can be expressed as follows:

1. Defining the problem
2. Evaluating the problem to determine if computer methods are applicable to aid in its solution
3. Researching existing programs to determine if a similar program can be used as is or modified relatively easily
4. Determining if using a computer is the most cost-effective method for solution of the problem
5. Defining the calculation procedures and mathematical models to be used within the program
6. Organizing the logic flow for the program so it will perform the desired function
7. Translating the logic flow and calculation procedures into the coded program
8. Testing the program to eliminate errors (referred to as *debugging*)
9. Documenting the program
10. Using the program to solve the problem

The first step, problem definition, is common to the solution of problems whether or not computer methods are considered. Steps 2, 3, and 4 determine if computer methods are to be used. This evaluation is extremely important and should be performed with care and judgment. Program development and debugging can be a lengthy and expensive process, and it is all too easy to embark on writing a program and realize by

the time it is completed and operational that solution by an alternate method would have been quicker, easier, and/or less costly. Key questions to address when evaluating computer methods for a given application would include:

- Does the solution require repetitive calculations on a large quantity of data that can be effectively performed by a computer?
- Does the solution require complex calculations that can be done quickly by computer but would be prohibitive to perform by hand?
- Does the solution involve performing the same calculations over and over using different input data to define an optimal solution?
- Would the effort required to obtain a computer solution be less than using other methods?
- Is it a recurring problem or application, so that writing a program now will save time and effort in similar situations in the future?

Steps 5 through 8 constitute the actual program development process. The testing phase (step 8) can require as much or more effort than planning, designing, and coding the program (steps 5 through 7). Errors within a program can be difficult to find and correct, so extra care taken in program planning and writing is usually worthwhile to minimize the debugging effort.

Step 9 is the documentation process, preparing a written record of key facts concerning the program. Documentation should include at a minimum

- The program's function
- A description of the mathematical model(s) utilized by the program
- Capabilities and limitations of the program
- Key assumptions inherent to the program
- Required input data to operate the program
- Output data generated by the program
- Instructions for accessing, start-up, and use of the program
- Technical description of the program, e.g., language, storage requirements, and hardware specifications

Documentation is important because it provides the information necessary for someone other than the developer to make use of the program.

Computer Applications in Environmental Engineering

Computer applications are becoming increasingly common as new ways are explored to make use of modern computer technology within various areas of the environmental engineering field. Problems that once were solved by hand, consuming substantial amounts of time and resources, can now be solved easier and more economically with the aid of computers. Also, problems that once were too complex to even attempt may now be solved comparatively easily. Environmental engineers are also benefiting from the extensive information management, storage, manipulation, and communication capabilities of today's computers.

This section illustrates some of the major ways computer methods have been applied within environmental engineering.

The availability of programs varies considerably. Code listings of some programs can be obtained at no cost, while others are available through software purchase or as part of timesharing libraries. Other programs are proprietary and can only be utilized through a direct consulting contract with the company that owns them.

The U.S. Department of Commerce National Technical Information Service (NTIS) makes many government programs and related literature available to the general public through their Computer Products Support Group in Springfield, Virginia.

Project Management. Numerous programs have been written to aid in the difficult task of managing budgets, schedules, and resources on large engineering and construction projects. These programs can be utilized on environmental as well as other types of engineering projects. Some of the programs develop project schedules utilizing CPM (critical path method), and/or PERT (Program Evaluation and Review Technique), approaches for minimizing completion time and optimizing resource allocation. Others monitor ongoing project activities and provide regular management reports showing project schedule, percentage completion, and budget status. Some programs provide combinations of both planning and monitoring capabilities.

Air Quality. Computer methods have been used extensively by environmental engineers within the air quality modeling field. Many programs have been written to model air quality impacts due to emission sources based on ambient meteorological conditions. Typical applications include predicting the impact of a proposed emission source, such as a factory or power plant, on local air quality or for establishing the allowable emission rate from an existing source. Programs are available that simulate point sources, e.g., a large smokestack; area sources, e.g., an industrial park with numerous emission sources; and line sources, e.g., a major urban highway.

One reason that computerized air quality modeling has received such extensive use is its recognition by governmental agencies as a viable method for establishing emission limits and assessing environmental impacts. The EPA's Office of Air Quality Planning and Standards has published guidelines on air quality models that identify specific programs that have been endorsed for environmental permit applications. The "guideline models," as well as other state-of-the-art dispersion models, are included in the UNAMAP (user's network for applied modeling of air pollution), system, which is available through NTIS and is supported by EPA's Environmental Sciences Research Laboratory.

Water Supply. A wide variety of programs have been developed to design and/or evaluate water supply distribution networks. The determination of flows and pressures in a community water distribution system requires iterative calculations to balance headlosses between various points of the system. These analyses can be time consuming if done by hand, even for relatively simple networks. As a result, extensive work has been done to develop computer programs in this area. These programs can be used in the design of new water distribution systems and in the evaluation of modifications or additions to existing systems.

Other areas where computers have been used in water supply include cost estimating for distribution systems, prediction of present and future water usage rates, and analyses of water treatment processes.

Storm and Sanitary Sewer Systems. Extensive use has been made of computer-aided storm and sanitary sewer design. Programs have been written that are capable of quickly performing the iterative hydraulic calculations required for design or evaluation of complex combined or separate sewer systems. Using computer methods, it is possible to design a sewer system for normally expected flows and then model stressed conditions, such as larger than normal rainfall events, to determine the potential for overflow and to examine impacts on treatment facilities. Some programs have cost-estimating capabilities and are useful in determining the most cost-effective sewer system designs.

Wastewater Treatment and Disposal. Computerized models have been developed to aid the environmental engineer in a number of aspects of wastewater treatment facility design, evaluation, and plant operation.

Design Applications. Programs have been written based on mathematical models of various wastewater treatment unit operations to assist in process design and evaluation. There are a few programs that include models of a variety of unit operations that can be interconnected to model a complete treatment system. These comprehensive programs allow the user to estimate the impacts of changing one unit or influent stream on the various components of the system and to determine the effects on the final effluent quality. Some programs have been designed to automatically iterate through a variety of potential treatment plant schemes to assess the optimum combination of units to achieve economic and performance criteria.

More specialized programs that have applicability in wastewater treatment facility design include programs for performing capital and O&M cost estimating, hydraulic design, and design and evaluation of discharge facilities.

Treatment Plant Operations. Computer programs and systems are being utilized to directly or indirectly operate wastewater treatment facilities. Typically, operations applications require specialized, plant-specific programs. The primary advantage of computerized operations is that a computer can absorb and evaluate large amounts of data very rapidly to make immediate operating decisions according to an established operating strategy.

There are wastewater treatment plants that are directly operated by computer systems. Operating data are input to the computer by human operators and/or automatic in-line monitoring systems (flow, pH, DO, TOC, turbidity, etc.). The computer analyzes the data and makes direct operating modifications, e.g., adjustment of activated sludge recirculation rate using an automatic valve. The function of the human operator is to monitor the system, assure that components such as valves and pumps are functional, and, most importantly, recognize and take over operations when circumstances arise that require human judgment.

An area of computer utilization that is gaining increasing popularity is indirect treatment plant operation; often referred to as “near real-time” operations assistance programs. In these situations, the computer functions as a tool for a human operator providing rapid analysis of data and recommendations on operating procedures. The operator applies judgment with consideration of the computer’s recommendation to maintain efficient operation of the plant.

Though direct computer operation systems may be justified for larger, complicated wastewater treatment plants, near real-time operation assistance systems have several advantages for smaller plants. One of the major advantages is that the initial computer equipment investment and O&M costs are significantly lower. Desktop microcomputers are being successfully used for industrial and municipal wastewater treatment plant operations assistance.

Water Quality Modeling. Computer-assisted water quality modeling has found application in a number of areas of importance to environmental engineers. The basic goal of water quality modeling is to predict downstream quality impacts on a natural waterway due to the introduction of urban runoff, treated or non-treated wastewater, or cooling water. Water quality modeling requires calculation and manipulation of large amounts of data related to climatic, stream, and waste load conditions.

A classic application of water quality modeling within environmental engineering is estimation of the dissolved oxygen “sag” downstream due to the biochemical oxygen demand from a proposed wastewater treatment discharge. Water-quality models have been written and are in use for evaluating the impacts of both conservative and nonconservative contaminants as well as thermal discharges. Some models also take naturally occurring processes, such as photosynthesis, into account.

Surface Water Hydrology. A significant effort has been made in applying computer analysis in the area of surface water hydrology. A number of major programs have been developed by the U.S. Army Corps of En-

gineers through the Hydrologic Engineering Center (HEC) located in Davis, California. Programs written by HEC and others model the hydrologic responses of natural waters to precipitation events and snowmelt. These programs are useful in a number of areas including operation of recreational and water supply reservoirs, defining floodplain elevations, determining potential flood impacts on facilities, and design and construction of diversion structures. Surface water hydrology computer programs can also be used in conjunction with water-quality modeling to evaluate the effects of sediment transport on the quality of surface waters.

Solid and Hazardous Waste Management. The safe and environmentally sound management of solid and hazardous wastes is an area that is receiving considerable attention. One of the key areas of concern is the protection of groundwater resources, which can be contaminated by inadequate waste containment and disposal practices. Computerized groundwater modeling, though not new, is receiving considerable attention, particularly the modeling of contaminant attenuation and transport processes. Predicting complicated groundwater flow patterns, dispersion processes, and chemical reactions can require a computational effort that is virtually impossible without the aid of high-speed computer technology. A typical solid waste management application of groundwater computer modeling would be predicting the down-gradient quality impacts on a drinkable groundwater source due to the installation of a proposed disposal site.

Several programs have been developed based on water budget techniques to predict leachate generation in closed landfills and impoundments due to percolation of rainfall through cover soils. These programs can be applied to aid in the design of more efficient cover and runoff diversion systems as well as in the design of leachate collection and treatment systems.

Cost estimating for construction and operation of waste-handling and disposal facilities is another area where computer techniques are being utilized. The speed of performing extensive comparative economic evaluations through computer programs is allowing for more cost-effective siting and design of solid waste management facilities.

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CHAPTER 2

ENVIRONMENTAL LEGISLATION

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Much of the formative environmental legislation in the United States has been initiated at the national level, but the individual states have enacted legislation focused on environmental issues affecting their own jurisdictions. Environmental regulations, which are the mechanism for implementing the intent of the enabling legislation, are issued by federal regulatory agencies. With the federal emphasis on delegation of authority, state agencies have the opportunity to acquire responsibility for enforcement of many environmental regulations.

Environmental legislation, and resulting regulations, are continually evolving. Clearly, legal requirements of environmental legislation can have a profound effect on economic activity as well as the future of environmental resources. This chapter addresses the rationale for and concerns about environmental legislation and regulations, legislative data systems, overviews of federal environmental legislation, and trends in regulatory actions. The legislative overview reviews legislation for the protection of air, land, and water resources and assesses its impact on environmental and cultural resources.

RATIONALE FOR ENVIRONMENTAL LEGISLATION AND REGULATIONS

The following discussion about the basis for promulgating environmental legislation and regulations focuses on the role of the market economy, the problem of the commons, and long-term viability of the environment.

Since labor and capital are scarce resources, their consumption is minimized by industry. Since the environment is, or rather has been in the past, an essentially free resource, its consumption has been ignored. Consequently, there has been considerable environmental degradation with attendant economic and social costs. Simply put, some economic and social costs are just not reflected in the market exchange of goods and services; also, one cannot ignore third-party interests (externalities) when considering two-party transactions of buyer and seller. This, in fact, is the case for many environmental control problems, and, thus, such transactions result in “market failure.” Basically, market failure could result from high transaction costs, large uncertainty, high information costs, and existence of externalities (*I*). There are two ways to correct market failure. One can try to isolate the causes of the failure and restore, as nearly as possible, an efficient market process (process orientation) or alternatively bypass the market process and promulgate regulations to achieve a certain degree of environmental protection (output orientation).

Some environmental legislation and regulations are needed to protect the health and welfare of society; market incentives alone will not work. For example, it would be very difficult to put a dollar value on discharge of toxic materials, such as PCB or mercury, into the environment. Another reason for environmental legislation and regulations is that long-term viability of the environment is essential in order to provide necessary life-support systems. Investment decisions can rarely be made to take into account long-term protection of the life-support systems that belong to everyone—a property of the commons.

2.1

Some projects involve exploitation of energy and other natural resources at an unprecedented rate, raising questions of temporal optimality of market allocations. In such cases, market economy is unable to properly account for all long-term economic and social benefits and costs. As Solow has pointed out, “. . . there are reasons to expect market interest rates to exceed the social interest rate of time preference. . .” (2) As a result, the market will tend to encourage consumption of exhaustible resources too rapid. Consequently, corrective public intervention—or regulations—aimed at slowing down this consumption needs to be structured. This can be accomplished through conservation, subsidies, or a system of graduated severance taxes (2).

CONCERNS REGARDING ENVIRONMENTAL LEGISLATION AND REGULATIONS

Many public administrators, engineers, planners, industrialists, and other decision makers recognize the need for environmental legislation and related regulations to protect the environment. They also recognize the importance of economic efficiency and utility. There are, indeed, a number of concerns regarding many environmental regulations. These concerns are shared by many who feel that environmental regulations can be structured so that they minimally affect efficiency and productivity of the industry, minimally interfere with essential federal programs such as national defense, and still achieve reasonable environmental protection goals. Some of the concerns related to environmental regulations are:

- Regulations seem to be structured in a way that the costs are excessive compared to the benefits they generate.
- In general, the regulations are of command-and-control type. Consequently, in a free-market economy, they are ineffective and do not preserve elements of voluntary choice.
- Regulations are ineffective because they lack properly structured incentives for achieving social goals.
- It is widely believed that command-and-control regulations generate inefficiencies, both at the macro- and microeconomic levels.
- Some environmental regulations require unnecessary paperwork and cause unnecessary delays in completion schedules, which, in turn, create additional costs.
- Many regulations at different government levels, such as federal, state, and local, are duplicative and, at times, incompatible with each other; consequently, they create unnecessary work and inefficiencies.

LEGISLATIVE DATA SYSTEMS

New amendments to major environmental legislation are continually being enacted by the U.S. Congress and regulatory agencies are continually modifying environmental regulations. Because of this, a number of environmental legislative data systems have been developed. Described here are some of the existing legislative data systems that readers may want to use, depending upon their specific needs.

Federal Legal Information through Electronics (FLITE)

The FLITE system operates as an information retrieval and analysis service (3). Although FLITE does not exist as an actual computerized system, it does provide access to other computerized full-text legal systems such as JURIS, LEXIS, and WESTLAW. Lawyers trained in information retrieval techniques field reference questions from federal agencies and conduct searches. This service is backed up by services such as infor-

mation analysis, abstracting, and legal research. It operates in a “batch mode,” with searches run overnight. The user seeking the information does not actually perform the search but relies on trained legal staff to perform that service. Its major drawbacks are cost and possible time delays involved in using the batch mode approach.

Justice Retrieval and Inquiry System (JURIS)

JURIS is a U.S. Department of Justice computer-based storage and retrieval system for federal legislative and litigatory information (4). It is specifically designed to serve federal lawyers. It is a full-text system and is designed for use by legal personnel. Information from the system could be rather voluminous, and consequently, difficult to use for environmental impact analysis (EIA) purposes.

Computer-Aided Environmental Legislative Data System (CELDS)

This system contains abstracts of environmental regulations and is designed for use in environmental impact analysis and environmental quality management (5, 6). The abstracts are written in an informative narrative style, with all legal jargon and excessive verbiage removed. Characteristics of this system are:

1. Legislative information is indexed to a hierarchical keyword thesaurus, in addition to being indexed to a set of environmental attributes, which number approximately 700.
2. Information can be obtained for federal and individual state environmental regulations, as well as regulatory requirements related to the keywords or environmental attributes.
3. Appropriate reference documents, such as enactment/effective date, legislative reference, administrative agency, and bibliographical reference are also included.

The system is structured in order to satisfy the user agency's (U.S. Department of Defense) specific needs for environmental regulations; consequently, needs of other agencies may not be completely satisfied by this system. Recent augmentations to the system include mining regulations of concern to U.S. Department of Energy and river-specific use classifications and regulations of concern to the U.S. Environmental Protection Agency.

LEXIS

LEXIS is a full-text system from Mead Data Central (7). It is a database with a family of files that contain the full text of the following:

1. *United States Code*—a codification by major title of the body of United States statutes
2. *Code of Federal Regulations*—a codification by major titles of current effective administrative agency regulations
3. *Federal Register* (July 1980 to the present)
4. Supreme Court decisions since 1960
5. State court decisions—courts of last resort, intermediary courts, and lower courts

WESTLAW

Like LEXIS, this is a full-text retrieval system from West Publishing Co. (8). It provides access to the text of the:

1. *United States Code*
2. *Code of Federal Regulations*
3. *Federal Register*
4. Supreme Court decisions as found in the *Supreme Court Reporter*, a West publication
5. State court decisions
6. *Shepard's Citations*—a publication providing cross references for court litigation and establishing legal precedents

OVERVIEW OF FEDERAL ENVIRONMENTAL LEGISLATION

An overview of federal environmental legislation is provided in this section. State environmental legislation and regulations have been patterned after the federal programs. Information on the selected major federal environmental laws is organized under the following main headings:

1. Basic objective
2. Key provisions
3. Enforcement responsibilities and federal–state relationship
4. Accomplishments and impacts

Clean Air Act (42 U.S.C. 7401 et seq.) 1970, 1977, 1990 Amendments

Enacted: 1967. Major amendments 1970, 1977, and 1990.

Basic Objective. The Clean Air Act of 1970, which amended the Air Quality Act of 1967, was established “to protect and enhance the quality of the Nation’s air resources so as to promote public health and welfare and the productive capacity of its population.” Since 1970, the basic act has been significantly amended to reflect national concern over air quality. Support for cleaner air has come from both environmentalists and the general public, although legislation has been politically controversial because of its impact on industry and economic growth.

The major provisions of the act are intended to set a goal for cleaner air by setting national primary and secondary ambient air quality standards. These standards define levels of air quality necessary to protect public health, while secondary standards define levels necessary to protect the public welfare from any known or anticipated adverse effects of a pollutant.

The basic objectives of the Clean Air Act Amendments of 1977 were to define issues related to significant deterioration and nonattainment areas, to implement a concept of a mission offset, to encourage use of innovative control technologies, to prevent industries from benefiting economically from noncompliance with air pollution control requirements, to state that using tall stacks to disperse air pollutants may not be considered a permanent solution to the air pollution problem, to state that federal facilities must comply with both procedural and substantive state pollution control requirements, and to establish guidelines for future EPA standard setting in a number of areas (9).

The 1990 Clean Air Act Amendments (CA 90) represent another major effort by the U.S. Congress to address many complex and controversial issues related to clean air legislation. CA 90 is expected to have profound and far-reaching effects on federal facilities and industry. One indication of the magnitude of efforts commanded by these amendments is their estimated cost. Expenditures to meet the new requirements are projected to be \$20 to \$25 billion per year. Basic objectives of CA 90 are to overhaul the nonattainment provisions, to create an elaborate technology-based control program for toxic air pollutants, to address acid precipitation and power plant emissions, to mandate the phaseout of CFCs, and to greatly strengthen enforcement powers of regulatory agencies.

Key Provisions. Key provisions of the seven most important titles of the act are summarized as follows:

Title I: Attainment and Maintenance of the National Ambient-Air Quality Standards. This title describes air pollution control requirements for geographic areas in the United States that have failed to meet the National Ambient Air Quality Standards (NAAQS). These areas are known as nonattainment areas. Ozone is currently the most pervasive nonattainment pollutant in the United States, and this title is directed at controlling the pollutants (volatile organic compounds and nitrogen oxides) that contribute to ground-level ozone formation. For cities out of attainment for ozone, the 1990 amendments require that such cities take steps to reduce ozone with the eventual goal of reaching attainment. Methods depend on level of nonattainment: marginal, moderate, serious, severe, and extreme. Similar categorization of U.S. cities is performed for cities out of attainment for other criteria pollutants (i.e., carbon monoxide and particulate matter). The more severe the category, the more drastic the pollutant reduction measure. Title VI of this act discusses stratospheric ozone issues.

Title II: Mobile Sources. This title deals with revised tailpipe emission standards for motor vehicles. Requirements under this title compel automobile manufacturers to improve design standards to limit carbon monoxide, hydrocarbon, and nitrogen oxide emissions. Manufacturers must also investigate the feasibility of controlling refueling emissions. For the worst ozone and carbon monoxide nonattainment areas, reformulated and oxygenated gasolines will be required.

Title III: Hazardous Air Pollutants. This title deals with control of hazardous air pollutant emissions and contingency planning for accidental release of these pollutants. Requirements of this title are, perhaps, the most costly aspects of CA 90.

Title IV: Acid Deposition Control. The amendments establish a totally new control scheme for addressing the acid rain problem. The exclusive focus is on power plant emissions of sulfur dioxide and nitrogen oxide. Sulfur dioxide emissions are to be reduced by approximately 10 million tons annually in two phases—the first to take effect in 1995, the second in 2000. It is important to note that these reductions are to be achieved through a new market-based system under which power plants are to be allocated “emissions allowances” that will require plants to reduce their emissions or acquire allowances from others to achieve compliance. The target for the reduction of nitrogen oxide is established at 2 million tons per year.

Title V: Permits. This title provides for the states to issue federally enforceable operating permits to applicable stationary sources. The permits are designed to improve the ability of the federal EPA, state regulatory agencies, and private citizens to enforce the requirements of CA 90. These permits will also be used to clarify operating and control requirements for stationary sources.

Title VI: Stratospheric Ozone Protection. This title limits emissions of chlorofluorocarbons (CFCs), halons, and other halogenic chemicals that contribute to the destruction of stratospheric ozone. Provisions of this title closely follow the control strategies recommended in June 1990 by the second meeting of parties to the Montreal protocol.

Title VII: Enforcement. Requirements of this title completely replace existing enforcement provisions in the Clean Air Act Amendments of 1977. New enforcement actions include higher maximum fines and terms of imprisonment. Seriousness of violations has been upgraded and liabilities are now targeted at senior management rather than on-site operators.

Enforcement Responsibilities; Federal–State Relationship. A major provision of the Clean Air Act establishes the concept of the state acceptance of the primary issue. Under the enforcement responsibilities established in the act, the EPA sets certain federal minimum standards and procedures. The states must then pass their own regulatory programs based upon these minimum standards. State programs must be submitted to the EPA for approval before the state can accept enforcement responsibilities. In lieu of an approved state program, the federal program will be in force. State regulatory programs must address the issue of how to improve air quality in areas not meeting NAAQS, and protecting areas that meet NAAQS from deterioration of air quality.

Since the 1990 amendments, states have been passing and reviewing their regulatory programs to reflect deadlines mandated by the act. The impact of these regulatory programs has been enormous. The EPA must review and approve or disapprove programs for 50 states, each of which must incorporate all the key provisions of the act into the programs.

Accomplishments and Impacts. Although significant strides have been made in improving air quality since the Clean Air Act was originally passed in 1970, the nation's concern with air pollution and its impacts is still evolving. Some politically unpopular control strategies in the area of land use regulations and transportation controls have been modified or eliminated. Many statutory deadlines have been postponed. In order to provide for continued economic growth, and recognizing the energy needs of the nation, many air pollution control requirements continue to be modified.

Noise Control Act (42 U.S.C. 4901 et seq.)

Enacted: 1972 (PL 92-574). Major amendments: 1976 (PL 94-301); 1978 (PL 95-609).

Basic Objective. Noise pollution is one of the most pervasive environmental problems. A report to the President and Congress on noise indicates that between 80 and 100 million people are bothered by environmental noise on a daily basis, and approximately 40 million people are adversely affected (10).

Since noise is a by-product of human activity, the extent of exposure increases as a function of population growth, population density, mobility, and industrial activities. Acts such as NEPA also have an effect on noise control requirements and related land uses.

In congressional hearings regarding federal aviation noise policy, it was pointed out that aviation noise is a serious environmental problem for those who live near airports. The Federal Aviation Administration has authority to regulate aircraft noise emissions, and classifies aircraft into three categories based on their noise levels. Stage 1 aircraft, with the highest emissions, are planes manufactured in the 1960s and 1970s. The original 707 and DC-8 are examples. Stage 2 aircraft represent newer designs, such as the 737 and later models of the 727. Stage 3 aircraft are the newest designs, mostly of mid-1980s production, such as the MD80 and 767, and are notably quieter than older designs.

Since 1988, operation of stage 1 aircraft have been flatly prohibited at many urban airports, which has reduced the number of persons seriously affected by noise from an estimated 7 million in the mid-1970s to 3.2 million in 1990. Stage 2 aircraft may continue to be operated, though their proportion in the fleet is decreasing through natural attrition, and all are expected to drop out of use after the year 2000. Many citizens' groups and airport authorities are requesting even faster phaseout of stage 2 aircraft. The European Community prohibits the purchase of new stage 2 aircraft, even as replacements, and plans to phase out their use well before 2000. The business and economic implications of this regulation of aircraft type are serious. The mix of stage 2 and stage 3 aircraft varies widely among airline companies, with some of the highest proportions of older aircraft being held by companies in relatively poor financial condition, who may not be able to afford the purchase of new aircraft (11).

The Noise Control Act has four basic objectives:

1. New product noise emission standards directed principally at surface transportation and construction noise sources
2. The utilization of "in-use" controls directed principally at aviation, interstate motor carriers, and railroad noise sources
3. The labeling of products for protection against voluntary high-level individual exposure
4. The development of state and local programs to control noise

Key Provisions. The act mandates the EPA to promulgate standards for noise emissions from the following new products:

- Portable air compressors
- Medium- and heavy-duty trucks
- Earth-moving machinery
- Buses
- Truck-mounted solid waste compactors
- Motorcycles
- Jackhammers
- Lawn mowers

Additionally, the act specifies that the following sources will be regulated via performance standards:

- Construction equipment
- Transportation equipment (with the aid of the Department of Transportation)
- Any motor or engine
- Electrical or electronic equipment
- Any other source that can feasibly be regulated

Section 7 of the act also amends the Federal Aviation Act and regulates aircraft noise and sonic booms. The Federal Aviation Administration (FAA) is given the authority to regulate such noise after consultation with and review by the EPA.

In 1978, the Noise Control Act was amended by the Quiet Communities Act. This amendment provided for greater involvement by state and local authorities in controlling noise. Its objectives are:

- The dissemination of information concerning noise pollution
- The conducting or financing of research on noise pollution
- The administration of the quiet communities program, which involves grants to local communities, the monitoring of noise emissions, studies on noise pollution, and the education and training of the public concerning the hazards of noise pollution
- The development and implementation of a national noise environmental assessment program to (a) identify trends in noise exposure, (b) set ambient levels of noise, (c) set compliance data, (d) assess the effectiveness of noise abatement
- The establishment of regional technical assistance centers

The EPA is further given the authority to certify a product as acceptable for low noise emission levels. These certified products are to be used by federal agencies in lieu of a like product that is not certified.

Enforcement Responsibilities; Federal–State Relationship. The EPA has enforcement responsibilities under the act, as indicated in the key provisions, and is mandated to promulgate noise emission standards. The FAA controls noise from aircraft and sonic booms.

The 1978 amendments (Quiet Communities Act) were an attempt to recognize that noise pollution is very often a local community problem and needs to be regulated at that level. Thus, many noise regulations are promulgated at the local level, with support from the state and national level in the form of grants and research results.

Accomplishments and Impacts. The effects of the law are

1. The establishment of noise emission standards for:

- Construction equipment
- Interstate motor vehicles (40 CFR, Part 202)
- Railroads (40 CFR, Part 201)

- Portable air compressors (40 CFR, Part 204)
 - Aircraft noise and sonic boom
2. The establishment of labeling requirements for certain types of equipment (40 CFR, Part 211)
 3. The establishment of the quiet communities program, which has encouraged more involvement by state and local agencies in the setting of more stringent noise levels and the enforcement of those levels
 4. The requirement for federal agencies to purchase equipment certified by the EPA as having low-noise emissions in lieu of like products not having a certificate.

On a more fundamental level, the act has served to increase noise pollution awareness on the part of the public and has validated concerns over this often overlooked type of pollution. It has stimulated more and better research into the effects of noise on the quality of life, as well as the health hazard aspects.

Safe Drinking Water Act (42 U.S.C. 300f et seq.)

Enacted: 1974 (PL 93-523). Major amendments: 1977 (PL 95-190); 1986 (PL 99-339); 1996 (PL 104-182).

Basic Objectives. The primary objectives of the Safe Drinking Water Act are twofold: (1) to protect the nation's sources of drinking water, and (2) to protect public health to the maximum extent possible, using proper water-treatment techniques (12). The act establishes the need to set contaminant levels to protect public health. These levels were established in regulations issued pursuant to the act, which requires the EPA to develop regulations for the protection of underground sources of drinking water. Any underground injection of wastewater must be authorized by a permit. Such a permit is not issued until the applicant can prove that such disposal will not affect drinking water sources. Finally, the act requires procedures for inspection, monitoring, record keeping, and reporting.

Key Provisions. Key provisions of the act can be summarized as:

1. The establishment of national primary drinking water standards based on maximum contaminant levels (MCLs)
2. The establishment of treatment techniques to meet the standards
3. The establishment of secondary drinking water standards
4. The establishment of those contaminants for which standards are set, based on studies conducted by the National Academy of Sciences. The EPA requests comments from the Science Advisory Board, established under the Environmental Research, Development, and Demonstration Act of 1978, prior to proposals on new or revised MCLs
5. The establishment of state management programs for enforcement responsibilities. States must submit regulatory programs to the EPA for approval. These programs must set primary and secondary drinking water standards that meet or better the national standards. They must also regulate by permit facilities that treat drinking water supplies
6. The protection of underground sources of drinking water
7. The establishment of procedures for development, implementation, and assessment of demonstration programs designed to protect critical aquifer protection areas located within areas designated as sole or principal source aquifers
8. The requirement for state programs to protect wellhead areas from contaminants that may have adverse effects on public health
9. Originally, the EPA was required to regulate twenty-five additional drinking water contaminants each

year. The 1996 amendments changed this requirement and instead mandated that the EPA regulate the contaminants that pose the greatest risk and are most likely to occur in water systems.

10. The 1996 amendments created a fund that aids water systems. The fund provides assistance for infrastructure upgrades and source water protection programs.

Enforcement Responsibilities; Federal–State Relationship. The passage of the Drinking Water Act in December 1974, and amendments passed through 1996, have broadened EPA’s authority and responsibility to regulate the quality of the nation’s drinking water regulations, with the states having the major responsibility for enforcing these regulations.

States must submit drinking water programs to the EPA for approval. These programs must meet, at a minimum, the federal standards for drinking water quality. They must also include procedural aspects for inspection and monitoring, as well as control technology and emergency procedures for noncompliance to protect the public health. States are also given enforcement responsibilities for the control of underground sources of water supply. These responsibilities must include permitting procedures.

Accomplishments and Impacts. There are more than 240,000 public water supply systems serving over 200 million people. Many of these systems are not using the most effective equipment and techniques to collect, treat, and deliver potable water to the public. According to the EPA (13), more than half of these systems are out of compliance because of

1. Inadequate treatment techniques
2. Inadequately trained operators
3. Poor system design
4. Inadequate monitoring procedures

The only variations from state to state are procedural, such as record keeping. Issues involving other legislation are also closely tied to safe drinking water; for example, the protection of the nation’s waterways under the Clean Water Act impacts the ultimate protection of the water supply for potable water. Similarly, the leaching of hazardous wastes into groundwater can affect underground water quality. Thus, the quality of sources of drinking water is closely tied by other major legislation to the control of pollution.

Clean Water Act (33 U.S.C. 1251 et seq.)

Enacted: 1948 (Ch. 758, 62 Stat. 1155). Major amendments 1956 (Ch. 518, §1, 70 Stat. 498); 1972 (PL 92-500); 1977 (PL 95-217); 1987 (PL 100-4) (14).

Basic Objective. The Clean Water Act is the primary authority for water pollution control programs with emphasis on surface waters. The objective of these programs is to “restore and maintain the chemical, physical, and biological integrity of the nation’s waters.” The act set national goals to:

1. Eliminate the discharge of pollutants into navigable waters by 1985
2. Set interim goals of water quality that will protect fish and wildlife and will provide for recreation by July 1, 1983
3. Prohibit the discharge of toxic pollutants in quantities that might adversely affect the environment
4. Construct publicly owned waste-treatment facilities with federal financial assistance
5. Establish waste-treatment management plans within each state
6. Establish the technology necessary to eliminate the discharge of pollutants
7. Develop and implement programs for the control of nonpoint sources of pollution to enable the goals of the act to be met

The goals are to be achieved by a legislative program that includes permits under the National Pollutant Discharge Elimination System (NPDES). Effluent limitations imposed under the initial legislation required the existing sources of pollution to use the “best practicable” treatment technology by 1977 and the “best available” technology by 1983; amendments provided means for modification of compliance dates. It requires an independent set of effluent limitations of new sources.

Key Provisions. Development of effluent standards and permit systems and state and local responsibilities are key provisions of the act.

Effluent standards for existing and new sources of water pollution are established. These are source-specific limitations. Also, the act lists categories of point sources for which the EPA must issue standards of performance for new sources. States must develop and submit to the EPA a procedure for applying and enforcing these standards.

The EPA may establish a list of toxic pollutants and establish effluent limitations based on the best available technology economically achievable for point sources designated by the EPA. The EPA has also issued pretreatment standards for toxic pollutants.

Anyone conducting an activity, including construction or operation of a facility, that may result in any discharge into navigable waters must first obtain a permit. Permit applications must include a certification that the discharge meets applicable provisions of the act, under NPDES. Permits for a discharge into ocean water are issued under separate guidelines from the EPA. The Corps of Engineers issues permits for the discharge of dredged or fill material in ocean water, based on criteria established by the Corps.

The act makes provision for direct grants to states to help them in administering pollution-control programs. It also provides grants to assist in the development and implementation of waste-treatment management programs, including the construction of waste-treatment facilities. The federal share of construction costs was to be no more than 55% after October 1, 1984.

To be eligible for these grants, states must develop waste-treatment management plans that are based upon federally issued guidelines. These programs must be approved by the EPA and must include:

1. Regulatory programs to assure that the treatment facilities will include applicable pretreatment requirements
2. The identification of sources of pollution and the process by which control will be achieved
3. A process to control sources of groundwater pollution
4. The control of pollution from dredged or fill material into navigable waters; this must meet Section 404 requirements of this act

Waste-treatment management is on an areawide basis, providing for the control of pollution from all point and nonpoint sources. In addition, the states are required to develop implementation plans for EPA approval to meet minimum water quality standards established by the EPA.

Other provisions of the act state that federal facilities must comply with all federal, state, and local requirements for the abatement and control of pollution. Also, the act provides grants to conduct a national wetlands inventory.

In November 1990, the EPA issued regulations setting forth the NPDES permit application requirements for stormwater discharges associated with industrial activities, discharge from municipal storm sewer systems that serve urban areas of 250,000 population or greater, and discharges from municipal storm sewer systems serving populations between 100,000 and 250,000.

Enforcement Responsibilities; Federal–State Relationship. Except for issuing permits for the discharge of dredged or fill material, the EPA has no enforcement responsibilities for the act. The Corps of Engineers has the responsibility of issuing permits for specific categories of activities involving discharge of dredged

or fill materials if the discharge will cause only minimal adverse effects. Sites for the discharge of dredged or fill material are specified by EPA guidelines.

Like many other major environmental statutes, the Clean Water Act emphasizes eventual state primacy and enforcement responsibilities. When the state has plans for preserving or restoring water quality, and the EPA has approved those programs, the state will then assume enforcement responsibilities. Both these programs are based upon a minimal federal regulatory involvement. The federal role is also one of providing grants to states for the implementation of these programs.

Accomplishments and Impacts. The Clean Water Act is enforced through two major interrelated strategies—a statutory program for the improvement of water quality and a related program of federal grants for the construction and expansion of wastewater treatment works.

A national clean water goal, initially to be achieved by 1983, first provides a statutory guideline for a legislative program intended to eliminate all pollution in national waters. Discharge permits were then required for all water effluent discharges into national waters, and these permits may not be granted unless the source of the discharge utilizes the effluent treatment technology required by the act. These discharge permits are granted and administered according to the NPDES, initially to be administered by EPA but which may be transferred for administration to the states subject to their compliance with detailed criteria contained in the federal law.

Effluent limitations imposed under the act generally require that existing sources of pollution make use of the “best practicable” treatment technology by 1977, and the “best available” technology by 1983, and the statute also imposes an independent set of effluent limitations on new sources of water pollution. Discharges from wastewater treatment plants also require a discharge permit under the NPDES system.

Water quality standards established under the earlier water quality act are also continued. Standards must be established by a state if it has not done so previously. The EPA and the states must establish more stringent effluent limitations than those otherwise required by the act if needed to meet water quality standards. As in the Clean Air Act, the water quality and discharge permit requirements of the Clean Water Act can be expected to have a major impact on land development patterns through the influence they exert on the location of water pollution sources.

As in the Clean Air Act, the water pollution control requirements are applied principally to point sources of pollution. Where the Clean Water Act differs from the Clean Air Act is in its specific statutory requirements for a water quality planning program that includes specific land development control authority.

The Clean Water Act also addresses problems caused by the diffusion of water from nonpoint areawide sources of pollution, such as stormwater runoff and water runoff from on-site construction activities. Controls over nonpoint sources are also required by the act. They are first required in the “regulatory program, which must be a part of the areawide waste treatment planning process. This program must include “procedures and methods,” including “land use requirements,” to control nonpoint pollution sources.

The dredge and fill program under Section 404 of the 1972 Water Pollution Control Act authorizes a permit program for dredge and fill activities in “waters of the United States” to be administered by the U.S. Army Corps of Engineers. Deliberate congressional selection of the language defining the jurisdiction of the Corps led to an expansion of the program to include coastal and freshwater wetlands as well as navigable waters. This extension of jurisdiction makes a federal dredge and fill permit necessary for residential and other development in wetlands areas. The Corps is authorized to issue permits for dredge and fill activities at disposal sites specified by the Corps under regulations jointly developed by it and the EPA. The required review covers analysis similar to the environmental assessments or environmental impact statements. The Corps is to consider the need for the permit, alternative locations and methods, beneficial and detrimental effects, and cumulative impacts.

The act also authorizes the EPA to veto dredge and fill permits issued by the Corps of Engineers if they have an “unacceptable adverse effect” on municipal water supplies or shellfish beds or on fishery, wildlife,

or recreational areas. With the 1977 amendments, Congress preserved the broad jurisdiction of the dredge and fill program over all waters, but authorized a delegation to the states of the authority to issue permits for waters not classed as navigable and shifted control over nonpoint sources of pollution to Section 208. An amendment to Section 404 exempts from the dredge and fill permit requirement a series of earth-moving activities such as normal farming and construction sites, as well as any nonpoint sources subject to control under a state nonpoint source control program approved under Section 208.

The Water Pollution Control Amendments of 1981 (PL 97-17) legalized oxidation ponds, lagoons and ditches, and trickling filters as the equivalent of secondary treatment if water quality is not adversely affected.

Under this act, the EPA administers programs that provide financial grants to local agencies for the planning of wastewater management facilities. The Corps of Engineers participates in the planning of wastewater facilities or systems as follows:

1. The Corps of Engineers may perform a single-purpose wastewater management study in response to a congressional resolution or an act of Congress.
2. The Corps of Engineers may engage in wastewater management planning as part of an urban study.
3. The Corps of Engineers may provide advisory assistance to local or state agencies engaged in areawide waste treatment planning at the request of such agency.

Water quality planning under Section 208, also referred to as “208 Planning,” was initiated under this act. A substantial number of 208 plans were developed.

The prevailing trend in water pollution control regulation and research has been in the direction of technology-based rather than water-quality-based, causing some point-source pollution control projects to become excessively costly by providing treatment beyond the levels required by receiving waters. On the other hand, pressing problems like surface runoff, combined sewer overflows, operation and maintenance, and toxic and hazardous waste disposal remained unresolved. Many scientists and engineers recommend that the facilities to treat point-source pollutants should be developed in concert with measures that may be needed for control of nonpoint sources.

Resource Conservation and Recovery Act (RCRA) (42 U.S.C. 6901 et seq.)

Enacted: 1976 (PL 94-580) (as an amendment that completely revised the Solid Waste Disposal Act). Major amendment: 1984 (PL 98-6 16) (Hazardous and Solid Waste Amendments of 1984); 1992 (PL 102-386) (Federal Facility Compliance Act) (15).

Basic Objective. RCRA, as it now exists, is the culmination of a long series of pieces of legislation, dating back to the passage of the Solid Waste Disposal Act of 1965, which address the problem of waste disposal. It began with the attempt to control solid waste disposal and eventually evolved into an expression of the national concern with the safe and proper disposal of hazardous waste. Establishing alternatives to existing methods of land disposal and to conversion of solid wastes into energy are two important needs noted by the act.

The RCRA of 1976 gives the EPA broad authority to regulate the disposal of hazardous wastes; encourages the development of the solid waste management plans and nonhazardous waste regulatory programs by states; prohibits open dumping of wastes; regulates underground storage tanks; and provides for a national research, development, and demonstration program for improved solid waste management and resource conservation techniques.

The control of hazardous wastes will be undertaken by identifying and tracking hazardous wastes as they are generated, ensuring that hazardous wastes are properly contained and transported, and regulating the storage, disposal, or treatment of hazardous wastes. A major objective of the RCRA is to protect the environment and conserve resources through the development and implementation of solid waste management plans by the states. The act recognizes the need to develop and demonstrate waste management practices that are not only environmentally sound and economically viable but also conserve resources. The act requires the EPA to undertake a number of special studies on subjects such as resource recovery from glass and plastic waste and managing the disposal of sludge and tires. An Interagency Resource Conservation Committee has been established to report to the President and the Congress on the economic, social, and environmental consequences of present and alternative resource conservation and resource recovery techniques.

Key Provisions. Hazardous wastes are identified by definition and publication. Four classes of definitions of hazardous waste have been identified: ignitability, reactivity, corrosivity, and toxicity. The chemicals that fall into these classes are regulated primarily because of the dangerous situations they can cause when land-filled with typical municipal refuse. Four lists, containing approximately 1000 distinct chemical compounds, have been published. (These lists are revised as new chemicals become available.) These lists include waste chemicals from nonspecific sources, by-products of specific industrial processes, and pure or off-specification commercial chemical products. These classes of chemicals are regulated primarily to protect groundwater from contamination by toxic products and by-products.

The act requires tracking of hazardous wastes from generation, to transportation, to storage, to disposal or treatment. Generators, transporters, and operators of facilities that dispose of solid wastes must comply with a system of record keeping, labeling, and manufacturing to ensure that all hazardous waste is designated only for authorized treatment, storage, or disposal facilities. The EPA must issue permits for these facilities, and they must comply with the standards issued by the EPA.

The states must develop hazardous waste management plans, which must be EPA-approved. These programs will regulate hazardous wastes in the states and will control the issuance of permits. If a state does not develop such a program, the EPA, based on the federal program, will do so.

Solid waste disposal sites are to be inventoried to determine compliance with the sanitary landfill regulations issued by the EPA. Open dumps are to be closed or upgraded within 5 years of the inventory. As with hazardous waste management, states must develop management plans to control the disposal of solid waste and to regulate disposal sites. EPA has issued guidelines to assist states in developing their programs.

As of 1983, experience and a variety of studies dating back to the initial passage of the RCRA legislation found that an estimated 40 million metric tons of hazardous waste escaped control annually through loopholes in the legislative and regulatory framework. Subsequently, Congress was forced to reevaluate RCRA, and in doing so found that RCRA fell short of its legislative intent by failing to regulate a significant number of small-quantity generators, regulate waste oil, ensure environmentally sound operation of land disposal facilities, and realize the need to control the contamination of groundwater caused by leaking underground storage tanks. Major amendments were enacted in 1984 in order to address the shortcomings of RCRA. Key provisions of the 1984 amendments include:

- Notification of underground tank data and regulations for detection, prevention, and correction of releases
- Incorporation of small-quantity generators (which generate between 100 and 1000 kg of hazardous waste per month) into the regulatory scheme
- Restriction of land disposal of a variety of wastes unless EPA determines that land disposal is safe from human health and environmental points of view
- Requirement of corrective action by treatment, storage, and disposal facilities for all releases of hazardous waste regardless of when the waste was placed in the unit

- Requirement of the EPA to inspect government-owned facilities (which handle hazardous waste) annually, and other permitted hazardous waste facilities at least every other year
- Regulation of facilities that burn wastes and oils in boilers and industrial furnaces

Enforcement Responsibilities; Federal–State Relationship. Subtitle C of the Solid Waste Disposal Act, as amended by the RCRA of 1976, directs the EPA to promulgate regulations for the management of hazardous wastes.

The hazardous waste regulations, initially published in May 1980, control the treatment, storage, transport, and disposal of waste chemicals that may be hazardous if landfilled in the traditional way. These regulations (40 CFR 261-265) identify hazardous chemicals in two ways—by listing and by definition. A chemical substance that appears on any of the lists or meets any one of the definitions must be handled as a hazardous waste.

Like other environmental legislation, RCRA enforcement responsibilities for hazardous waste management will eventually be handled by each state, with federal approval. Each state must submit a program for the control of hazardous waste. These programs must be approved by the EPA before the state can accept enforcement responsibilities.

The state programs will pass through three phases before final approval will be given. The first phase is the interim phase, during which the federal program will be in effect. The states will then begin submitting their programs for the control of hazardous wastes. The second-phase programs will address permitting procedures. A final phase will provide federal guidance for design and operation of hazardous waste disposal facilities. Many states have chosen to allow the federal programs to suffice as the state program to avoid the expense of designing and enforcing the program.

It should also be noted that the Department of Transportation has enforcement responsibilities for the transportation of hazardous wastes and for the manifest system involved in transporting.

Accomplishments and Impacts. The 1980 regulations for the control of hazardous wastes were a response to the national concern over hazardous waste disposal. States have begun to discover their own “Love Canals,” and the impacts of unregulated disposal of hazardous wastes on their communities. While the “Superfund” legislation provides funds for the cleanup of such sites, RCRA attempts to avoid future “Love Canals.”

Comprehensive Environmental Response, Compensation and Liability Act (CERCLA) (42 U.S.C. 9601 et seq.)

Enacted: 1980 (PL 96-510). Major amendment: 1986 (PL 99-499) (Superfund Amendments and Reauthorization Act) (16).

Basic Objective. The act, known as Superfund or CERCLA, has four objectives:

1. To provide the enforcement agency the authority to respond to the releases of hazardous wastes (as defined in the Federal Water Pollution Control Act, Clean Air Act, Toxic Substances Control Act, Solid Waste Disposal Act, and by the administrator of the enforcement agency) from “inactive” hazardous waste sites which endanger public health and the environment
2. To establish a hazardous substance Superfund
3. To establish regulations controlling inactive hazardous waste sites
4. To provide liability for releases of hazardous wastes from such inactive sites

The act amends the Solid Waste Disposal Act. It provides for an inventory of inactive and “orphan” hazardous waste sites and for appropriate action to protect the public from the possible dangers of such sites. It is a response to the concern for the dangers of negligent hazardous waste disposal practices.

Key Provisions

1. The establishment of a hazardous substance Superfund based on fees from industry and federal appropriations to finance response actions
2. The establishment of liability to recover costs of response from liable parties and to induce the cleanup of sites by responsible parties
3. The determination of the number of inactive hazardous waste sites by conducting a national inventory. This inventory includes coordination by the Agency for Toxic Substances and Disease Registry (ATSDR) of the Public Health Service for the purpose of implementing the health-related authorities in the act.
4. The provision of the authority of the EPA to act when there is a release or threat of release of a pollutant from a site that may endanger public health. Such action may include “removal, remedy, and remedial action.”
5. The revision, within 180 days of enactment of the act, of the National Contingency Plan for the Removal of Oil and Hazardous Substances (40 CFR, Part 300). This plan must include a section to establish procedures and standards for responding to releases of hazardous substances, pollutants, and contaminants and abatement actions necessary to offset imminent dangers.

Enforcement Responsibilities, Federal–State Relationship. The EPA has responsibility for enforcement of the act as it pertains to the inventory, liability, and response provisions. The EPA is also responsible for claims against the Hazardous Substance Superfund, which is administered by the president. The EPA is responsible for promulgating regulations to designate hazardous substances, reportable quantities, and procedures for response. The National Response Center, established by the Clean Water Act, is responsible for notifying appropriate government agencies of any release.

The following Department of Transportation agencies also have responsibilities under the act:

- U.S. Coast Guard—response to releases from vessels
- Federal Aviation Administration—responses to releases from aircraft
- Federal Highway Administration—responses to releases from motor carriers
- Federal Railway Administration—responses to releases from rolling stock

States are encouraged by the act to participate in response actions. The act authorizes the EPA to enter into contracts or cooperative agreements with states to take response actions. The fund can be used to defray costs to the states. The EPA must first approve an agreement with the state, based on the commitment by the state to provide funding for remedial implementation. Before undertaking any remedial action as part of a response, the EPA must consult with the affected state(s).

Accomplishments and Impacts. On July 16, 1982, the EPA published the final regulations pursuant to Section 105 of the act, revising the National Contingency Plan for Oil and Hazardous Substances under the Clean Water Act, reflecting new responsibilities and powers created by CERCLA. The plan establishes an effective response program. Because the act requires a national inventory of inactive hazardous waste sites, the intent is to identify potential danger areas and effect a cleanup or remedial actions to avoid or mitigate public health and environmental dangers. In studying a sampling of these sites, the House Committee on Interstate and Foreign Commerce (House Report No. 96-1016) found four dangerous characteristics common to all the sites. These characteristics are:

1. Large quantities of hazardous wastes
2. Unsafe design of the sites and unsafe disposal practices
3. Substantial environmental danger from the wastes
4. The potential for major health problems for people living and working in the area of the sites

The intent of the act is to eliminate the above problems by dealing with the vast quantities of hazardous and toxic wastes in unsafe disposal sites in the country. The immediate impact of the act has been the identification of the worst sites where the environmental and health dangers are imminent. This priority list will be used to spend the money available in the Hazardous Waste Response Fund in the most effective way to eliminate the imminent dangers. The long-term impact of the act will be to eliminate and clean up all the identified inactive sites and develop practices and procedures to prevent future hazards in such sites, whether active or inactive. Another accomplishment of the act is to establish liability for the cost of cleanup to discourage unsafe design and disposal practices. The act has armed the EPA with the authority to pursue an active program of cost recovery for cleanup from responsible parties.

Superfund Amendments and Reauthorization Act (SARA) (42 U.S.C. 11001 et seq.)

Enacted: 1986 (PL 99-499) (17).

Basic Objective. This act revises and extends CERCLA (Superfund authorization). CERCLA is extended by the addition of new authorities provided for by the Emergency Planning and Community Right-to-Know Act of 1986 (also known as Title III of SARA). Title III of SARA provides for “emergency planning and preparedness, community right-to-know reporting, and toxic chemical release reporting. This act also establishes a special program within the Department of Defense for restoration of contaminated lands, somewhat similar to the Superfund under CERCLA.

Key Provisions. There are key provisions that apply when a hazardous substance is handled and when an actual release has occurred. Even before any emergency has arisen, certain information must be made available to state and local authorities, and to the general public upon request. Facility owners and operators are obligated to provide information pertaining to any regulated substance present on the facility to the appropriate state or local authorities (Subtitle A). Three types of information are to be reported to the appropriate state and local authorities (Subtitle B):

1. Material safety data sheets (MSDSs), which are prepared by the chemical manufacturer of any hazardous chemical and are retained by the facility owner or operator (or, if confidentiality is a concern, a list of hazardous chemicals for which MSDSs are retained can be made available). These sheets contain general information on a hazardous chemical and provide an initial notice to the state and local authorities.
2. Emergency and hazardous chemical inventory forms, which are submitted annually to the state and local authorities. Tier I information includes the maximum amount of a hazardous chemical that may be present at any time during the reporting year and the average daily amount present during the year prior to the reporting year. Also included is the “general location of hazardous chemicals in each category.” This information is available to the general public upon request. Tier II information is reported only if requested by an emergency entity or fire department. This information provides a more detailed description of the chemicals, the average amounts handled, the precise location, storage procedures, and whether the information is to be made available to the general public (allowing for the protection of confidential information).
3. Toxic chemical release reporting, which releases general information about effluents and emissions of any “toxic chemicals.”

In the event that a release of a hazardous substance does occur, a facility owner or operator must notify the authorities. This notification must identify the hazardous chemical involved; amounts released; time, duration, and environmental fate; and suggested action.

A multilayer emergency planning and response network on the state and local government levels is to be established, also providing a notification scheme in the event of a release.

Enforcement Responsibilities; Federal–State Relationship. Local emergency planning committees or an emergency response commission appointed by the governor of the state are responsible for the response scheme. The primary drafters of the local response plans are local committees, which are also responsible for initiating the response procedure in the event of an emergency. Each state commission will supervise the local activities.

Accomplishments and Impacts. SARA legislation to promote emergency planning and to provide citizen information at the local level was a response to the Bhopal, India, disaster. A major intent is to reassure U.S. citizens that a similar tragedy will not occur in this country, and thus have a calming effect. The standardization of reporting and record keeping should produce long-term benefits and well-designed response plans. Whether a high-quality emergency response involvement can be maintained indefinitely at the local level remains a question.

Federal Insecticide, Fungicide, and Rodenticide Act (FIFRA) (7 U.S.C. 136 et seq.)

Enacted: 1947 (Ch. 125, 61 Stat. 163). Major amendments: 1972 (PL 92-5 16); 1978 (PL 95-396); 1988 (PL 100-532); 1991 (PL 102-237) (18).

Basic Objective. FIFRA is designed to regulate the use (which is in excess of 1 billion pounds) and safety of pesticide products within the United States. The 1972 amendments, a major restructuring that established the contemporary regulatory structure, are intended to ensure that the environmental harm resulting from the use of pesticides does not outweigh the benefits.

Key Provisions

1. The evaluation of risks posed by pesticides (requiring registration)
2. The classification and certification of pesticides by specific use (as a way to control exposure)
3. The restriction of the use of pesticides that are harmful to the environment (or suspending or canceling the use of the pesticide)
4. The enforcement of the above requirements through inspections, labeling, notices, and state regulation

Enforcement Responsibilities; Federal–State Relationship. The EPA is allowed to establish regulations concerning registration, inspection, fines, and criminal penalties, and to stop the sales of pesticides. Primary enforcement responsibility, however, has been assumed by almost every state. Federal law only specifies that each state must have adequate laws and enforcement procedures to assume primary authority.

As in the case for almost any federal law, FIFRA preempts state law to the extent that it addresses the pesticide problem. Thus, a state cannot adopt a law or regulation that counters a provision of FIFRA, but can be more stringent.

Accomplishments and Impacts. While the volume of pesticides and related information is enormous, FIFRA has enabled the EPA to acquire much information for analysis of risk and environmental degradation that results from the use of pesticides. This information has been, and will continue to be, generally invaluable.

able in such analyses. However, Congress continues to struggle with the balancing of benefits and detriments of the use of pesticides in its attempt to deal with the economic, scientific, and environmental issues that are involved in the regulation of pesticides.

Marine Protection, Research, and Sanctuaries Act (33 U.S.C. 1401 et seq.)

Enacted: 1972 (PL 92-532). Major amendments: 1984 (PL 98-498); 1988 (PL 100-627) (19).

Basic Objective. This act regulates the dumping of all types of materials into the ocean. It prevents or severely restricts the dumping of materials adversely affecting human welfare, the marine environment, ecological systems, or economic potentialities. It provides for a permitting process to control the ocean dumping of dredged material.

The act also establishes the marine sanctuaries program, which designates certain areas of the ocean waters as sanctuaries, when such designation is necessary to preserve or restore these areas for their conservation, recreation, ecology, or aesthetic values. States are involved in the program through veto powers to prohibit a designation.

Key Provisions. The EPA is responsible for issuing permits for the dumping of materials in ocean waters except for dredged material (regulated by the Corps of Engineers), radiological, chemical, and biological warfare agents, and high-level radioactive waste for which no permits are issued.

The EPA has established criteria for reviewing and evaluating permit applications (40 CFR, Subchapter H). These criteria consider:

1. The need for the proposed dumping
2. The effect of such dumping on human health and welfare, including economic, aesthetic, and recreational values
3. The effect of such dumping on marine ecosystems
4. The persistence and permanence of the effects of the dumping
5. The effect of dumping particular volumes and concentrations of such materials
6. Locations and methods of disposal or recycling, including land-based alternatives
7. The effect on alternate uses of oceans, such as scientific study, fishing, and other living resource exploitation

The Secretary of the Army is responsible for issuing permits for the transportation and disposal of dredged material in ocean waters. The Secretary applies the same criteria for the issuance of permits as the EPA uses and will issue permits in consultation with the EPA. Permits issued by the EPA or the Corps of Engineers designate:

1. The type of material authorized to be transported for dumping or to be dumped
2. The amount of material authorized to be transported for dumping or to be dumped
3. The location where such transport for dumping will be terminated or where such dumping will occur
4. The length of time for which the permits are valid
5. Any special provisions

The other major provision of the act is the establishment of the Marine Sanctuaries program.

Enforcement Responsibilities; Federal–State Relationship. The EPA has responsibility for issuing and administering permits for the dumping of all materials (except for dredged material) into ocean waters. The

Corps of Engineers has responsibility for permits for the dumping of dredged or fill material in ocean waters. Each agency has issued regulations to control ocean dumping. The National Oceanic and Atmospheric Administration (NOAA) in the Department of Commerce is responsible for administering the Marine Sanctuaries Program and issuing regulations to implement it. The states in which a sanctuary is designated can stop the designation by certifying that the terms are unacceptable to the state.

Accomplishments and Impacts. In January 1982, the Department of Commerce released the “Program Development Plan” for the National Marine Sanctuary Program. In this program, emphasis is on the use of marine sanctuaries for both public and private concerns. This will be particularly evident in the exploitation of the areas for mineral resources. A greater participation by those states in which the sanctuaries are located is being fostered. This greater involvement on the part of affected states will also extend to the permitting process for the dumping of wastes and dredged material into ocean waters.

Toxic Substances Control Act (TSCA) (15 U.S.C. 2601 et seq.)

Enacted: 1976 (PL 94-469). Major amendments: 1986 (PL 99-419); 1988 (PL 100-551) (20).

Basic Objective. This act sets up the toxic substances program, which is administered by the EPA. If the EPA finds that a chemical substance may present an unreasonable risk to health or to the environment and that there are insufficient data to predict the effects of the substance, manufacturers may be required to conduct tests to evaluate the characteristics of the substance, such as persistence, acute toxicity, or carcinogenic effects. Also, the act establishes a committee to develop a priority list of chemical substances to be tested. The committee may list up to 50 chemicals that must be tested within 1 year. However, the EPA may require testing for chemicals not on the priority list.

Manufacturers must notify the EPA of the intention to manufacture a new chemical substance. The EPA may then determine if the data available are inadequate to assess the health and environmental effects of the new chemical. If the data are determined to be inadequate, the EPA will require testing. Most importantly, the EPA may prohibit the manufacture, sale, use, or disposal of a new or existing chemical substance if it finds the chemical presents an unreasonable risk to health or the environment. The EPA can also limit the amount of the chemical that can be manufactured and used and the manner in which the chemical can be used.

The act also regulates the labeling and disposal of polychlorinated biphenyls (PCBs) and prohibited their production and distribution after July 1979.

In 1986, Title II, “Asbestos Hazard Emergency Response,” was added to address issues of inspection and removal of asbestos products in public schools and to study the extent of, and response to, the public health danger posed by asbestos in public and commercial buildings.

Key Provisions. Testing is required on chemical substances meeting certain criteria to develop data with respect to the health and environmental effects for which there are insufficient data relevant to the determination that the chemical substance does or does not present an unreasonable risk of injury to health or the environment.

Testing includes identification of the chemical and standards for test data. Testing is required for the following:

1. Manufacturers of a chemical meeting certain criteria
2. Processors of a chemical meeting certain criteria
3. Distributors or persons involved in disposal of chemicals meeting certain criteria

Test data required by the act must be submitted to the EPA, identifying the chemical, listing the uses or intended uses, and listing the information required by the applicable standards for the development of test data.

The EPA will establish a priority list of chemical substances for regulation. Priority is given to substances known to cause or contribute to cancer, gene mutations, or birth defects. The list is revised and updated as needed.

A new chemical may not be manufactured without notification to the EPA at least 90 days before manufacture begins. The notification must include test data showing that the manufacture, processing, use, and disposal of the chemical will not present an unreasonable risk of injury to health or the environment. Chemical manufacturers must keep records for submission to the EPA as required. The EPA will use these reports to compile an inventory of chemical substances manufactured or processed in the United States.

The EPA can prohibit the manufacture of a chemical found to present an unreasonable risk of injury to health or the environment or otherwise restrict a chemical. The act also regulates the disposal and use and prohibits the future manufacture of PCBs, and requires the EPA to engage, through various means, in research, development, collection, dissemination, and utilization of data relevant to chemical substances.

Enforcement Responsibilities; Federal–State Relationship. The EPA has enforcement responsibilities for the act, but the act makes provisions for consultations with other federal agencies involved in health and environmental issues, such as OSHA and the Department of Health and Human Services. Initially, the states could receive EPA grants to aid them in establishing programs at the state level to prevent or eliminate unreasonable risks to health or the environment related to chemical substances.

Accomplishments and Impacts. TSCA has provided a framework for establishing that chemical manufacturers take responsibility for the testing of chemical substances as related to their health and environmental effects. It places the burden of proof on the manufacturer to establish the safety of a chemical, yet still gives the EPA the final authority to prohibit or severely restrict chemicals in commerce. Thus, it is an attempt at the introduction of a chemical to prevent significant health and environmental problems that may surface later on. The fact that when this legislation was initially passed, PCB effects were such an issue because of their widespread and uncontrolled use is reflective of public concerns over the number of other possible chemicals commonly used which could be carcinogenic. Public concern was so visible that an immediate need was perceived to regulate PCBs. Thus, PCBs are controlled and specifically prohibited by TSCA rather than RCRA.

National Environmental Policy Act (NEPA) (42 U.S.C. 4341 et seq.) 1969/70

The complete text of this legislation is presented in Fig. 10.1. Basic objectives and key provisions of the act are well defined in the language of this legislation (21). Please see Chapter 10 for further discussion of this act.

Enforcement Responsibilities; Federal–State Relationship. The President's Council on Environmental Quality (CEQ) has the main responsibility for overseeing federal efforts to comply with NEPA. In 1978, CEQ issued regulations to comply with the procedural provisions of NEPA. Other provisions of NEPA apply to major federal actions significantly affecting the quality of the human environment.

Accomplishments and Impacts. The enactment of this act has added a new dimension to the planning and decision-making process of federal agencies in the United States (22). This act requires federal agencies to assess the environmental impact of implementing their major programs and actions early in the planning process. For those projects or actions that are either expected to have a significant effect on the quality of the

human environment or are expected to be controversial on environmental grounds, the proponent agency is required to file a formal environmental impact statement (EIS) (22, 23). Other accomplishments and impacts of the act are:

- It has provided a systematic means of dealing with environmental concerns and including environmental costs in the decision-making process.
- It has opened governmental activities and projects to public scrutiny and public participation.
- Some projects have been delayed because of the time required to comply with the NEPA requirements.
- Many projects have been modified or abandoned to balance environmental costs with other benefits.
- It has served to accomplish the four purposes of the act as stated in its text.

National Historic Preservation Act (16 U.S.C. 470-470t)

Enacted: 1966 (PL 89-665). Major amendments: 1973 (PL 93-54); 1976 (PL 94-422); 1980 (PL 96-515); 1992 (PL 102-575) (24).

Basic Objective. The act establishes a national policy of preserving, restoring, and maintaining cultural resources. The Advisory Council on Historic Preservation is given responsibility under the act to implement this national policy. The National Register of Historic Places is the mechanism by which historic properties can be protected. Any property in the National Register or eligible for inclusion is protected from certain types of activities and can receive funding for restoration and maintenance. Amendments to the act gave the National Park Service the responsibility for determination of eligibility for inclusion in the National Register. State historic preservation officers are ultimately given enforcement responsibilities within the state.

Key Provisions

1. Regulations for determination of eligibility in the National Register of Historic Places
2. A federal agency must take into account the effect of a project on any property included in or eligible for inclusion in the National Register.
3. The advisory council must be given an opportunity to comment on a federal project.
4. Federal agencies must inventory all property and nominate any eligible properties to the National Register.
5. Federal agencies must provide for the maintenance of federally owned registered sites.
6. Agencies must coordinate projects with the state historic preservation officer of the state in which the project is located.
7. States can qualify for federal grants for the protection, restoration, and maintenance of properties in the National Register.

Enforcement Responsibilities; Federal–State Relationship. Enforcement responsibilities involve a triad of agencies. The Advisory Council on Historic Preservation is given the ultimate authority to comment on a federal project that may impact a property in or eligible for inclusion in the National Register. The National Park Service has responsibility for making determinations or eligibility. The state historic preservation officer has the final responsibility for protecting and maintaining eligible properties.

Accomplishments and Impacts. The greatest impact of the act has been the inclusion of cultural concerns in the environmental area. Federal agencies are including cultural assessments as part of the environmental assessment process. The act has served to highlight the national concern to preserve its cultural heritage in the form of the protection of historic sites, buildings, and properties.

The major accomplishment has been the publication of a list of protected sites in the National Register and the provision of funds to restore and maintain those sites for future generations. Many new projects in urban areas to be located in a historic district may be opposed by the community on the grounds of their adverse effects in terms of character, scale, or style of the historic district.

Wild and Scenic Rivers Act (16 U.S.C. 1271 et seq.)

Enacted: 1968 (PL 90-542). Major amendment: 1974 (PL 93-279) (25).

Basic Objective. This act establishes the Wild and Scenic River System. It essentially protects rivers designated for their wild and scenic values from activities that may adversely impact those values. It provides for a mechanism to determine if a river can meet certain eligibility requirements for protection as a wild and/or scenic river.

Key provisions. In planning for the use and development of water and land resources, federal agencies must give consideration to potential wild and scenic river areas. This potential must be discussed in all river basin and project plans submitted to Congress. No federal agency is allowed to assist in any way in the construction of a water resources project having a direct and adverse effect on the values for which a river was designated as part of the Wild and Scenic River System.

Likewise, no agency is allowed to recommend authorization or request appropriations to begin construction of a project on a designated river without informing the administering secretary (Secretary of the U.S. Department of Interior or Agriculture) in writing, 60 days in advance, and without specifically reporting to Congress on how construction would conflict with the act and affect values of the river being protected by the act.

No agency is permitted to recommend authorization of, or request appropriations to initiate, construction of a project, on or directly affecting a river designated for potential addition to the system during the full 3 fiscal years after the designation, plus 3 more years for congressional consideration, unless the Secretary of Interior or Agriculture advises against including the segment in the system in a report that lies before Congress for 180 in-session days. The comparable time limit for state-promised additions is 1 year.

Agencies must inform the Secretary of any proceedings, studies, or other activities which would affect a river that is designated as a potential addition to the system. Agencies having jurisdiction over lands that include, border upon, or are adjacent to any river within or under consideration for the system protect the river with management policies and plans for the lands as necessary.

Enforcement Responsibilities; Federal–State Relationship. The Department of the Interior has ultimate authority for administering the program, but the states can designate rivers for inclusion in the system. The Department of Agriculture administers and designates rivers in national forests.

Accomplishments and Impacts. As of July 1996, 160 rivers had been designated either wild, scenic, or recreational as part of the act. The Act has attempted to preserve designated rivers and their values from adverse impacts.

Coastal Zone Management Act (16 U.S.C. 141 et seq.)

Enacted: 1972 (PL 92-583). Major amendments: 1976 (PL 94-370); 1980 (PL 96-464); 1985 (PL 99-272); 1990 (PL 101-508); 1996 (PL 104-150) (26).

Basic Objective. The act was passed in response to the public's concern for balanced preservation and development activities in coastal areas. It was designed to help states manage these competing demands and provided funding to states participating in the federal program.

The legislation emphasized the state leadership in the program, and allowed states to participate in the federal program by submitting their own coastal zone management proposals. The purpose of these state programs, which are federally approved, is to increase protection of coastal areas while better managing development and government activities at all levels. The act established the Office of Coastal Zone Management (OCZM) in the National Oceanic and Atmospheric Administration (NOAA). Once the OCZM has approved a state program, federal agency activities within a coastal zone must be consistent "to the maximum extent practicable" with the program.

Key Provisions. Federal agencies must assess whether their activities will directly affect the coastal zone of a state having an approved program.

The 1980 amendments included, as part of coastal areas, wetlands, floodplains, estuaries, beaches, dunes, barrier islands, coral reefs, and fish and wildlife and their habitat. The act also provides public access to the coast for recreational purposes.

States are encouraged to prepare special area management plans addressing such issues as natural resources, coastal-dependent economic growth, and protection of life and property in hazardous areas. Federal grants are available to the states to cover 80% of the costs of administering their federally approved coastal zone management programs. They may use 30% of their grants to implement the 1980 amendment provisions.

The states are also encouraged to inventory coastal resources, designate those of "national significance," and establish standards to protect those so designated.

Enforcement Responsibilities; Federal-State Relationships. The act is administered by OCZM as part of the NOAA. However, the underlying objective of the act is to involve agencies at the state and local levels in the administering process. While the act does not require states to submit a coastal zone management program for approval, it does provide two major incentives for states to join the federal program; one is financial assistance to administer the program, and the other is that any federal activity in a coastal zone must include the consistency determination process, which involves consultation with the state.

Accomplishments and Impacts. Because the consistency determination is a major factor or incentive in encouraging states to participate in the coastal zone management program, it is imperative to clearly define when such a consistency determination is required. The act states that it is necessary when a federal activity will "directly affect" the coastal zone. Since 1979, NOAA has been attempting to define "directly affecting." The latest attempt in January, 1982, was withdrawn in May, 1982. Thus there is not yet a clear definition of this term.

The central issue is whether off-coast survey (OCS) activities by the Department of Interior are subject to consistency determinations. The recent extensive off-shore tracts opened for lease by the Secretary of the Interior serve to highlight the conflict between the federal government and affected states. At the present time, NOAA is waiting the outcome of litigation involving OCS activities before attempting a further definition of "directly affecting," although at the appeals court level, the court ruled in favor of including a specific lease in the consistency determination process. Thus, the two major incentives for encouraging states to participate in the program are currently in jeopardy.

Endangered Species Act (16 U.S.C. 1 531-1 542)

Enacted: 1973 (PL 93-205). Major amendments: 1978 (PL 95-632); 1982 (PL 97-304); 1988 (PL 100-478) (27).

Basic Objective. This act seeks to conserve endangered and threatened species. It directs the Fish and Wildlife Service of the U.S. Department of Interior to promulgate a list of endangered and threatened species and designate critical habitat for those species. Amendments also created the Endangered Species Committee to grant exemptions for the act.

Federal agencies must carry out programs for the conservation of listed species and must take actions to ensure that projects they authorize, fund, or carry out are not likely to jeopardize the existence of the listed species or result in the destruction or modification of habitat declared to be critical.

The act divides procedures for those projects begun before and after November 10, 1978. For those not under construction before November 10, 1978, agencies must request the Fish and Wildlife Service to furnish information as to whether any species listed, or proposed to be listed, are in the area. If such species are present, a biological assessment must be completed by the proponent agency within 180 days.

If the biological assessment or other project information reveals that a species may be affected, the agency must consult with the Fish and Wildlife Service (or National Marine Fisheries Service). Consultation must be completed by the service within a 90-day period. The Department of the Interior provides the agency with an opinion as to how the action will affect the species or its critical habitat, and suggests reasonable alternatives. The agency may apply for an exemption to the act to the Endangered Species Committee.

Key Provisions. Of major significance is the promulgation of a list of species that have been found to be either threatened or endangered and the protection of species on the list from activities that may impact their continued protection and survival. Also, the act provides for the designation of habitat from activities that may harm the delicate ecological balance necessary for the existence of a listed species. Federal agencies are required to perform a biological assessment before undertaking a project to determine the impact of a project on a listed species or its habitat. If that impact is negative, the agency must undertake mitigation procedures or the project must be halted. An important provision of the act is the establishment of an Endangered Species Committee to grant exemptions from the act. A federal agency must consult with the Fish and Wildlife Service if the results of the biological assessment show a listed species may be affected by a project. The Fish and Wildlife Service will suggest alternatives to the agency. A process is established whereby a species can be determined to be threatened or endangered, and thus eligible for the list, or cannot be removed from the list.

Enforcement Responsibilities; Federal–State Relationship. The Fish and Wildlife Service of the Department of Interior has enforcement responsibilities under the act and must ultimately decide on all biological assessments and mitigation procedures. While states can compile their own lists of species and the degrees of protection required, species on the federal list are under the jurisdiction and protection of the federal government, and a violation of the act carries federal penalties.

Accomplishments and Impacts. The Endangered Species Act has served to stop the rapid rate of extinction of many species. Perhaps the greatest success has been with the bald eagle, which is making a successful return, largely due to its protection under the act. Perhaps the most visible of its impacts was the halting of a major water project, the Tellico Dam, due to its impact on a listed species. The result of that action and the result of the Supreme Court decision was the 1978 amendment establishing the Endangered Species Committee, which can grant exemptions from the act.

For many of the species listed, it is too late to prevent ultimate extinction, but for others, such as the bald eagle, the grizzly bear, and the alligator, the act has protected the species and its habitat to allow for its survival.

Fish and Wildlife Coordination Act (16 U.S.C. 661 et seq.)

Enacted: 1934 (Ch. 55, 48 Stat. 401). Major amendments: 1946 (Ch. 965, 60 Stat. 1080); 1958 (PL 85-624); 1965 (PL 89-72) (28).

Basic Objective. This act provides that wildlife conservation be given equal consideration and be coordinated with other aspects of water resource development programs. It establishes the need to coordinate activities of federal, state, and private agencies in the development, protection, and stocking of wildlife resources and their habitat. Also, the act provides procedures for consultation between agencies with the purpose of preventing loss of and damage to wildlife resources from any water-resource-related project. Any such consultation includes the Fish and Wildlife Service, the head of the agency having administrative control of state and wildlife resources, and the agency conducting the project.

Key Provisions. The act requires officers of the agency conducting the project to give full consideration to Fish and Wildlife Service recommendations and those of the state agency. "Full consideration" includes mitigation measures.

Any report recommending authorization of a new project must contain an estimate of wildlife benefits and losses and the costs and amount of reimbursement required. Adequate provision must be made for the use of projects lands and water for the conservation, maintenance, and management of wildlife resources, including their development and improvement.

Lands to be measured by a state for the conservation of wildlife must be managed in accordance with a plan that must be jointly approved by the federal agency exercising primary administrative responsibility, the Secretary of the Interior, and the administering state agency.

Enforcement Responsibilities; Federal–State Relationship. Enforcement of the act involves the triad coordination of the Department of Interior (Fish and Wildlife Service), the state wildlife agency, and the agency undertaking the water-resource-related project.

Accomplishments and Impacts. The Department of the Interior has issued regulations to implement the act. Under the initial regulations, federal agency consultation procedures would be consistent. However, in July, 1982, the proposed regulations were withdrawn in favor of administrative actions preparing memoranda of agreement. Likewise, the Department of the Interior has withdrawn from attempts to establish procedures for mitigation techniques. Thus, federal agencies are left with the requirement to coordinate with state agencies and with the Fish and Wildlife Service but are under no legal mandate to implement recommendations arising from such consultation.

Pollution Prevention Act of 1990 (PL 101-508, Title VI, Subtitle F. Sections 6601-6610)

Basic Objective. Traditionally, environmental legislation in the United States has focused on an end-of-pipe-control approach for minimizing discharge of pollutants to the environment. By using this approach, considerable progress has been made in reducing the total discharge of pollutants to the environment. However, this often has resulted in transferring pollutants from one medium to the other and in many cases is not cost-effective. The basic objective of the Pollution Prevention Act (29) is to establish a national policy of preventing or reducing pollution at the source wherever feasible, and it directs the federal EPA to undertake certain steps in that regard. Prior to this act, RCRA Hazardous and Solid Waste Amendments of 1984 had established a program of waste minimization through many provisions including requiring large-quantity generators to certify on their waste manifests that they have a program in place to minimize the amount and toxicity of wastes generated to the extent economically feasible.

Key Provisions. The Pollution Prevention Act of 1990 established as national policy the following waste management hierarchy:

1. Prevention. The waste management priority is to prevent or reduce pollution at the source whenever feasible.
2. Recycling. Where pollution cannot be prevented, it should be recycled in an environmentally safe manner whenever feasible.
3. Treatment. In the absence of feasible prevention and recycling, pollution should be treated to applicable standards prior to release or transfers.
4. Disposal. Only as a last resort are wastes to be disposed of safely.

The Pollution Prevention Act further directed the EPA to:

1. Establish a prevention office independent of the agency's single-medium program offices (EPA added pollution prevention to the existing function of Assistant Administrator for Pesticides and Toxic Substances). Congress appropriated \$8 million for each of the fiscal years 1991, 1992, and 1993 for the new office to fulfill the functions delineated in the act.
2. Facilitate the adoption by business of source-reduction techniques by establishing a source-reduction clearinghouse and a state matching grants program. Congress further appropriated 58 million for each of the fiscal years 1991, 1992, and 1993 for state grants, with a 50% state match requirement.
3. Establish a training program on source-reduction opportunities for state and federal officials working in all agency program offices.
4. Identify opportunities to use federal procurement to encourage source reduction.
5. Establish an annual award program to recognize a company or companies that operate outstanding or innovative source reduction programs.
6. Issue a biennial status report to Congress (initial report to be issued October-November 1992).
7. Require an annual toxic chemicals source reduction and recycling report for each owner or operator of a facility already required to file an annual toxic chemical release form under section 313 of the Superfund Amendments and Reauthorization Act (SARA) of 1986.

The EPA is pursuing integrating pollution prevention into all its programs and activities and has developed unique voluntary reduction programs with public and private sectors. The EPA 33/50 program, through voluntary enrollment and direct action by industry, seeks to reduce the generation of high-priority wastes from a target group of 17 toxic chemicals by 50% by 1995, with an interim goal of 33% reduction by 1992. This reduction is measured against a 1988 baseline.

Enforcement Responsibilities; Federal–State Relationship. EPA conducts a yearly audit of major users of toxic substances and producers of toxic wastes. “The purpose of the audits is to determine:

- whether there are better and less environmentally damaging ways to complete the task without use of toxic substances
- whether there are ways to minimize the production of toxic wastes
- whether there are ways to recycle the toxic substances
- and who is regulated” (30)

Again, the federal government's statutes take precedence over state statutes. The act also pledges federal assistance to states (up to 50%) with pollution prevention programs under the act.

Accomplishments and Impacts. “The primary purpose of the Pollution Prevention Act is to discourage the disposal of recyclable toxic substances” (30). The Act focuses on industry, government, and public attention on reducing the amount of pollution through cost-effective changes in production, operation, and raw mate-

rials use (31). “Opportunities for source reduction are often not realized because of existing regulations, and the industrial resources required for compliance, focus on treatment and disposal. Source reduction is fundamentally different and more desirable than waste management or pollution control” (31).

“Pollution prevention also includes other practices that increase efficiency in the use of energy, water or other natural resources, and protect our resource base through conservation. Practices include recycling, source reduction, and sustainable agriculture” (31).

TRENDS IN ENVIRONMENTAL LEGISLATION AND REGULATIONS

The 1970s was a decade of extensive new federal legislation covering all spheres of environmental concern. The pace of new initiatives seems certain to slow considerably, barring a significant crisis or disaster that might spark new legislative initiatives. In the 1980s, emphasis was directed toward refining existing legislation and fine tuning current regulatory and enforcement policies. In the 1990s emphasis has been on balancing economic and environmental costs and on pollution prevention.

New Legislation

Concern for protecting the quality of groundwater resources, casually expressed in the Clean Water Act, and more forcibly articulated by RCRA, is likely to be the focus of new environmental legislation in the near future. Groundwater supplies water to a considerable percent of our population, which is accustomed to withdrawing water from nature and using it without treatment, except perhaps for softening. Recently, it has become widely known that groundwater supplies are very vulnerable to permanent damage due to seepage from chemical waste disposal sites. Often pollutants are persistent trace organics that defy treatment with conventional technology at affordable costs.

Experience and expertise developed in response to RCRA groundwater requirements will have to be expanded greatly to provide the degree of protection and capability for corrective action that is likely to be called for.

Another major initiative stemming from concern for the disposal of hazardous wastes will revolve around limitations on land disposal of such wastes. States have begun the process that will likely ban disposal of certain kinds of wastes that can be shown to be treated and handled by alternate methods. Federal initiatives in the form of an amendment to RCRA are likely to establish national limitations on land disposal of certain kinds of waste.

Another high-profile environmental issue is the protection of wetlands in the near term and possible restoration and creation of wetlands in the future. Key issues for forthcoming legislation will be changes in Section 404 of the Clean Water Act, designation of a single, lead federal agency, and delegation procedures for states with approved plans to have primary responsibility for planning and permitting wetland protection. For the most part, protection refers to actions to prevent destruction of wetlands. Unless man-made for wastewater treatment, wetlands in the United States are protected from pollution damage by the Clean Water Act.

In other areas, legislation and regulations will continue to evolve to address air pollution, such as global warming, ozone depletion, acid rain, and indoor air quality. Water supply and water pollution issues that will become important in the future include nonpoint (and stormwater) controls, and effective use of water resources management practices, i.e., allocations for withdrawal and for waste assimilation. Medical and infectious wastes are newer public issues in the management of solid and hazardous wastes.

Nuclear waste management, always a controversial and emotional issue, is likely to create major environmental and economic problems for society. Regulations for effective nuclear waste management are likely to be made more stringent.

Balancing Federal and Nonfederal Roles

The legislation of the 1970s and the implementing regulations were structured largely on the basis of a very dominant federal role in environmental protection. This balance is shifting in the 1980s as part of an overall change in federal government policy that is transferring much of the regulatory enforcement responsibilities to the states.

For a number of years, popular rhetoric of state and local agencies expressed a desire for more say in environmental affairs. Along with reduced federal direction, fewer federal dollars are earmarked for the federal share in implementing environmental protection programs. In fact, it is the desire to lower federal expenditures that is driving and decreasing federal involvement in environmental programs and not a basic philosophical shift in how government affairs can best be conducted on behalf of the populace.

Reduced federal financial support, however, is not part of the package previously espoused by state and local politicians. In fact, several states have voiced objections to taking over the administration and enforcement of certain environmental programs if federal financial support drops below a certain threshold level.

Balancing Economic and Environmental Costs

The common theme of the environmental movement is that good environmental quality is good for the economy in the long run. The short-run economic dislocation problems with this philosophy were largely ignored in the 1970s. Corporations and municipalities were expected to pay whatever was needed to correct past environmental problems and to provide future environmental protection, no matter the price. Federal laws and regulations established ambitious compliance schedules, which were occasionally relaxed, but which for the most part committed industry and public to considerable expenditures.

Opposition to spend what it takes was often stated ineffectively, mostly because the arguments advanced tended to overstate the problem. Too often, decisions to close companies or shut down plants were attributed solely to the cost of environmental regulations. No doubt these were important factors and may have been the sole factor in several instances, but not to the extent that was often claimed.

The national priority of the 1980s was renewal of the economy and a trend toward cost-effective regulations. There is, and will be, a requirement on regulators and enforcers to collect the facts before imposing major and costly requirements. The philosophy of the 1970s was that all potential problems imaginable had to be prevented. Now, it is recognized that the possibilities that could be safeguarded are too numerous for this approach to be affordable.

Another manifestation of the recognition that priority must be given to the economy will be reduced paperwork requirements for industry. A theme of the environmental regulators in the 1970s was that industry was a source of extensive potential useful and interesting information on a variety of environmental topics and that a proper role for the regulator was to ask industry to submit such information whether or not immediate need was apparent. This placed an extensive and costly burden on industry that is gradually being relaxed.

In summary, the trends toward environmental regulations and environmental protection can be stated as follows:

1. Adjustments in the federal and nonfederal roles are likely to increase state participation in the enforcement and administration of environmental regulations.
2. Balancing of economic and environmental goals is likely to take the form of moderation in achieving some environmental goals that adversely affect economic activities.
3. Public support for environmental protection and related life-support systems is expected to continue, especially in the industrialized countries.
4. In the United States, midcourse correction to major environmental legislation is expected to be made by

the legislative bodies. This midcourse correction will be based upon benefits (environmental protection and enhancement) and costs associated with environmental requirements.

5. To the extent possible, regulations will move away from the command-and-control type approach presently used in most cases. The reason being that in a free-market economy, command-and-control regulations are inefficient and do not preserve elements of voluntary choice. To the extent technologically practical, future regulatory approaches will focus on the use of economic incentives, such as marketable discharge licenses or permits and effluent charges.
6. With increasing experience in the pollution control technology areas, regulatory controls will move away from the “hothouse” types of control technologies that deteriorate rather quickly and end up contributing large amounts of pollutants and incurring high operation and maintenance costs during the life cycle of the control devices. Instead, more practical emission standards, with built-in economic incentives, will be established so that cost-effective pollution control technology that provides overall lower pollutants during the life cycle of the equipment could be used.
7. More emphasis will be placed on new concepts such as pollution prevention, industrial ecology, and sustainable development.
8. Many problems of the global commons, such as acid rain, global warming, deforestation, and biodiversity, will become issues of international concern and international cooperation.
9. Industrialization in developing countries and continued population increases will further adversely affect environmental quality, especially in the developing countries.
10. Vigorous public support for incorporating environmental concerns into the decision-making process, as embodied in the provisions of legislation such as the National Environmental Policy Act (NEPA), is expected to continue.

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CHAPTER 3

AIR AND WATER QUALITY STANDARDS

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In the United States, the national goals for air and stream water quality to protect public health and welfare and environmental resources are established by the Clean Air Act and the Clean Water Act. The Safe Drinking Water Act and its regulations are solely for public health protections; they are the final regulatory barrier for protection of public drinking water and, as such, include standards for contaminants of concern and identify contaminants requiring further study.

Since the relationship between health and air quality is less understood than that between health and water quality, the Clean Air Act and the Clean Water Act are not parallel. The approach of the Clean Air Act is to determine the relationship between public health and air quality while restoring, maintaining, and improving the air environment. The Clean Water Act moves forward, based on the accomplishments made and knowledge gained, to achieve the ultimate goal of clean, fishable, swimmable waters devoid of pollutants and toxicants.

This legislation requires the use of quality standards and criteria for control of pollutants in the environment. Two approaches are used to set standards to restore, maintain, and improve the quality of the air and water environments. One approach establishes environmental or ambient conditions, such as air quality or stream water quality, and the second stipulates discharge limitations. Discharge limits are developed based on emission or effluent limitations for the desired air or water quality or based on available technology.

AIR QUALITY OBJECTIVES

The goal of the Clean Air Act is to protect the public health and welfare from the harmful effects of air pollutants. To this end, national ambient air quality standards were developed as a quality reference point. Also, state implementation plans were required to manage and regulate the provisions of the act, including permitting. Table 3.1 provides a reference list of U.S. Environmental Protection Agency (EPA) regulations for implementing the Clean Air Act.

Ambient Air Quality Standards

Congress mandated that the EPA promulgate national ambient air quality standards as maximum levels of selected pollutants that would lead to unacceptable air quality. These numerical standards were to be based

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TABLE 3.1 Air Program Regulations

40 CFR Part	Topic
50	National primary and secondary ambient air quality standards
51	Requirements for preparation, adoption, and submittal of implementation plans
52	Approval and promulgation of implementation plans
53	Ambient air monitoring reference and equivalent methods
54	Prior notice of citizen suits
55	Outer continental shelf air regulations
56	Regional consistency
57	Primary nonferrous smelter orders
58	Ambient air quality surveillance
59	[Reserved]
60	Standards of performance for new stationary sources
61	National emission standards for hazardous air pollutants
62	Approval and promulgation of state plans for designated facilities and pollutants
63	National emission standards for hazardous air pollutants for source categories
66	Assessment and collection of noncompliance penalties by EPA
67	EPA approval of State noncompliance penalty program
68	Chemical accident prevention provisions
69	Special exemptions from requirements of the Clean Air Act
70	State operating permit programs
71	Federal operating permit programs
72	Permits regulation
73	Sulphur dioxide allowance system
74	Sulfur dioxide opt-ins
75	Continuous emission monitoring
76	Acid rain nitrogen oxides emission reduction program
77	Excess emissions
78	Appeal procedures for acid rain program
79	Registration of fuels and fuel additives
80	Regulation of fuels and fuel additives
81	Designation of areas for air quality planning purposes
82	Protection of stratospheric ozone
85	Control of air pollution from motor vehicles and motor vehicle engines
86	Control of air pollution from new and in-use motor vehicles and new and in-use motor vehicle engines: Certification and test procedures
87	Control of air pollution from aircraft and aircraft engines
88	Clean-fuel vehicles
89	Control of emissions from new and in-use nonroad engines
90	Control of emissions from nonroad spark-ignition engines
91–92	[Reserved]
93	Determining conformity of federal actions to state or federal implementation plans
94	[Reserved]
95	Mandatory patent licenses
96–99	[Reserved]

on background studies that included control technology, costs, energy requirements, emission reduction benefits, and environmental impacts.

The national primary ambient air quality standards are judged necessary, with an adequate margin of safety, to protect public health. Secondary standards were developed to protect the public welfare from any known or anticipated adverse effects of a pollutant, such as impaired vision or damage to buildings and life-forms.

The pollutant levels of the national primary and secondary ambient air quality standards are presented in Table 3.2. The reference condition for these standards is a temperature of 25°C and a pressure of 760 mm Hg.

The promulgation of the standards stated that they should not in any way be considered to allow significant deterioration of an area's existing air quality. Furthermore, the states may establish statewide or regional ambient air quality standards that are more stringent than the national standards.

In 1997, the ambient air quality standards were revised to include particulate matter that is 2.5 microns or less in diameter (PM_{2.5}). While the standards for particulate matter that is 10 microns or less in diameter (PM₁₀) retain the same values as the prior National Ambient Air Quality Standards, their forms are new. Previously, the particulate material applied to the highest 24-hour or annual averages found within a planning area, and monitoring networks were often designed to measure these highest values. These networks did not

TABLE 3.2 National Primary and Secondary Ambient Air Quality Standards (*I*)

Pollutant	Air Quality Standard		
Sulfur Dioxide			
Primary	80 $\mu\text{g}/\text{m}^3$	0.03 ppm	Annual arithmetic mean
	365 $\mu\text{g}/\text{m}^3$	0.14 ppm	Maximum 24-h concentration not to be exceeded more than once per year
Secondary	1300 $\mu\text{g}/\text{m}^3$	0.5 ppm	Maximum 3-h concentration not to be exceeded more than once per year
Particulate Matter			
PM _{2.5}	150 $\mu\text{g}/\text{m}^3$	—	Annual arithmetic mean, 3-year averaging, spatial averaging
	65 $\mu\text{g}/\text{m}^3$	—	24-h 98th percentile, 3-year averaging, at each monitor
PM ₁₀	50 $\mu\text{g}/\text{m}^3$	—	Annual arithmetic mean
	150 $\mu\text{g}/\text{m}^3$	—	24-h 99th percentile, averaged over 3 years
Carbon Monoxide			
Primary and secondary	10,000 $\mu\text{g}/\text{m}^3$	9 ppm	8-h average concentration not to be exceeded more than once per year
	40,000 $\mu\text{g}/\text{m}^3$	35 ppm	1-h average concentration not to be exceeded more than once per year
Ozone			
Primary and secondary	157 $\mu\text{g}/\text{m}^3$	0.08 ppm	8-h average concentration, 3-year average of annual fourth-highest daily maximum 8-h average
Nitrogen dioxide			
Primary and secondary	100 $\mu\text{g}/\text{m}^3$	0.53 ppm	Annual arithmetic mean
Lead			
Primary and secondary	1.5 $\mu\text{g}/\text{m}^3$	—	Maximum arithmetic mean averaged over a calendar quarter

necessarily represent the overall exposure of populations to excessive particulate matter concentrations. Some data from these networks were disregarded by epidemiologists as being unrelated to health indicators, such as hospital admissions and death. The new forms for these standards are intended to provide more robust measures for the particulate indicator. The new PM_{2.5} standards will enhance the value of compliance measurements for evaluating health effects.

The implementation steps for the revised PM_{2.5} and PM₁₀ ambient standards include:

- PM_{2.5} and PM₁₀ standards promulgated by the EPA
- PM_{2.5} compliance networks installed and operating and PM₁₀ networks revised. Many PM₁₀ compliance sites will be discontinued in favor of new PM_{2.5} sites.
- Metropolitan Planning Areas (MPA) defined and designated as unclassifiable with respect to PM_{2.5} and PM₁₀ standards. MPAs must be defined for Metropolitan Statistical Areas (MSAs) containing more than 200,000 inhabitants. States may define additional MPAs with smaller populations. Areas with existing or pending PM₁₀ State Implementation Plans (SIPs) are obligated to implement the measures in those plans.
- Collect compliance data and conduct special studies. Compliance data are PM_{2.5} and PM₁₀ measurements with Federal Reference Methods or Federal Equivalent Methods. Fifty sites in the United States will acquire PM_{2.5} samples amenable to chemical characterization for elements, ions, and carbon year after year. Two hundred and fifty sites will acquire chemical data for shorter time periods and may be moved from one area to another.
- Five-year evaluation of particulate matter health criteria completed by the EPA. By presidential order, no planning areas will be declared in nonattainment until the technical basis for the new standards is reevaluated in light of new research.
- Presuming the current PM standards are justified by the reevaluation, planning areas exceeding the standards will be assigned attainment or nonattainment status.
- SIPs are formulated and submitted to demonstrate how planned emissions reductions will bring an area into attainment of the standard. SIPs will be based on measured concentrations and source apportionment studies.
- Emissions reduction measures are implemented and attainment is demonstrated by measured PM_{2.5} and PM₁₀ concentrations below the National Ambient Air Quality Standards levels.

The regulatory schedule to implement these standards is on par with the three- to ten-year schedule needed to extract the relevant science from health effects and field studies. Of particular note is that nonattainment areas will not be designated prior to a reevaluation of the scientific basis for a particulate standard. This provides business, governmental, and scientific groups with sufficient time to complete experiments that will better elucidate the relationships between emissions, human exposures to particles, and adverse health effects.

Air Quality Regions

Regions within a state are designated as either attainment or nonattainment areas. When the air quality exceeds national ambient air quality standards, the region is an "attainment area." The designations are pollutant-specific, which means an area may fall into both categories for different pollutants.

Permits are issued to new or modified existing major sources in attainment areas. These permits are part of the "prevention of significant deterioration" program and apply to facilities expected to emit over 100 tons per year of national ambient air quality standard pollutants or 250 tons per year of other pollutants following application of emission controls.

Also, the states are required to issue permits for controlling emissions from major stationary sources in nonattainment areas. The principal considerations for issuance of the permit are

1. A net reduction in the total relevant pollutant emissions by regulation of a source or areawide controls
2. Compliance with the lowest achievable emission rate
3. Compliance of all sources owned or operated by the applicant in the state with applicable state and federal regulations
4. Determination that the applicable implementation plan is being carried out for the subject nonattainment area

EMISSION STANDARDS

Congress also required the EPA to develop emissions standards as a means for future controls of air pollution. In particular, there are standards on new source performance and hazardous pollutant controls.

New Source Performance Standards

The Clean Air Act established the mechanism for promulgation of standards of performance for new stationary sources of air pollution. Table 3.3 provides the regulation reference for the source categories that are judged to contribute significantly to air pollution with respect to public health or welfare.

The new source performance standards are set for specific processes and apply to all new processes of that type and to existing sources that are substantially modified or rebuilt. The regulations also specify test standards that apply nationwide. On the state level, existing facilities must meet a state-promulgated standard pursuant to requirements of the Clean Air Act.

Hazardous Air Pollutant Standards

National emission standards for hazardous air pollutants apply to new and existing sources. The selected pollutants are those which may cause or contribute to increased mortality or an increase in serious irreversible or incapacitating reversible illness. Table 3.4 lists substances designated as hazardous air pollutants by the EPA. The pollutants and source categories promulgated as national emission standards are presented in Table 3.5.

DRINKING WATER QUALITY STANDARDS

The Safe Drinking Water Act, as amended in 1996, has numerous provisions directed to the identification and treatment of contaminants that may endanger public health and welfare. Three important drinking water quality regulations for both biological and chemical contaminants are the:

1. Primary Drinking Water Standards (*13*) specifying enforceable minimum contaminant levels for protection of related human health
2. Secondary Drinking Water Standards (*14*) identifying nonenforceable guidelines for minimum contaminant levels oriented to protection of aesthetic values
3. Contaminant Candidate List (*15*) designating contaminants for study that may require regulation in the future

TABLE 3.3 Standards of Performance for New Stationary Sources (2)

40 CFR 60 Subpart	Source
A	General Provisions
B	Adoption and Submittal of State Plans for Designated Facilities
C	Emissions Guidelines and Compliance Times
Cb	Emissions Guidelines and Compliance Schedules for Municipal Waste Combustors
Cc	Emissions Guidelines and Compliance Schedules for Municipal Waste Landfills
Cd	Emissions Guidelines and Compliance Schedules for Sulfuric Acid Production Units
D	Fossil-Fuel Fired Steam Generators (Construction Commenced after 8-17-71)
Da	Electric Utility Steam Generating Units (Construction Commenced after 9-18-78)
Db	Industrial–Commercial–Institutional Steam Generating Units
Dc	Small Industrial–Commercial–Institutional Steam Generating Units
E	Incinerators
Ea	Municipal Waste Combustors (Construction between 12-20-89 and 9-20-94)
Eb	Municipal Waste Combustors (Construction after 9-20-94)
F	Portland Cement Plants
G	Nitric Acid Plants
H	Sulfuric Acid Plants
I	Asphalt Concrete Plants
J	Petroleum Refineries
K	Storage Vessels for Petroleum Liquids (Construction Commenced after 6-11-73 and Prior to 5-19-78)
Ka	Storage Vessels for Petroleum Liquids (Construction Commenced after 5-18-78 and Prior to 7-23-84)
Kb	Volatile Organic Liquid Storage Vessels (Construction Commenced after 7-23-84)
L	Secondary Lead Smelters
M	Secondary Brass and Bronze Ingot Production Plants
N	Primary Emissions from Basic Oxygen Process Furnaces (Construction Commenced after 6-11-73)
Na	Secondary Emissions From Basic Oxygen Process Steelmaking Facilities (Construction Commenced After 1-20-83)
O	Sewage Treatment Plants
P	Primary Copper Smelters
Q	Primary Zinc Smelters
R	Primary Lead Smelters
S	Primary Aluminum Reduction Plants
T	Phosphate Fertilizer Industry: Wet-Process Phosphoric Acid Plants
U	Phosphate Fertilizer Industry: Superphosphoric Acid Plants
V	Phosphate Fertilizer Industry: Diammonium Phosphate Plants
W	Phosphate Fertilizer Industry: Triple Superphosphate Plants
X	Phosphate Fertilizer Industry: Granular Triple Superphosphate Storage Facilities
Y	Coal Preparation Plants
Z	Ferroalloy Production Facilities
AA	Steel Plants: Electric Arc Furnaces
AA ^a	Steel Plants: Electric Arc Furnaces and Argon–Oxygen Decarburization Vessels (Constructed After 8-17-83)
BB	Kraft Pulp Mills
CC	Glass Manufacturing Plants
DD	Grain Elevators
EE	Surface Coating of Metal Furniture
GG	Stationary Gas Turbines
HH	Lime Manufacturing Plants
KK	Lead–Acid Battery Manufacturing Plants
LL	Metallic Mineral Processing Plants

TABLE 3.3 Standards of Performance for New Stationary Sources (2) (*continued*)

40 CFR 60 Subpart	Source
MM	Automobile and Light-Duty Truck Surface Coating Operations
NN	Phosphate Rock Plants
PP	Ammonium Sulfate Manufacture
QQ	Graphic Arts Industry: Publication Rotogravure Printing
RR	Pressure Sensitive Tape and Label Surface Coating Operations
SS	Industrial Surface Coating: Large Appliances
TT	Metal Coil Surface Coating
UU	Asphalt Processing and Asphalt Roofing Manufacture
VV	Equipment Leaks of VOC in the Synthetic Organic Chemicals Manufacturing Industry
WW	Beverage Can Surface Coating Industry
XX	Bulk Gasoline Terminals
AAA	New Residential Wood Heaters
BBB	Rubber Tire Manufacturing Industry
DDD	VOC Emissions from the Polymer Manufacturing Industry
FFF	Flexible Vinyl and Urethane Coating and Printing
GGG	Equipment Leaks of VOC in Petroleum Refineries
HHH	Synthetic Fiber Production Facilities
III	VOC Emissions from the Synthetic Organic Chemical Manufacturing Industry Air Oxidation Unit Processes
JJJ	Petroleum Dry Cleaners
KKK	Equipment Leaks of VOC From Onshore Natural Gas Processing Plants
LLL	Onshore Natural Gas Processing: SO ₂ Emissions
OOO	Nonmetallic Mineral Processing Plants
PPP	Wool Fiberglass Insulation Manufacturing Plants
NNN	VOC Emissions from the Synthetic Organic Chemical Manufacturing Industry Distillation Operations
QQQ	VOC Emissions from Petroleum Refinery Wastewater Treatment
RRR	VOC Emissions from Synthetic Organic Chemical Manufacturing Industry Reactor Processes
SSS	Magnetic Tape Coating Facilities
TTT	Industrial Surface Coating: Surface Coating of Plastic Parts for Business Machines
UUU	Calciners and Dryers in Mineral Industries
VVV	Polymeric Coating of Supporting Substrate Facilities
WWW	Municipal Solid Waste Landfills

Primary Drinking Water Standards

Primary drinking water standards are established by EPA for microbiological and chemical contaminants that may be found in drinking water and could have adverse effects on humans. The National Primary Drinking Water Regulations and the National Primary Drinking Water Regulations Implementation are set forth under 40CFR141 and 40CFR142, respectively.

The regulated contaminants, MCLG, MCL, potential health effects from ingestion of water, and sources of contaminant in drinking water are listed in Table 3.6. The maximum contaminant level goals (MCLGs) are the maximum levels of a contaminant in drinking water at which no known or anticipated adverse effect on human health would occur and allow for a margin of safety. MCLGs are not enforceable. However, the maximum contaminant levels (MCLs), which are the maximum allowable concentration of the contaminant, are enforceable and are mandatory for all public water supplies.

TABLE 3.4 Hazardous Air Pollutants (3)

Acrylonitrile	Hexachlorocyclopentadiene
Asbestos	Inorganic arsenic
Benzene	Manganese
Beryllium	Methyl chloroform
1,3-Butadiene	Methylene chloride
Cadmium	Mercury
Carbon tetrachloride	Nickel
Chlorinated benzenes	Perchloroethylene
Chlorofluorocarbon	Phenol
Chloroform	Polycyclic organic matter
Chloroprene	Radionuclides
Chromium	Toluene
Coke oven emissions	Trichloroethylene
Copper	Vinyl chloride
Epichlorohydrin	Vinylidene chloride
Ethylene dichloride	Zinc and zinc oxide
Ethylene oxide	

TABLE 3.5 National Emission Standards for Hazardous Air Pollutants (3)

40 CFR 61 subpart	Emission
B	Radon-222 Emissions from Underground Uranium Mines
C	Beryllium
D	Beryllium Rocket Motor Firing
E	Mercury
F	Vinyl Chloride
H	Radionuclide Emissions from Department of Energy (DOE) Facilities
I	Radionuclide Emissions from Facilities. Licensed by the Nuclear Regulatory Commission (NRC) and Federal Facilities other than Department of Energy
J	Equipment Leaks (Fugitive Emission Sources) of Benzene
K	Radionuclide Emissions from Elemental Phosphorus Plants
L	Benzene Emission from Coke By-Product Recovery Plants
M	Asbestos
N	Inorganic Arsenic Emissions from Glass Manufacturing Plants
O	Inorganic Arsenic Emissions from Primary Copper Smelters
P	Inorganic Arsenic Emissions from Arsenic Trioxide and Metallic Arsenic Production Facilities
Q	Radon Emissions form Department of Energy Facilities
R	Radon Emissions form Phosphogypsum Stacks
T	Radon Emissions from Disposal of Uranium Mill Tailings
V	Equipment Leaks (Fugitive Emission Sources)
W	Radon-222 Emissions from Licensed Uranium Mill Tailings
Y	Benzene Emissions form Benzene Storage Vessels
BB	Benzene Emissions from Benzene Transfer Operations
FF	Benzene Waste Operations

TABLE 3.6 National Primary Drinking Water Standards (13)

Contaminants	MCLG (mg/L)	MCL (mg/L)	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Fluoride	4.0	4.0	Skeletal and dental fluorosis	Natural deposits; fertilizer, aluminum industries; water additive
Volatile Organics				
Benzene	zero	0.005	Cancer	Some foods; gas, drugs, pesticides, paint, plastic industries
Carbon Tetrachloride	zero	0.005	Cancer	Solvents and their degradation products
<i>p</i> -Dichlorobenzene	0.075	0.075	Cancer	Room and water deodorants, and "mothballs"
1,2-Dichloroethane	zero	0.005	Cancer	Leaded gasoline, fumigants, paints
1,1-Dichloroethylene	0.007	0.007	Cancer	Plastics, dyes, perfumes, paints
Trichloroethylene	zero	0.005	Cancer	Textiles, adhesives, and metal degreasers
1,1,1-Trichloroethane	0.2	0.2	Liver, Nervous system effects	Adhesives, aerosols, textiles, paints, inks, metal degreasers
Vinyl Chloride	zero	0.002	Cancer	May leach from PVC pipe; formed by solvent break down
Coliform and Surface Water Treatment				
<i>Giardia lamblia</i>	zero	TT	Gastroenteric disease	Human and animal fecal waste
<i>Legionella</i>	zero	TT	Legionnaire's disease	Indigenous to natural waters; can grow in water heating systems
Standard Plate Count	N/A	TT	Indicates water quality, effectiveness of treatment	
Total Coliform*	zero	<5%+	Indicates gastroenteric pathogens	Human and animal fecal waste
Turbidity*	N/A	TT	Interferes with disinfection filtration	Soil runoff
Viruses	zero	TT	Gastroenteric disease	Human and animal fecal waste
Inorganics				
Antimony	0.006	0.006	Cancer	Fire retardants, ceramics, electronics, fireworks, solder
Asbestos (>10m)	7MFL	7MFL	Cancer	Natural deposits; asbestos cement in water systems
Barium*	2	2	Circulatory system effects	Natural deposits, pigments, epoxy sealants, spent coal
Beryllium	0.004	0.004	Bone, lung damage	Electrical, aerospace, defense industries
Cadmium*	0.005	0.005	Kidney effects	Galvanized pipe corrosion; natural deposits; batteries, paints
Chromium* (total)	0.1	0.1	Liver, kidney, circulatory disorders	Natural deposits; mining, electroplating, pigments
Cyanide	0.2	0.2	Thyroid, nervous system damage	Electroplating, steel, plastics, mining, fertilizer

(continues)

TABLE 3.6 National Primary Drinking Water Standards (13) (continued)

Contaminants	MCLG (mg/L)	MCL (mg/L)	Potential health effects from ingestion of water	Sources of contaminant in drinking water
<i>Inorganics (cont.)</i>				
Mercury* (inorganic)	0.002	0.002	Kidney, nervous system disorders	Crop runoff; natural deposits; batteries, electrical switches
Nitrate	10	10	Methemoglobinemia	Animal waste, fertilizer, natural deposits, septic tanks, sewage
Nitrite	10	10	Methemoglobinemia	Same as nitrate; rapidly converted to nitrate
Selenium*	0.05	0.05	Liver damage	Natural deposits; mining, smelting, coal/oil combustion
Thallium	0.0005	0.002	Kidney, liver, brain intestinal	Electronics, drugs, alloys, glass
<i>Organics (1 of 4)</i>				
Acrylamide	zero	TT	Cancer, nervous system effects	Polymers used in sewage/ wastewater treatment
Adipate, (di(2-ethylhexyl))	0.4	0.4	Decreased body weight	Synthetic rubber, food packaging, cosmetics
Alachlor	zero	0.002	Cancer	Runoff from herbicide on corn, soybeans, other crops
Atrazine	0.003	0.003	Mammary gland tumors	Runoff from use as herbicide on corn and non-cropland
Carbofuran	0.04	0.04	Nervous, reproductive system effects	Soil fumigant on corn and cotton; restricted in some areas
Chlordane*	zero	0.002	Cancer	Leaching from soil treatment for termites
Chlorobenzene	0.1	0.1	Nervous system and liver effects	Waste solvent from metal degreasing processes
Dalapon	0.2	0.2	Liver and kidney effects	Herbicide on orchards, beans, coffee, lawns, road/railways
Dibromochloropropane	zero	0.0002	Cancer	Soil fumigant on soybeans, cotton, pineapple, orchards
<i>o</i> -Dichlorobenzene	0.6	0.6	Liver, kidney, blood cell damage	Paints, engine cleaning compounds, dyes, chemical wastes
<i>cis</i> -1,2-Dichloroethylene	0.07	0.07	Liver, kidney, nervous circulatory	Waste industrial extraction solvents
<i>trans</i> -1,2-Dichloroethylene	0.1	0.1	Liver, kidney, nervous circulatory	Waste industrial extraction solvents
Dichloromethane	zero	0.005	Cancer	Paint stripper, metal degreaser, propellant, extraction
1,2-Dichloropropane	zero	0.005	Liver, kidney effects; cancer	Soil fumigant; waste industrial solvents
Dinoseb	0.007	0.007	Thyroid, reproductive organ damage	Runoff of herbicide from crop and non-crop applications
Dioxin	zero	0.00000003	Cancer	Chemical production by-product; impurity in herbicides
Diquat	0.02	0.02	Liver, kidney, eye effects	Runoff of herbicide on land and aquatic weeds
2,4-D*	0.07	0.07	Liver and kidney damage	Runoff from herbicide on wheat, corn, rangelands, lawns

TABLE 3.6 National Primary Drinking Water Standards (13) (continued)

Contaminants	MCLG (mg/L)	MCL (mg/L)	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Endothall	0.1	0.1	Liver, kidney, gastrointestinal	Herbicide on crops, land/aquatic weeds; rapidly degraded
Endrin	0.002	0.002	Liver, kidney, heart damage	Pesticide on insects, rodents, birds; restricted since 1980
Epichlorohydrin	zero	TT	Cancer	Water treatment chemicals; waste epoxy resins, coatings
Ethylbenzene	0.7	0.7	Liver, kidney, nervous system	Gasoline; insecticides; chemical manufacturing wastes
Ethylene dibromide	zero	0.00005	Cancer	Leaded gasoline additives; leaching of soil fumigant
Glyphosate	0.7	0.7	Liver, kidney damage	Herbicide on grasses, weeds, brush
Heptachlor	zero	0.0004	Cancer	Leaching of insecticide for termites, very few crops
Heptachlor epoxide	zero	0.0002	Cancer	Biodegradation of heptachlor
Hexchlorobenzene	zero	0.001	Cancer	Pesticide production waste by-product
Hexachlorocyclopentadiene	0.05	0.05	Kidney, stomach damage	Pesticide production intermediate
Lindane	0.0002	0.0002	Liver, kidney, nervous immune, circulatory	Insecticide on cattle, lumber, gardens; restricted in 1983
Methoxychlor	0.04	0.04	Growth, liver, kidney nerve effects	Insecticide for fruits, vegetables, alfalfa livestock, pets
Oxamyl (Vydate)	0.2	0.2	Kidney damage	Insecticide on apples, potatoes, tomatoes
PAHs (benzo(a)pyrene)	zero	0.0002	Cancer	Coal tar coatings, burning organic matter volcanoes, fossil fuels
PCBs	zero	0.0005	Cancer	Coolant oils from electrical transformers; plasticizers
Pentachlorophenol	zero	0.001	Liver and kidney effects, and cancer	Wood preservatives, herbicide, cooling tower wastes
Phthalate, (di(2-ethylhexyl))	zero	0.006	Cancer	PVC and other plastics
Picloram	0.5	0.5	Kidney, liver damage	Herbicide on broadleaf and woody plants
Simazine	0.004	0.004	Cancer	Herbicide on grass sod, some crops, aquatic algae
Styrene	0.1	0.1	Liver, nervous system damage	Plastics, rubber, resin, drug industries; leachate from city landfills
Tetrachloroethylene	zero	0.005	Cancer	Improper disposal of dry cleaning and other solvents
Toluene	1	1	Liver, kidney, nervous, circulatory	Gasoline additive; manufacturing and solvent operations
Toxaphene	zero	0.003	Cancer	Insecticide on cattle, cotton, soybeans; canceled in 1982
2,4,5-TP	0.05	0.05	Liver and kidney damage	Herbicide on crops, right-of-way, golf courses; canceled in 1983
1,2,4-Trichlorobenzene	0.07	0.07	Liver, kidney damage	Herbicide production, dye carrier
1,1,2-Trichloroethane	0.003	0.005	Kidney, liver, nervous system	Solvent in rubber, other organic products; chemical production wastes

(continues)

TABLE 3.6 National Primary Drinking Water Standards (13) (continued)

Contaminants	MCLG (mg/l)	MCI (mg/l)	Potential health effects from ingestion of water	Sources of contaminant in drinking water
Organics (cont.) Xylenes (total)	10	10	Liver, kidney, nervous system	By-product of gasoline refining; paints, inks, detergents
Lead and Copper Lead*	zero	TT++	Kidneys, nervous system damage	Natural/industrial deposits; plumbing, solder, brass alloy faucets
Copper	1.3	TT#	Gastrointestinal irritation	Natural/industrial deposits; wood preservatives, plumbing
Other Interim Standards Beta/phon emitters	zero	4mrem/yr	Cancer	Decay of radionuclides in natural and man-made deposits
Alpha emitters	zero	15pCi/L	Cancer	Decay of radionuclides in natural deposits
Combined Radium 226/228	zero	5pCi/L	Bone cancer	Natural deposits
Arsenic*	0.05	0.05	Skin, nervous system toxicity	Natural deposits; smelters, glass, electronics wastes; orchards
Total Trihalomethanes	zero	0.10	Cancer	Drinking water chlorination by-products

Notes: TT = Special treatment techniques required.
 *Contaminants with interim standards which have been revised.
 + = Less than 5% positive samples.
 MFL = million fibers per liter.
 ++ = Action Level 0.01 5mg/L.
 # = Action Level 1.3 mg/L.
 pCi = picocuries

Secondary Drinking Water Standards

Secondary drinking water standards (Table 3.7) are established by the EPA for control of aesthetic qualities relating to public acceptance and include contaminants that affect taste, color, odor, and appearance. These standards provide recommended secondary maximum contaminant levels (SMCLs) that are not enforceable maximum values. The National Secondary Drinking Water Regulations are set forth in 40CFR143.

Drinking Water Contaminant Candidate List

Drinking water contamination generally results from:

- Contaminants that find their way into drinking water sources from industrial waste releases, agricultural runoff, atmospheric deposition, and other pollution sources
- Contaminants formed during the treatment of water supplies, including disinfection by-products
- Materials used for treatment, storage, and distribution of water

TABLE 3.7 National Secondary Drinking Water Standards (14)

Contaminant	SMCL
Aluminum, mg/L	0.05 to 0.2
Chloride, mg/L	250
Color, color units	15
Copper, mg/L	1.0
Corrosivity	noncorrosive
Fluoride, mg/L	2.0
Foaming agents	0.5
Iron, mg/L	0.3
Manganese, mg/L	0.05
Odor, threshold odor numbers	3
pH, units	6.5 to 8.5
Silver, mg/L	0.1
Sulfate, mg/L	250
Total dissolved solids, mg/L	500
Zinc	5

Prior to the 1996 Amendments, the Safe Drinking Water Act (SDWA) required the EPA to publish a drinking water priority list (DWPL) of contaminants every three years. The DWPL was to include contaminants that were known or anticipated to occur in drinking water and which may have required regulation under the SDWA. During the two-year period before the 1996 Amendments, the EPA developed a National Drinking Water Program Redirection Strategy (16) to:

- Establish priorities for setting safety standards based on health risks and sound science
- Support strong, flexible partnerships among the EPA, states, local governments, and other stakeholders to protect public health
- Promote effective community-based source water protection

The Redirection Strategy provided an overall framework for the development of the Contaminant Candidate List (CCL), as well as for other drinking water program activities.

The SDWA, as amended in 1996, required the EPA to publish a list of contaminants that are not subject to any proposed or promulgated national primary drinking water regulation (NPDWR), are known or anticipated to occur in public water systems, and which may require regulations under the SDWA (13).

The list of candidates were to be published after consultation with the scientific community, including the Science Advisory Board, after notice and opportunity for public comment, and after consideration of the occurrence database established under the SDWA. Unregulated contaminants considered for the list must include, but are not limited to, substances referred to in the Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA), and substances registered under the Federal Insecticide, Fungicide, and Rodenticide Act (FIFRA).

The CCL is the source of priority contaminants for the EPA's drinking water program. Priorities for drinking water research, occurrence monitoring, guidance development (including the development of health advisories), are drawn from the list. The CCL also serves as the list of contaminants from which the EPA makes determinations of whether or not to regulate specific contaminants.

The chemical and microbiological contaminants presented as the 1998 Drinking Water Contaminant Candidate List, are listed in Table 3.8. Chemical contaminants are identified by name and Chemical Abstracts Service Registry Number (CASRN). The 1998 CCL includes 50 chemical contaminants/contaminant groups and 10 microbiological contaminants.

TABLE 3.8 Drinking Water Contaminant Candidate List (15)

Chemical contaminants	CASRN
1,1,2,2-tetra-chloroethane	79-34-5
1,2,4-trimethylbenzene.	95-63-6
1,1 -dichloro-ethane	75-34-3
1,1 -dichloro-propene	563-58-6
1,2-diphenylhydrazine	122-66-7
1,3 -dichloropropane	142-28-9
1,3-Dichloropropene	542-75-6
2,4,6-trichlorophenol	88-06-2
2,2-dichloropropane	594-20-7
2,4-dichlorophenol	120-83-2
2,4-dinitrophenol	51-28-5
2,4-dinitrotoluene	121-14-2
2,6-dinitrotoluene	606-20-2
2-methyl-Phenol (<i>o</i> -cresol)	95 -48-7
Acetochlor	34256-82-1
Acetone	67-64-1
Alachlor ESA and other acetanilide pesticide degradation products	
Aldrin	309-00-2
Aluminum	7429-90-5
Boron	7440-42-8
Bromobenzene	108-86-1
DCPA mono-acid degradate	887-54-7
DCPA di-acid degradate	2136-79-0
DDE	72-55-9
Diazinon	333-41-5
Dieldrin	60-57-1
Disulfoton	298-04-4
Diuron	330-54-1
EPTC (<i>s</i> -ethyl-dipropylthiocarbamate)	759-94-4
Fonofos	944-22-9
Hexachlorobutadiene	87-68-3
<i>p</i> -Isopropylbenzene (<i>p</i> -cymene)	99-87-6
Linuron	330-55-2
Manganese	7439-96-5
Methyl bromide	74-83-9
Methyl- <i>t</i> -butyl ether (MTBE)	1634-04-4
Metolachlor	5 1218-45-2
Metribuzin	21087-64-9
Molinate	2212-67-1
Naphthalene	91-20-3
Nitrobenzene	98-95-3
Organotins	NA
Perchlorate	NA
Prometon	1610-18-0
RDX	121-82-4
Sodium	7440-23-5
Sulfate	14808-79-8
Terbacil	5902-51-2
Terbufos	13071-79-9

TABLE 3.8 Drinking Water Contaminant Candidate List (15) (continued)

Chemical contaminants	CASRN
Triazines and degradation products of triazines (including, but not limited to)	
Cyanazine	21725-46-2
Atrazine-desethyl	6190-65-4
Vanadium	7440-62-2
Microbiological Contaminants	
Acanthamoeba (guidance expected for contact lens wearers)	
Adenoviruses	
<i>Aeromonas hydrophila</i>	
Caliciviruses	
Coxsackieviruses	
Echoviruses	
<i>Helicobacter pylori</i>	
Microsporidia (Enterocytozoon and Septata)	
<i>Mycobacterium avium intracellulare</i> (MAC)	

WATER QUALITY OBJECTIVES

The intent of the Clean Water Act is to restore, maintain, and enhance the chemical, physical, and biological integrity of the nation's waters. To this end, a program of establishing stream water quality standards and effluent limitations was set forth.

Water Quality Standards

The relationship between water quality standards and criteria evolved from the Water Quality Act of 1965. Standards of stream water quality were required to be established while considering their use and value for public water supplies; propagation of fish and wildlife, recreational purposes, and agricultural, industrial, and other legitimate uses. Criteria are a scientific requirement on which a decision is based concerning the suitability of water quality to support a designated use.

Water quality standards are enforceable by law and apply to all navigable waters. A water quality standard, as defined in the 1965 act, is a process which consists of

1. Determining the designated water use
2. Adopting applicable criteria to maintain that use
3. Developing an implementation and enforcement plan for the adopted water quality criteria

The purposes of water quality standards are to protect the public health or welfare and enhance the quality of water consistent with the designated water uses. An example of criteria used to support a designated stream use is presented in Table 3.9. In most cases, a criterion is a numeric constituent concentration or level. However, where a numeric value is not available or known, a narrative statement can be used. When a criterion becomes a part of the state's water quality standards, it is then an enforceable requirement that is used to regulate pollutants under the Clean Water Act.

A point of confusion often arises when working with standards and criteria under the Clean Water Act. Under the Clean Water Act, a *standard* is a process and *criteria* are part of the standard. Under the Safe Drinking Water Act, numeric constituent concentrations or levels (criteria) are standards.

TABLE 3.9 Example Water Quality Standard (4)*Basic Water Use*

All waters of the State shall be protected for the basic uses of water contact recreation, fish, other aquatic life, and wildlife. These uses compose Class I. Criteria for Class I Waters shall apply to all waters of the State unless contravened by more restrictive criteria for other specific classes. Criteria to protect Class I waters are sufficiently stringent to afford protection also for public water supply in freshwater areas (with treatment by filtration and disinfection), agricultural water supply, and industrial water supply. More restrictive criteria are established to protect shellfish harvesting waters, natural trout waters, and recreational trout waters.

General Water Quality Criteria

The waters of the State at all times shall be free from:

- (1) Substances attributable to sewage, industrial waste, or other waste that will settle to form sludge deposits that are unsightly, putrescent, or odorous to a degree as to create a nuisance, or that interfere directly or indirectly with water uses;
- (2) Floating debris, oil, grease, scum, and other floating materials, attributable to sewage, industrial waste, or other waste in amounts sufficient to be unsightly to a degree as to create a nuisance, or that interfere directly or indirectly with water uses;
- (3) Materials attributable to sewage, industrial waste, or other waste which produce taste, odor, or change the existing color or other physical and chemical conditions in the receiving waters to a degree as to create a nuisance, or that interfere directly with water uses; and
- (4) High-temperature, toxic, corrosive, or other deleterious substances attributable to sewage, industrial waste, or other waste in concentrations or combinations which interfere directly or indirectly with water uses, or which are harmful to human, animal, plant, or aquatic life.

Class I: Water Contact Recreation and Aquatic Life. Waters which are suitable for:

- (i) Water contact sports;
- (ii) Play and leisure time activities where the human body may come in direct contact with the surface water; and
- (iii) The growth and propagation of fish (other than trout), other aquatic life, and wildlife.

These criteria shall apply during periods of flow greater than or equal to the 7-day, 10-year low flow. Where the waters of the State are or may be affected by discharges from point sources, these standards shall apply outside of any mixing zones which may be designated by the Administration. It is recognized that in some cases the natural water quality of a stream segment may not be consistent with the criteria established for the stream. In these cases, it is not intended that these natural conditions constitute a violation of the water quality standards, or that the water quality to be maintained and achieved be substantially different from that which would occur naturally.

Criteria for Class I Waters: Water Contact Recreation and Aquatic Life

1. Bacteriological. There may not be any sources of pathogenic or harmful organisms in sufficient quantities to constitute a public health hazard. A public health hazard will be presumed if the fecal coliform density exceeds a log mean of 200 per 100 ml, based on a minimum of not less than 5 samples taken over any 30-day period, or if 10 percent of the total number of samples taken during any 30-day period exceed 400 per 100 ml, unless a sanitary survey approved by the Department of Health and Mental Hygiene disclosed no significant health hazard.
2. Dissolved Oxygen. The dissolved oxygen concentration shall be not less than 5.0 mg/liter at any time.
3. Temperature. For all discharges of heat, the maximum temperature outside the mixing zone may not exceed 90°F (32°C) or ambient temperature of the receiving waters, whichever is greater. In addition, a discharge of heat may not create thermal barriers that adversely affect aquatic life.
4. pH. Normal pH values may not be less than 6.5 or greater than 8.5.
5. Turbidity. Turbidity may not exceed levels detrimental to aquatic life. Turbidity in the receiving water, resulting from any discharge may not exceed 150 NTU (nephelometer turbidity units) at any time or 50 NTU as a monthly average. NTU are equivalent measures to FTUs (formazin turbidity units) and JTUs (Jackson turbidity units).
6. Toxic Materials. The toxic materials listed here may not exceed these designated limits at any time:
 - (i) Polychlorinated Biphenyls (PCBs)—0.001 µg/liter;
 - (ii) Endrin—.004 µg/liter;
 - (iii) Toxaphene—.005 µg/liter;
 - (iv) DDT—0.001 µg/liter;
 - (v) Benzidine—0.1 µg/liter;
 - (vi) Aldrin-Dieldrin—0.003 µg/liter;

The first step in establishing water quality standards is to determine the stream uses. States have primary responsibility for designating stream segment uses, and the standards are then established based on the designated use. The water uses are determined by analyzing the water's existing uses and the potential to attain particular uses based on physical, chemical, biological, and hydrological characters of the waters and the economic impacts of attaining various uses. The Clean Water Act requires that, wherever attainable, water quality standards should provide for the protection and propagation of fish, shellfish, and wildlife and to provide for recreation in and on the water. Stream uses, of course, vary from state to state, but one state's stream use cannot result in the violation of another state's stream use of the same stream.

The second step is to establish criteria that will protect an organism, an organism community, or a prescribed water use or quality with an adequate degree of safety. The Clean Water Act provides guidance on establishing criteria, in lieu of which experimental bioassay or biological criteria must be used. Each criterion should be based on the latest scientific information available on the effect of a pollutant on aesthetics, human health, and aquatic life. In terms of commonly used criteria, all of the states' water quality standards contain criteria for dissolved oxygen, pH, fecal coliform bacteria, temperature, and aesthetic qualities.

TABLE 3.10 Water Program Regulations

40 CFR Part	Topic
100	[Reserved]
104	Public hearings on effluent standards for toxic pollutants
108	Employee protection hearings
109	Criteria for state, local and regional oil removal contingency plans
110	Discharge of oil
112	Oil pollution prevention
113	Liability limits for small onshore storage facilities
114	[Reserved]
116	Designation of hazardous substances
117	Determination of reportable quantities for hazardous substances
121	State certification of activities requiring a federal license or permit
122	EPA administered permit programs: the national pollutant discharge elimination system
123	State program requirements
124	Procedures for decision making
125	Criteria and standards for the national pollutant discharge elimination system
129	Toxic pollutant effluent standards
130	Water quality planning and management
131	Water quality standards
132	Water quality guidance for the Great Lakes System
133	Secondary treatment regulation
135	Prior notice of citizen suits
136	Guidelines establishing test procedures for the analysis of pollutants
140	Marine sanitation device standard
141	National primary drinking water regulations
142	National primary drinking water regulations implementation
143	National secondary drinking water regulations
144	Underground injection control program
145	State underground injection control program requirements
146	Underground injection control program: Criteria and standards
147	State underground injection control programs
148	Hazardous waste injection restrictions
149	Sole source aquifers

Several documents have been published that may be of use in assisting in the establishment of water quality criteria (5–8, 17). After criteria are established, the state holds hearings to review the water quality standards and for public comment. Following any changes or revisions, the state adopts the standards pursuant to state law. The standards are then reviewed by the EPA and accepted or rejected. Once the standards have been accepted, the standards for interstate navigable waters become federal and state standards. The standards should be reviewed and revised where applicable, at least once every three years.

Water Quality Protection Programs

In addition to standards per se, there are many federal “water” programs affecting water quality. Table 3.10 lists selected regulations promulgated by the EPA for the Clean Water Act and the Safe Drinking Water Act.

EFFLUENT STANDARDS

Effluent discharge limitation guidelines and standards have been, and are being, developed to help meet stream standards and associated use designations. The enforcement mechanism legislated by the Clean Water Act of 1977 is the National Pollutant Discharge Elimination System (NPDES). NPDES permits are issued to municipal and industrial dischargers to ensure that pollutant discharges do not result in a violation of water quality standards. State and federal monitoring, inspection, and enforcement programs ensure compliance with standards and permits.

The NPDES permit contains technology-based treatment requirements for domestic or industrial facilities and for phased improvements in technology to allow higher levels of treatment.

For waters that will not achieve water quality standards after implementation of technology-based controls, a total maximum daily load (TMDL) analysis is required by the Clean Water Act. A TMDL analysis includes the determination of the relative contributions of pollutants from point, nonpoint, and natural background sources, including a margin of safety of pollutants that can be discharged to a water quality-limited waterbody to meet water quality standards.

More specifically, an allowable TMDL is defined as the sum of the individual wasteload allocations for existing and future point sources (including stormwater) and load allocations for existing and future non-

TABLE 3.11 Secondary Treatment Requirements (9)

Pollutant	Effluent limitations*	
BOD ₅ [†]	30 (45) mg/L	Maximum 30-day average
	45 (65) mg/L	Maximum 7-day average
	85% (65) removal	Minimum 30-day average
Suspended solids	30 (45) mg/L	Maximum 30-day average
	45 (65) mg/L	Maximum 7-day average
	85% (65) removal	Minimum 30-day average
pH	6.0–9.0	Range

*() denotes value applicable to treatment equivalent to secondary treatment. Adjustment available for effluents from trickling filter facilities and waste stabilization pond facilities.

[†]CBOD₅ is approved substitute for BOD₅, the CBOD₅ limitations are:

40 mg/L	Maximum 30-day average
60 mg/L	Maximum 7-day average
65% removal	Minimum 30-day average

point sources (including diffuse runoff and agricultural stormwater) and natural background materials with a margin of safety incorporated to account for uncertainty in the analysis. Load reductions obtained through the implementation of BMPs required in the NPDES program for stormwater should be reflected in the TMDL analysis.

Secondary Treatment

The baseline level of treatment for effluents from publicly owned treatment works is secondary treatment. Table 3.11 shows the required level of effluent quality for secondary treatment or its equivalent. Industrial wastes discharged to the treatment works may alter the stated limits.

Especially since federal grant monies were used to assist in construction funding of most municipal treatment facilities, a national program was developed for industrial waste pretreatment. The requirements of this program are summarized in Table 1.12.

Industrial Effluents

Permits for industrial effluents are subject to state and/or federal requirements. In particular, many industrial categories are covered by federal effluent guidelines and standards (11), as shown by Table 3.13.

Toxic Pollutants

Special emphasis is placed on the control of an effluent containing toxic pollutants. Table 3.14 lists pollutants that have been designated as toxic by the EPA. Effluent standards have been promulgated for some toxic pollutants; these are highlighted in Table 3.15.

TABLE 3.12 National Pretreatment Standards (10)

-
1. General Prohibitions: A user may not introduce into a publicly owned treatment works (POTW) any pollutant(s) which cause "pass through" without a change in nature or "interference" with treatment processes, including sludge use or disposal.
 2. Specific Prohibitions: The following pollutants shall not be introduced into a POTW:
 - a. Pollutants which create a fire or explosion hazard in the POTW;
 - b. Pollutants that will cause corrosive structural damage to the POTW, but in no case discharges with pH lower than 5.0, unless the works is specifically designed to accommodate such discharges;
 - c. Solid or viscous pollutants in amounts which will cause obstruction to the flow in the collection system and the treatment plant resulting in interference with operations;
 - d. Any pollutant, including oxygen-demanding pollutants (BOD, etc.) released in a discharge at a flow rate and/or pollutant concentration that will cause interference with the POTW operations;
 - e. Heat in amounts that will inhibit biological activity in the POTW resulting in interference, but in no case heat in such quantities that the temperature at the POTW exceeds 40°C (104°F) unless the approval authority, upon request of the POTW, approves alternate temperature limits.
 3. Categorical Requirements: Standards specifying quantities or concentrations of pollutants or pollutant properties which may be discharged to a POTW by existing or new industrial users in specific industrial subcategories will be established as separate regulations under the appropriate subpart of 40 CFR Chapter I, Subchapter N. These standards, unless specifically noted otherwise, shall be in addition to the general prohibitions established above (40 CFR 403.5).
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TABLE 3.13 Industrial Effluent Limitations and Guidelines

Aluminum Forming (40 CFR 467)

Development Document for Effluent Limitations Guidelines and Standards for the Aluminum Forming Point Source Category [Draft], U.S. Environmental Protection Agency, EPA 440/1-80-073a, Washington, D.C., 1980.

Development Document for Effluent Limitations Guidelines and Standards for the Aluminum Forming Point Source Category [Proposed], U.S. Environmental Protection Agency, EPA 440/1-82-073b, Washington, D.C., 1982.

Asbestos Manufacturing (40 CFR 427)

Development Document for Proposed Effluent Limitations Guidelines and New Source Performance Standards for the Building, Construction and Paper Segment of the Asbestos Manufacturing Point Source Category [Proposed], U.S. Environmental Protection Agency, EPA 440/1-73-017, Washington, D.C., 1973.

Battery Manufacturing (40 CFR 461)

Development Document for Effluent Limitations Guidelines and Standards for the Battery Manufacturing Point Source Category [Draft], U.S. Environmental Protection Agency, EPA 440/1-80-067a, Washington, D.C., 1980.

Development Document for Effluent Limitations Guidelines and Standards for the Battery Manufacturing Point Source Category [Proposed], U.S. Environmental Protection Agency, EPA 440/1-82-06Th, Washington, D.C., 1982.

Builders' Paper and Board Mills (40 CFR 431)

Development Document for Effluent Limitations Guidelines and New Source Performance Standards for the Builders' Paper and Roofing Felt Segment of the Builders' Paper and Board Mills Point Source Category, U.S. Environmental Protection Agency, EPA 440/1-74-026a, Washington, D.C., 1974.

Development Document for Effluent Limitations Guidelines and Standards for the Pulp, Paper and Paperboard and the Builders' Paper and Board Mills Point Source Categories [Proposed], U.S. Environmental Protection Agency, EPA 440/1-80-025b, Washington, D.C., 1980.

Centralized Waste Treatment (Landfills and Incinerators) (40 CFR 437)

Detailed Costing Document for the Centralized Waste Treatment Industry, U.S. Environmental Protection Agency, EPA 821/R-95-002, Washington, D.C., 1995

Environmental Assessment of Proposed Effluent Guidelines for the Centralized Waste Treatment Industry, U.S. Environmental Protection Agency, EPA 821/R-95-003, Washington, D.C., 1995.

Preliminary Data Summary for the Hazardous Waste Treatment Industry, U.S. Environmental Protection Agency, EPA 440/1-89-100, Washington, D.C., 1989.

Statistical Support Document for Proposed Effluent Limitations Guidelines and Standards for the Centralized Waste Treatment Industry, U.S. Environmental Protection Agency, EPA 821/R-95-005, Washington, D.C., 1995.

Coal Mining (40 CFR 434)

Development Document for Interim Final Effluent Limitations Guidelines and New Source Performance Standards for the Coal Mining Point Source Category—Group II, U.S. Environmental Protection Agency, EPA 440/1-75-057, Washington, D.C., 1975.

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TABLE 3.14 Toxic Water Pollutants

<i>Volatile</i>	benzo(k)fluoranthene
acrolein	bis(2-chloroethoxy)metha.ne
acrylonitrile	bis(2-chloroethyl)ether
benzene	bis(2-chloroisopropyl)ether
bromoform	bis(2-ethylhexyl)phthalate
carbon tetrachloride	4-bromophenyl phenyl ether
chlorobenzene	butylbenzyl phthalate
chlorodibromomethane	2-chloronaphthalene
chloroethane	4-chlorophenyl phenyl ether
2-chloroethylvinyl ether	chrysene
chloroform	dibenzo(a,h)anthracene
dichlorobromomethane	1,2-dichlorobenzene
1,1-dichloroethane	1,3-dichlorobenzene
1,2-dichloroethane	1,4-dichlorobenzene
1,1-dichloroethylene	3,3'-dichlorobenzidine
1,2-dichloropropane	diethyl phthalate
1,3-dichloropropylene	dimethyl phthalate
ethylbenzene	di-n-butyl phthalate
methyl bromide	2,4-dinitrotoluene
methyl chloride	2,6-dinitrotoluene
methylene chloride	di-n-octyl phthalate
1,1,2-tetrachloroethane	1,2-diphenylhydrazine (as azobenzene)
tetrachloroethylene	fluroranthene
toluene	fluorene
1,2-trans-dichloroethylene	hexachlorobenzene
1,1,1-trichloroethane	hexachlorobutadiene
1,1,2-trichloroethane	hexachlorocyclopentadiene
trichloroethylene	hexachloroethane
Vinyl chloride	indeno(1,2,3-cd)pyrene
<i>Acid Compounds</i>	isophorone
2-chlorophenol	naphthalene
2,4-dichlorophenol	nitrobenzene
2,4-dimethylphenol	N-nitrosodimethylamine
4,6-dinitro-o-cresol	N-nitrosodi-n-propylamine
2,4-dinitrophenol	N-nitrosodiphenylamine
2-nitrophenol	phenanthrene
4-nitrophenol	pyrene
p-chloro-m-cresol	1,2,4-trichlorobenzene
pentachlorophenol	<i>Pesticides</i>
phenol	aldrin
2,4,6-trichlorophenol	alpha-BHC
<i>Base/Neutral</i>	beta-BHC
acenaphthene	gamma-BHC
acenaphthylene	delta-BHC
anthracene	chlordan
benzidine	4,4'-DDT
benzo(a)anthracene	4,4'-DDE
benzo(a)pyrene	4,4'-DDD
3,4-benzofluoranthene	dieldrin
benzo(ghi)perylene	alpha-endosulfan

(continues)

TABLE 3.14 Toxic Water Pollutants (*continued*)

<i>Pesticides (cont.)</i>	<i>Metals//Cyanide and Total Phenols</i>
beta-endosulfan	Antimony, Total
endosulfan sulfate	Arsenic, Total
endrin	Beryllium, Total
endrin aldehyde	Cadmium, Total
heptachlor	Chromium, Total
heptachlor epoxide	Copper, Total
PCB-1242	Lead, Total
PCB-1254	Mercury, Total
PCB-1221	Nickel, Total
PCB-1232	Selenium, Total
PCB-1248	Silver, Total
PCB-1260	Thallium, Total
PCB-1016	Zinc, Total
toxaphene	Cyanide, Total
	Phenols, Total

TABLE 3.15 Toxic Pollutant Effluent Standards (12)

Toxic pollutant	Source	Effluent limitation	
Aldrin/Dieldrin	Manufacturer	Prohibited	
	Formulator	Prohibited	
DDT	Manufacturer	Prohibited	
	Formulator	Prohibited	
Endrin	Manufacturer	Existing	1.5 µg/L (A)
			0.0006 kg/kkg (B)
			7.5 µg/L (C)
	New	0.1 µg/L (A)	
		0.00004 kg/kkg (B)	
		0.5 µg/L (C)	
Toxaphene	Formulator	Prohibited	
	Manufacturer	Existing	1.5 µg/L (A)
			0.00003 kg/kkg (B)
			7.5 µg/L (C)
	New	0.1 µg/L (A)	
		0.000002 kg/kkg (B)	
0.5 µg/L (C)			
Benzidine	Formulator	Prohibited	
	Manufacturer	Existing and New	10 µg/L (A)
			0.130 kg/kkg (B)
	Applicator	Existing and New	50 µg/L (C)
			10 µg/L (A)
		25 µg/L (C)	
Polychlorinated Biphenyls (PCBs)	Electrical Capacitor Manufacturer	Prohibited	
	Electrical Transformer Manufacturer	Prohibited	

(A) An average per working day, calculated over any calendar month.
 (B) Monthly average daily loading per quantity of pollutant produced.
 (C) A sample(s) representing any working day.

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CHAPTER 4

AIR QUALITY CONTROL

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Air quality control needs vary from correction of air pollution problems originating from relatively small areas, such as an industrial park impacted by one or more emission sources, to those from large areas, such as an urban area impacted by a number of sources and a variety of contaminants.

Air pollution is used to describe the presence in the atmosphere of one or more contaminants in quantities and/or characteristics that will, over a period of time be injurious to or unreasonably interfere with public health and welfare or natural environmental processes. Contaminants are categorized as particulate matter and gases and their associated forms, including dust, smoke, fumes, mist, and vapor. The primary gaseous air contaminants are carbon monoxide, hydrocarbons, nitrogen oxides, and sulfur oxides (1, 2, 3).

Meteorological and topographical factors contribute to the creation and continuation of air pollution under specific site conditions. Temperature inversions prevent upward diffusion, and very low wind speeds allow emissions to remain near their source. Some terrains cause emissions to follow specific patterns from one area to another.

Generally, sources of air contaminants may be classified as stationary, mobile, or fugitive. Respectively, they are attributed to point sources, such as industrial stack emissions; transportation activities, such as automobile emissions; and uncontrolled (fugitive) sources, such as wind-blown dusts from stockpiles.

The environmental engineer is instrumental in controlling particulate and gas sources of air contaminants. Source control is the first abatement method considered. For particulates, settling chambers, inertial separators, wet scrubbers, and fabric filters are used. Gas controls include absorption, adsorption, condensation, flaring, and incineration. Other areas of practice address acid rain issues, fugitive emissions, odor control, indoor air quality, and noise abatement.

AIR POLLUTANTS: SOURCES AND EFFECTS

A substance is not normally identified as an air contaminant until its presence and concentration actually or potentially produce or contribute to development of a deleterious effect. The composition of an unpolluted dry air is shown in Table 4.1.

Sources of air contaminants may be classified as stationary, mobile, or fugitive. Mobile sources are attributed to transportation activities, such as automobile exhausts, and are not included per se in this text. This chapter focuses on emissions from stationary sources. Special sections are provided on fugitive emissions and related "air" topics of odor, indoor air quality, and noise pollution. These sections address source, effect, and control of pollutants.

*This chapter includes contributions from Harold J. Rafson.

TABLE 4.1 Composition of Dry Air

Component	Concentration, ppm
Nitrogen	780,800
Oxygen	209,500
Argon	9,300
Carbon dioxide	315
Neon	18
Helium	5.2
Methane	1.0+
Krypton	1.0
Nitrous oxide	0.5
Hydrogen	0.5
Xenon	0.08
Nitrogen dioxide	0.02
Ozone	0.01+

The effects of an air pollution problem range in scope from a single type of contaminant to a multifaceted problem resulting from multiple contaminants complexed by atmospheric interactions.

Sources of Air Contaminants

Air contaminants originate in a wide variety of chemical compositions and different physical states and are emitted from a diversity of sources. Discussions follow that identify primary air contaminants and their principal sources. The term “primary” is used to denote direct emissions of a contaminant. In some cases, secondary air contaminants are formed in the atmosphere by chemical interactions among primary contaminants under normal atmospheric constituents.

Carbon Monoxide. Carbon monoxide (CO) is a colorless, odorless, tasteless gas formed primarily by the incomplete combustion of carbonaceous fuels. Important variables affecting its emission concentration include combustion chamber residence time and turbulence (physical characteristics), flame temperature, and oxygen concentration.

By far, the major source of carbon monoxide is fuel combustion in the internal combustion engine of mobile sources. Miscellaneous combustion sources and industrial processes contribute to a much lesser extent.

A companion gas emission from fuel combustion is carbon dioxide (CO₂), which is also a secondary contaminant formed by the oxidation (very slow reaction rate) of carbon monoxide in the atmosphere.

Hydrocarbons. Compounds of carbon and hydrogen constitute primary hydrocarbon contaminants, such as aromatics, olefins, and paraffins, which originate with the processing and use of petroleum and its products. Derivatives or secondary contaminants, such as aldehydes, ketones, and organic acids, are formed when hydrogen is replaced with oxygen, halogens, or other substituent groups.

Like carbon monoxide, the most significant source of hydrocarbons is the use of petroleum products to fuel motor vehicles. Petroleum processing, solvent use, and related uses of petroleum products in commercial and industrial operations contribute to air pollution by hydrocarbons.

Lead. Unlike other toxic heavy metals, lead is relatively abundant throughout the world and, thus, has many potential pathways into the human body via inhalation and ingestion. The potential for lead in air pollution is greatest in the vicinity of sources and in densely populated areas.

Historically, the primary source of lead emissions has been the result of tetraethyl lead additives to gasoline. Legislative controls have done much to lessen the impact of this source in the United States. Stationary sources include mining, smelting, waste incineration, iron and steel production, lead alkyl manufacturing, and battery manufacturing, where the lead is released in stack and fugitive emissions.

Nitrogen Oxides. Of the numerous forms of nitrogen oxide possible, nitric oxide (NO) and nitrogen dioxide (NO₂) are the most significant air contaminants. Nitric oxide is the primary contaminant and is formed by all high-temperature, atmospheric combustion through the direct combination of nitrogen and oxygen. In the presence of sunlight, nitric oxide combines with atmospheric oxygen to form nitrogen dioxide as a secondary contaminant.

The major source of nitrogen oxides is from fuel combustion, where the quantity of nitrogen oxides is a function of the available nitrogen and oxygen concentrations, reaction time, and temperature. The chemical processing industry is another notable source of localized emissions.

Ozone. Photochemical oxidants, mostly as ozone, are the product of atmospheric reactions of such certain contaminants (precursors), such as hydrocarbons and nitrogen oxides, in the presence of sunlight. The formation of ozone also involves the physical processes of dispersion and transport of precursors.

Particulate Matter. The term “particulate matter” (or “aerosols”) is used to denote solid and liquid matter of organic or inorganic composition that is suspended as the result of a stack or fugitive emission. The matter may be individual elements and/or compounds and may or may not be emitted along with gaseous contaminants.

Particle size may be used to classify the types of sources. For example, particles less than 1 μm (in diameter) are mostly products of condensation and combustion. Large particles, above 10 μm, result from physical actions, such as wind erosion and grinding or spraying operations. Those particles between 1 and 10 μm tend to be fugitive dusts, process dusts, and combustion products.

Sulfur Oxides. The air contaminants categorized as sulfur oxides are sulfur dioxide (SO₂), which is predominant, and sulfur trioxide (SO₃). The primary source for both is the combustion of fuels, mainly coal, containing sulfur in the presence of air (oxygen). The most significant secondary contaminant is sulfuric acid.

Effects of Air Contaminants

Impacts on public health, vegetation, materials, and/or visibility are used to describe the effects of air contaminants. In the United States, the Environmental Protection Agency (EPA) develops and administers emission source criteria and ambient air quality standards (Chapter 3) intended to control negative effects. Discussions follow on the effects of the contaminants included in the ambient air quality standards.

Carbon Monoxide. The significance of carbon monoxide is its effects on human and other animal health; plants are relatively insensitive, and other deleterious effects are not notable.

Carbon monoxide is absorbed by the lungs and is associated with a decrease of oxygen-carrying capacity of the bloodstream and of available oxygen for body tissue. As a function of concentrations and exposure time, effects may include physiologic stress and impaired motor skills, visual discrimination, and time interval recognition.

Hydrocarbons. The effects of hydrocarbons vary according to the individual compounds and derivatives. Generally, hydrocarbons are recognized as components and promoters of photochemical smog. The associated effects are eye irritation and other manifestations, injury to sensitive plants, and reduced visibility.

Lead. The multitude of sources and exposures to lead make it more than an air contaminant. Primary exposure occurs from direct inhalation, and secondary exposure comes from various routes to produce ingestion of lead. Furthermore, lead affects human health at the subcellular, cellular, and organ system levels. Some specific effects of higher blood levels of lead include anemia, cognitive deficits, peripheral neuropathies, and encephalopathy symptoms.

Nitrogen Oxides. The effects of nitrogen oxides are often referenced as NO, NO₂, and NO_x; however, the most deleterious effects tend to be from NO₂. For example, since nitrogen dioxide absorbs the full visible spectrum of light energy, it can reduce visibility even in the absence of particulate matter.

Other effects include a number of respiratory disorders, depending on concentration, time of exposure, and affected age group. Material effects include fading of certain dyes, deterioration of selected fabrics, and cases of metal corrosion. Also, extended exposures can cause leaf or other damage to vegetation.

Ozone. The effects of ozone are widespread and are the result of its characteristics as a high-strength oxidizing chemical. Human health, materials, and vegetation are all subject to adverse impact as a function of ozone concentration and contact time.

Particulate Matter. Some effects of particulate matter are very evident to the affected general public. Problems of reduced visibility, eye irritation, and soiling of clothes are readily noticeable.

Other effects, such as interactions with other contaminants, are less obvious. For example, the adverse effects of sulfur oxides are increased in the presence of particulate matter, and human respiratory problems may be accelerated due to contaminants associated with inhaled particulates.

Sulfur Oxides. The effects of sulfur oxides are manifested in the presence of particulate matter. This is shown by case studies and research investigations with respect to health impacts related to irritation of the respiratory system, to reduced visibility, to corrosion of materials, and to varying sensitivity of plant species.

An effect widely publicized is the formation of acid (H₂SO₄) rain by the reaction of sulfur oxides and atmospheric moisture. Acid rain has a pH of 2 or less and is responsible for acidifying streams and lakes and, thus, not only killing fish but leaving waterways too acidic to be reinhabited. European and North American countries are confronted with not only air pollution control problems but also political issues due to the migration of acid rain across political boundaries.

CHARACTERIZATION

The characterization of an emission stream begins with a survey of facility operations and a determination of emission locations. The stack emissions must be quantified through a sampling and measurement program, and, in some cases, ambient air monitoring is required or desirable. The following discussion focuses on procedures for an industrial installation (4), but largely applies to other facilities, such as a solid waste incinerator.

Emission Survey

The first step in characterizing air pollution from an industrial facility is the emission survey, which locates sources and defines quantities for all air contaminants.

Source Identification. The identification of emission sources begins with an in-depth review of process flow sheets and associated data. This data base is then tested and verified by a tour of plant operations.

Process Flow Sheets. Design, or preferably, “record” drawings of the facility’s processes will identify the location of emission sources. Typically, process flow sheets will provide sufficient information on input materials and process functions to make a qualitative assessment of the emissions. Figure 4.1 illustrates the components of a simple, generic process flow sheet.

Design development reports, permit applications, historical process and emission records for the facility or a similar facility, and other such data will aid in the preliminary identification of emission sources.

This step of the emission survey concludes with the development of plant-specific survey data sheets pertaining to the process(es) and emission control(s) data and with the establishment of an appropriate filing or coding system for data management (4).

Plant Inspection. With the foregoing material in hand, an inspection tour is made of the plant to verify available records, to record undocumented changes, and to provide input (Table 4.2), to the design of control equipment. Candid interviews with plant operating personnel are a very important function during the tour.

Completing the plant inspections with accurate stack information is another important task. These data are needed for development of a testing program.

Emission Quantifications. Data from the records search and plant tour are next organized to formulate specific emission survey functions as part of the plant’s compliance schedule.

Compliance Program. Source testing is required to define needs to achieve compliance, to demonstrate effectiveness of new control techniques, or to provide records of continuing compliance based on quantitative field results from individual sources. Steps in achieving compliance are charted in Figure 4.2 (5).

Quantification Procedures. The emission survey is developed by applying a combination of procedures related to quantity measurements from existing files, ongoing record keeping and monitoring, and new data collection. These procedures include review of permit application and design data, analysis of fuel and raw material usage, calculation of mass balance, application of expected emission factors from the literature (6), and new testing of emission sources.

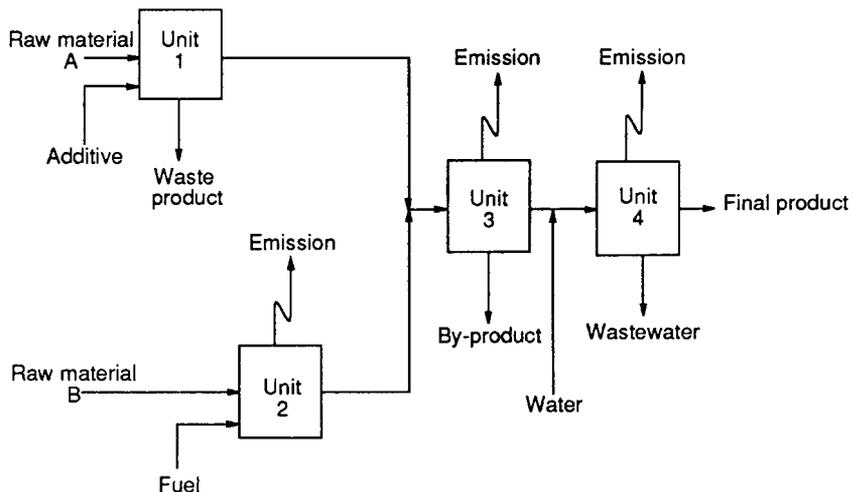


FIG. 4.1 Example process flow sheet.

TABLE 4.2 Plant Inspection Data Needed for Control Equipment Design (4)

Site characteristics	Process characterization
1. Environmental conditions, such as ambient temperatures and wind patterns	1. Capacity and capability of existing control equipment
2. Proximity to sensitive areas, such as residential or public access developments	2. Reuse/recycling of collected emissions and process by-products and wastes
3. Availability of water and power	3. Status and future of control regulations
4. Availability of solid waste and waste-water disposal facilities	4. Anticipated changes in raw material and additives
5. Physical limitations, such as roof loads and space	5. Frequency of startups and shutdowns
	6. Plant expansion plans

Emission Measurements

Measurement of plant emissions provides a data base for determination of needs for new control equipment, effectiveness of existing control equipment, compliance with emission regulations and/or permit requirements, and losses of products or by-products via the emission. The process involves sampling and testing procedures and physical and chemical measurements.

Emission Testing. The planning and conduct of emission testing begins with development of the testing program, including definition of sampling point requirements. Finally, the test procedures are defined.

Testing Program Development. A first step in developing a compliance testing program is to define and coordinate with the participants. The participants normally include plant process and pollution control personnel, the testing team, and the regulatory agency. The goal of this group is to agree on general and often very specific aspects of the testing program.

Using the applicable regulatory requirements, compliance schedule criteria, and the findings of the emission survey, a written testing program is initiated. Contents include the location of sampling points, parameters to be tested, and the facility production operations for the test period.

Sampling Point Requirements. In addition to defining an emission for testing, sampling requirements must be refined to best obtain representative samples for analysis. Sampling ports, a work platform, and a power supply are the primary features of a stack sampling arrangement. Using U.S. customary units for dimensioning, Figure 4.3 illustrates and denotes requirements for stack sampling.

Emission Measurements. In order to standardize procedures for obtaining representative samples, the EPA has published reference sampling methods for most parameters of interest. These methods are compiled in Table 4.3. The following discussions and illustrations are used to highlight requirements for selected measurements.

Velocity. Reference method 2 is used for determining the velocity and volumetric flow rate of stack gases. At several sampling points with equal portions of the stack volume, the velocity and temperature are measured with the system shown in Figure 4.4.

Moisture Content. Reference method 4 describes procedures for determining moisture content of a stack gas, and Figure 4.5 illustrates a moisture sampling train. This method is not applicable if liquid droplets are present.

When liquid droplets are in the emission stream, gas temperatures should be obtained at several designated points in the stack. By assuming that the emission stream is saturated, the moisture content is determined from a psychrometric chart or saturated vapor pressure tables.

Particulates. Reference method 5 is designed for material collectible at 120°C (250°F) on a filtering

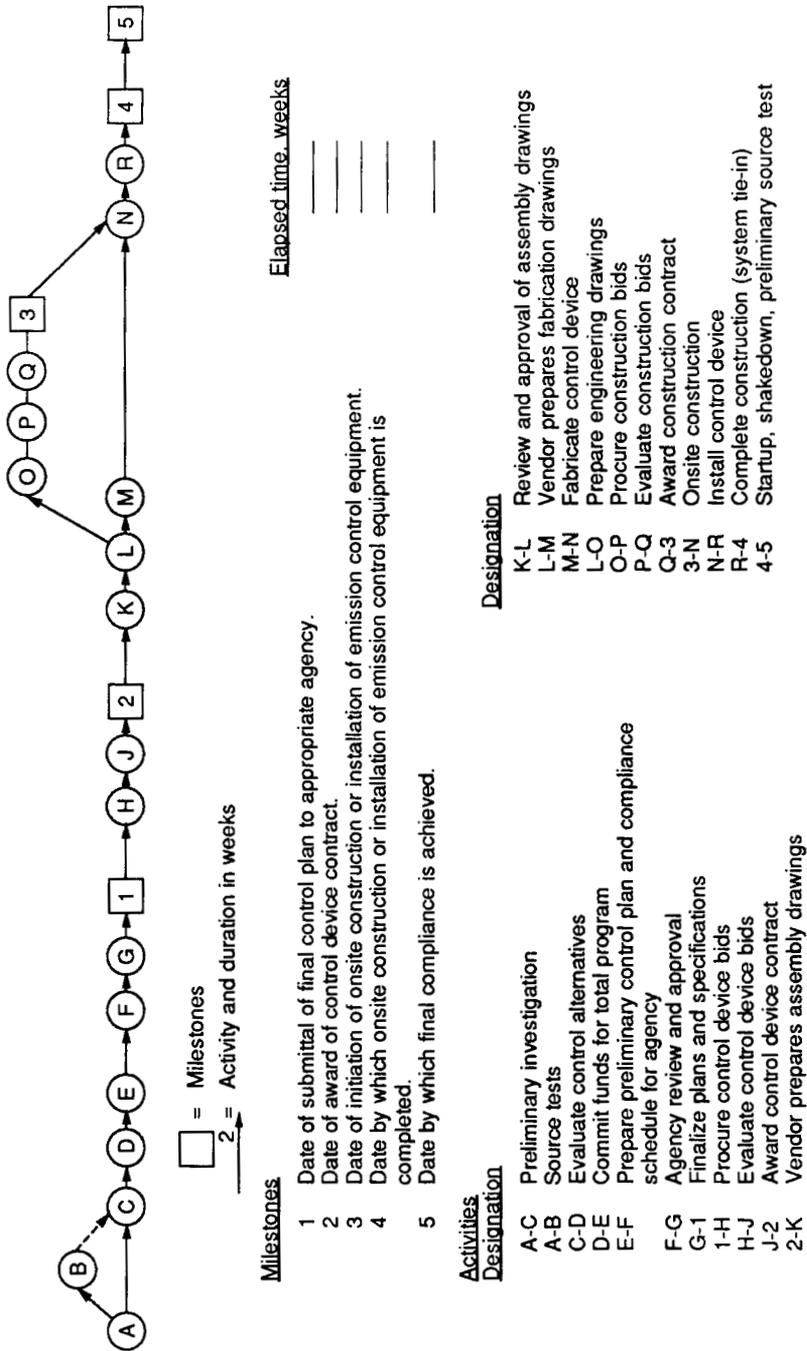


FIG. 4.2 Compliance schedule chart (4).

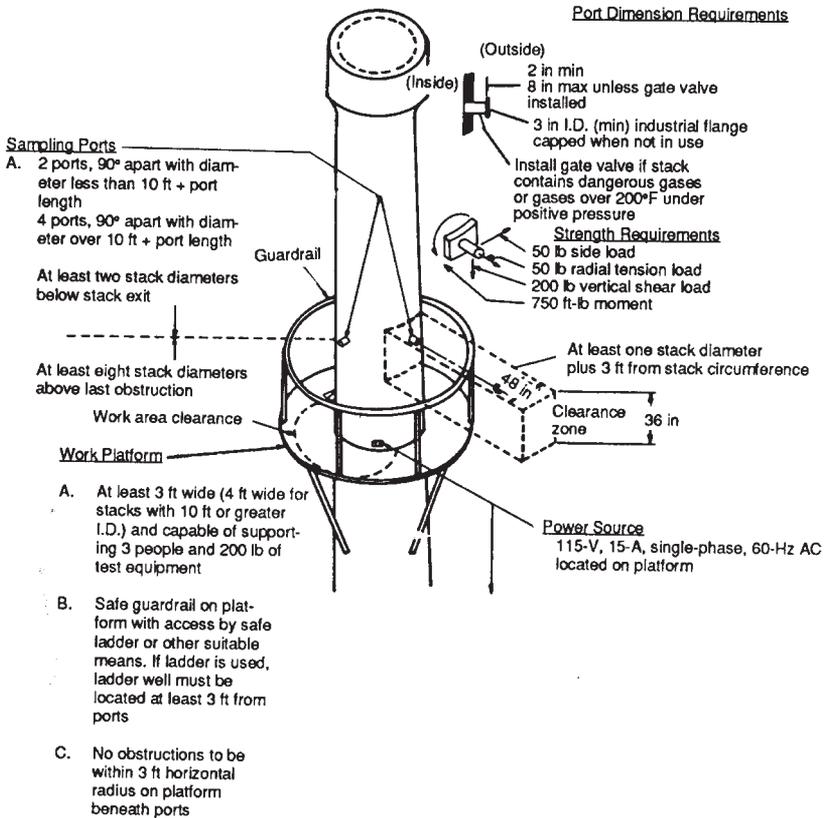


FIG. 4.3 Typical sampling point provisions (4).

medium. Alternate reference methods are published for special applications. In addition, some sampling system alterations may be required by other regulatory agencies.

Sampling for particulates is performed at several designated points, representing equal areas, in the stack. The most widely used sampling apparatus is illustrated in Figure 4.6. Selection of the sampling points must be sensitive to potential stratification of particulates; common problems are shown in Figure 4.7.

Sulfur Dioxide. Reference method 6 uses the sample train presented in Figure 4.8 and is applicable to all stationary sources of sulfur dioxide except sulfuric acid plants.

Sampling for sulfur dioxide requires only a single collection point located at the center of the stack or at least 1 m (3.28 ft) from the inner wall. In addition, the sample must be extracted at a constant rate, which requires adjustments for any changes in stack gas velocity.

Continuous Emission Monitoring. Continuous emission monitoring (CEM) consists of two steps: extracting or locating a representative sample, and analyzing the sample. The types of CEM methods are:

- Extractive method, in which gas is withdrawn from the stack, conditioned, and analyzed
- In-situ method, in which gas is not extracted but is monitored in the stack by the analyzer

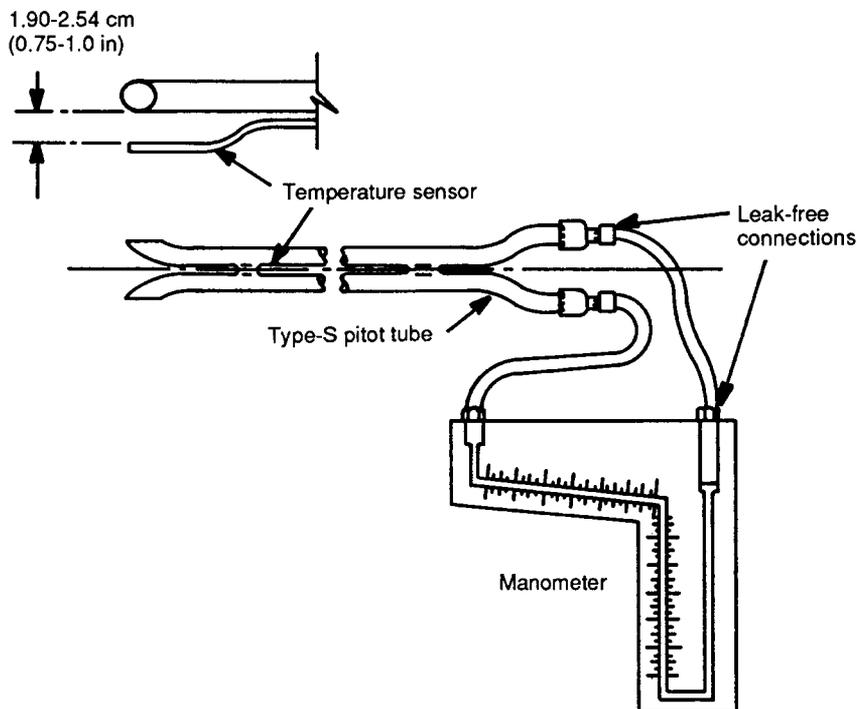
TABLE 4.3 Reference Sampling Methods (7)

Method 1—Sample and velocity traverses for stationary sources	Method 7E—Determination of nitrogen oxides emissions from stationary sources (instrumental analyzer procedure)
Method 2—Determination of stack gas velocity and volumetric flow rate (Type S Pitot tube)	Method 8—Determination of sulfuric acid mist and sulfur dioxide emissions from stationary sources
Method 2A—Direct measurement of gas volume through pipes and small ducts	Method 9—Visual determination of the opacity of emissions from stationary sources
Method 2B—Determination of exhaust gas volume flow rate from gasoline vapor incinerators	Method 10—Determination of carbon monoxide emissions from stationary sources
Method 3—Gas analysis for carbon dioxide, oxygen, excess air, and dry molecular weight	Method 10A—Determination of carbon monoxide emissions in certifying continuous emission monitoring systems at petroleum refineries
Method 3A—Determination of oxygen and carbon dioxide concentrations in emissions from stationary sources (instrumental analyzer procedure)	Method 11—Determination of hydrogen sulfide content of fuel gas streams in petroleum refineries
Method 4—Determination of moisture content in stack gases	Method 12—Determination of inorganic lead emissions from stationary sources
Method 5—Determination of particulate emissions from stationary sources	Method 13A—Determination of total fluoride emissions from stationary sources—SPADNS zirconium lake method
Method 5A—Determination of particulate emissions from the asphalt processing and asphalt roofing industry	Method 13B—Determination of total fluoride emissions from stationary sources—specific ion electrode method
Method 5B—Determination of nonsulfuric acid particulate matter from stationary sources	Method 14—Determination of fluoride emissions from potroom roof monitors of primary aluminum plants
Method 5D—Determination of particulate matter emissions from positive pressure fabric filters	Method 15—Determination of hydrogen sulfide, carbonyl sulfide, and carbon disulfide emissions from stationary sources
Method 5E—Determination of particulate emissions from the wool fiberglass insulation manufacturing industry	Method 15A—Determination of total reduced sulfur emissions from sulfur recovery plants in petroleum refineries
Method 5F—Determination of nonsulfate particulate matter from stationary sources	Method 16—Semicontinuous determination of sulfur emissions from stationary sources
Method 6—Determination of sulfur dioxide emissions from stationary sources	Method 16A—Determination of total reduced sulfur emissions from stationary sources (impinger technique)
Method 6A—Determination of sulfur dioxide, moisture, and carbon dioxide emissions from fossil fuel combustion sources	Method 16B—Determination of total reduced sulfur emissions from stationary sources
Method 6B—Determination of sulfur dioxide and carbon dioxide daily average emissions from fossil fuel combustion sources	Method 17—Determination of particulate emissions from stationary sources (instack filtration method)
Method 6C—Determination of sulfur dioxide emissions from stationary sources (instrumental analyzer procedure)	Method 18—Measurement of gaseous organic compound emissions by gas chromatography
Method 7—Determination of nitrogen oxide emissions from stationary sources	Method 19—Determination of sulfur dioxide removal efficiency and particulate matter, sulfur dioxide and nitrogen oxides emission rates
Method 7A—Determination of nitrogen oxide emissions from stationary sources	Method 20—Determination of nitrogen oxides, sulfur dioxide, and oxygen emissions from stationary gas turbines
Method 7B—Determination of nitrogen oxide emissions from stationary sources (ultraviolet spectrophotometry)	Method 21—Determination of volatile organic compounds leaks
Method 7C—Determination of nitrogen oxide emissions from stationary sources	
Method 7D—Determination of nitrogen oxide emission from stationary sources	

(continues)

TABLE 4.3 Reference Sampling Methods (*continued*)

Method 22—Visual determination of fugitive emissions from material sources and smoke emissions from flares	Method 27—Determination of vapor tightness of gasoline delivery tank using pressure-vacuum test
Method 24—Determination of volatile matter content, water content, density, volume solids, and weight solids of surface coating	Appendix B—Performance Specifications
Method 24A—Determination of volatile matter content and density of printing inks and related coatings	Performance Specification 1—Performance specifications and specification test procedures for transmissometer systems for continuous measurement of the opacity of stack emissions
Method 25—Determination of total gaseous nonmethane organic emissions as carbon	Performance Specification 2—Specifications and test procedures for SO ₂ and NO _x continuous emission monitoring systems in stationary sources
Method 25A—Determination of total gaseous organic concentration using a flame ionization analyzer	Performance Specification 3—Specifications and test procedures for O ₂ and CO ₂ continuous emission monitoring systems in stationary sources
Method 25B—Determination of total gaseous organic concentration using a nondispersive infrared analyzer	

**FIG. 4.4** Velocity measurement system (4, 7).

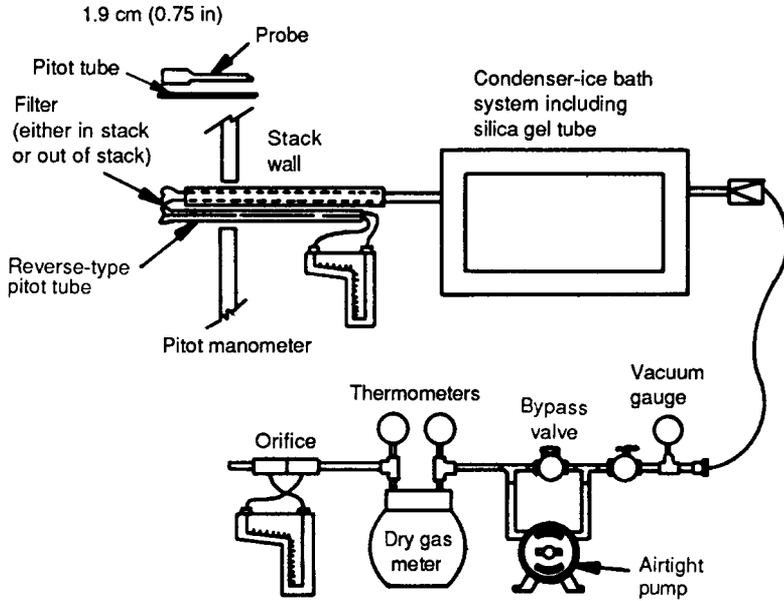


FIG. 4.5 Moisture sampling train (4, 7).

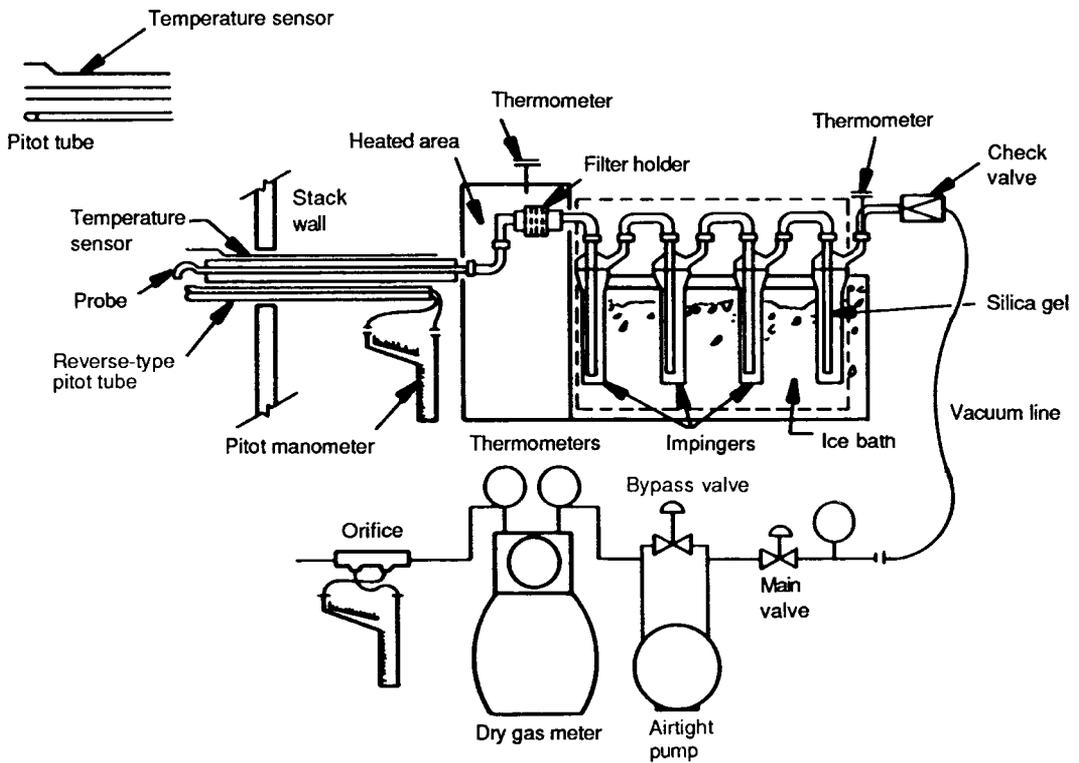


FIG. 4.6 Particulate sampling train (4, 7).

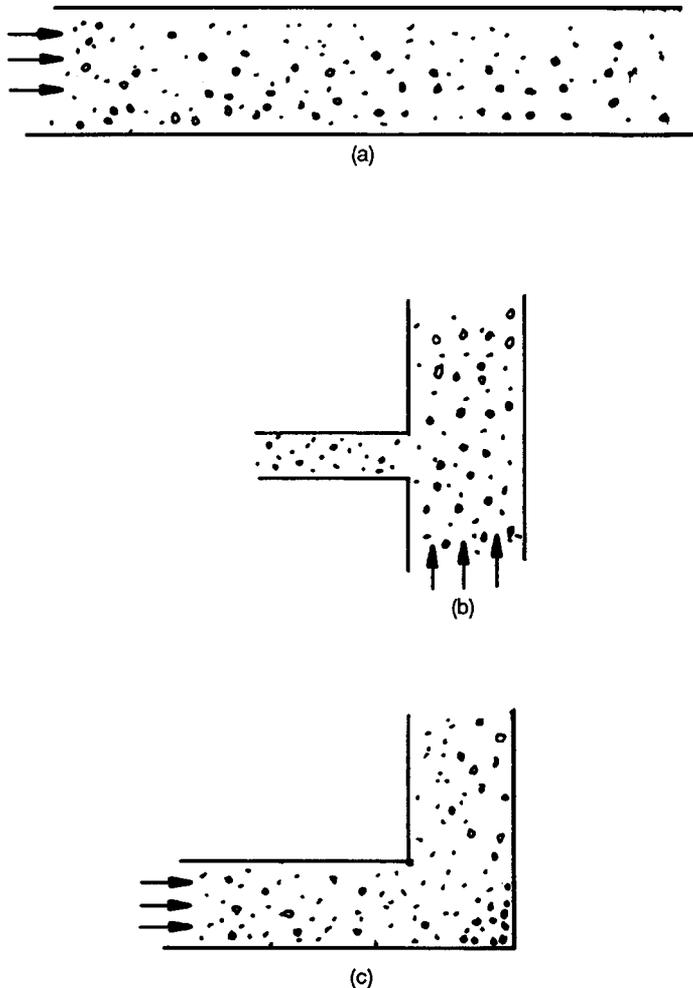


FIG. 4.7 Stratification of particulates. (a) Horizontal duct, stratification due to settling; (b) junctions, stratification due to poor mixing; (c) turns and obstruction, stratification due to inertia of particles and turbulence (4).

- Indirect parameter method, in which a surrogate or predictive parameter is validated for use in lieu of the (regulated) emission of interest

The determination of which method to use is dependent on the emission monitoring requirements, emission characteristics, stack characteristics, and availability of equipment and test methods to accomplish the analysis (111, 112).

A summary of the new source performance standards (NSPS) or “Standards of Performance for New Stationary Sources” criteria and the NSPS noncriteria for pollutant monitoring are presented in Tables 4.4 and 4.5, respectively (111). The latest version of 40 CFR 60 should be consulted for current information, specific provisions, averaging periods, reporting requirements, and other information (113).

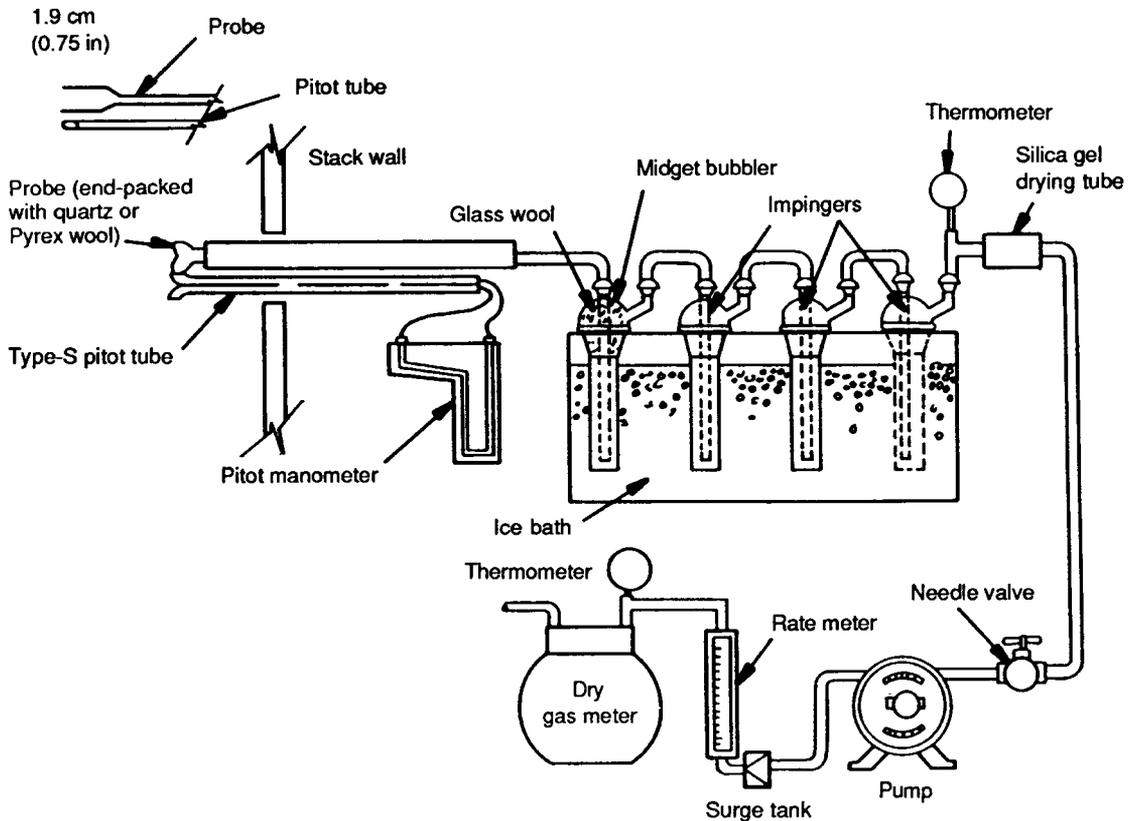


FIG. 4.8 Sulfur dioxide sampling train (4, 7).

Ambient Air Monitoring

Monitoring of ambient air has several applications. For example, the regulatory agency may require ambient air monitoring for specific pollutants, and, in fact, the limiting pollutant concentration may be stipulated for measurement at the property line. Ambient air monitoring also provides useful data on background levels and trends in air quality.

The specifics of ambient air monitoring programs are unique in their development and conduct. A common goal is to obtain a representative sample from an unconfined volume of air in the vicinity of one or more emission sources. The program further requires measurements for one or more primary and/or secondary pollutants.

Site Selection. The monitoring program objectives influence the number of monitoring sites, whereas meteorology and topography are primary factors in locating individual monitoring sites.

The simplest case is one in which there is a predominant wind directly over a uniform topography for an isolated plant site emitting a single pollutant that does not change composition in the atmosphere. Two monitoring sites are used: one upwind to provide background concentrations and one downwind to monitor the effects of the plant's emission. However, most sites have several variables with respect to meteorological and topographic factors and to off-site pollutant migration.

TABLE 4.4 NSPS Criteria for Pollutant Monitoring Requirements

Source Category and Type	Pollutant and Diluent Monitors	Compliance Averaging Period for CEMS Data	Excess Emissions Reporting Period
Fossil-Fuel-Fired Steam Generators > 250 × 10 ⁶ Btu/hour heat input	SO ₂ and NO _x (O ₂ or CO ₂ as diluent)	—	3 hours
	Opacity	—	6 minutes
Electric Utility Steam Generating Units > 73 MW (250 × 10 ⁶ Btu/hour) heat input	SO ₂ and NO _x emissions, SO ₂ percent removal (O ₂ or CO ₂ as diluent)	30 day rolling average (boiler operating days)	—
	Opacity	—	6 minutes
Industrial–Commercial–Institutional Steam Generators >29 MW (100 × 10 ⁶ Btu/hour) heat input	SO ₂ and NO _x emissions, SO ₂ percent removal (O ₂ or CO ₂ as diluent)	30-day rolling average (boiler operating days)	—
	Opacity	—	6 minutes
Small Industrial–Commercial–Institutional Steam Generators >2.9 MW <29 MW (10 to 100 × 10 ⁸ Btu/hour) heat input	SO ₂ emissions, SO ₂ percent removal (certain sources) (O ₂ or CO ₂ as diluent)	30-day rolling average (boiler operating days)	—
	Opacity	—	6 minutes
Municipal Waste Combustors >250 tons/day	SO ₂ with O ₂ as diluent, SO ₂ percent removal (certain sources)	24 hours	—
	NO _x emissions with O ₂ as diluent	24 hours	—
	CO with O ₂ as diluent	4 hour and 24 hour periods	—
	Opacity	—	6 minutes
Portland Cement Plants	Opacity	—	6 minutes
Nitric Acid Plants	NO _x emissions	—	3 hours
Sulfuric Acid Plants	SO ₂ emissions (O ₂ or CO ₂ monitors or other procedures to calculate emissions)	—	3 hours
Petroleum Refineries	—	—	—
Fluid Catalytic Cracking Unit Regenerators	CO	—	1 hour
	SO ₂ emissions SO ₂ removal, sulfur oxides (certain sources)	7-day rolling average	—
	Opacity	—	6 minutes
Fuel Gas Combustion Devices	SO ₂ with O ₂ as diluent, (H ₂ S in fuel gas as alternative)	—	3 hour rolling average

AIR QUALITY CONTROL

TABLE 4.4 NSPS Criteria for Pollutant Monitoring Requirements (*continued*)

Source Category and Type	Pollutant and Diluent Monitors	Compliance Averaging Period for CEMS Data	Excess Emissions Reporting Period
Claus Sulfur recovery Plant >20 LTD* with oxidation control system	SO ₂ with O ₂ as diluent	—	12 hours
Claus Sulfur recovery Plant >20 LTD with reduction control system	TRS with O ₂ as diluent or <i>dilution sampling system with oxidation and SO₂ with O₂ as diluent</i>	—	12 hours
Primary Copper Smelter	—	—	—
Dryer	Opacity	—	6 minutes
Roaster, smelting furnace, and copper converter	SO ₂	6 hour period	—
Primary Lead Smelter	—	—	—
Blast furnace, dross reverberatory furnace, or sintering machine discharge	Opacity	—	6 minutes
Sintering machine, electric smelting furnace and converter	SO ₂	2-hour period	—
Ferroalloy Product Facilities, submerged electric arc furnaces	Opacity	—	6 minutes
Steel Plants—Electric Arc Furnaces and Argon Decarburization Vessels (exceptions for certain controls)	Opacity	—	6 minutes
Steel Plants—Electric Arc Furnaces and Argon Decarburization Vessels (exceptions for certain controls)	Opacity	—	6 minutes
Kraft Pulp Mills	—	—	—
Recovery Furnaces	Opacity	—	6 minutes
Glass Manufacturing Plants	Opacity	—	6 minutes
Lime Manufacturing Plants, rotary lime kilns	Opacity	—	6 minutes
Phosphate Rock Plants, dryers, calciners, and grinders	Opacity	—	6 minutes
Onshore Natural Gas Processing, sweetening units	Velocity (also SO ₂ if oxidation control system or reduction control system followed by incinerator is used)	—	24 hours
Calciners and Dryers in Mineral Industries (with dry control devices)	Opacity	—	6 minutes

*LTD = Long tons per day

TABLE 4.5 NSPS Noncriteria for Pollutant Monitoring Requirements

Source Category and Type	Pollutant and Diluent Monitors	Excess Emissions Reporting Period
Petroleum Refineries	—	—
Fuel Gas Combustion Devices	H ₂ S in fuel gas (as alternative to SO ₂ with O ₂ in emissions)	3-hour rolling average
Claus Sulfur recovery Plant >20 LTD with reduction control system	TRS with O ₂ as diluent or dilution sampling system with oxidation and SO ₂ with O ₂ as diluent	12 hours
Kraft Pulp Mills—Emissions from recovery boilers, lime kilns, digester system, brown stock washer system, evaporator, and condensate stripper systems	TRS with O ₂ as diluent	12 hours
VOC Emissions from Polymer Industry	—	—
Carbon absorbers	VOC	3 hours
Condensers	Temperature or VOC	3 hours
Flexible Vinyl and Urethane Coating and Printing—Rotogravure printing lines with recovery units	VOC	3 hours
Synthetic Organic Chemical Manufacturing Industry—Units with air oxidation reactors	Specified parameters or <i>VOC CEMS</i>	3 hours
Onshore Natural Gas Processing, sweetening units	velocity (TRS and/or SO ₂ as alternative for certain sources)	24 hours
Petroleum Refinery Waste Water System—Units with carbon absorbers	VOC	3 hours
Magnetic Tape Coating Facilities—Units with carbon absorbers	VOC inlet and outlet streams for certain sources	3 hours
Polymeric Coating of Supporting Substrates—Units with carbon absorbers	VOC inlet and outlet streams for certain sources	3 hours

Monitoring Equipment. Most ambient air monitoring programs include meteorological monitoring of wind velocity and direction. In some cases, temperature sensing at multiple elevations is used to monitor stability of the air mass. Particulates and sulfur dioxide are the pollutants most often monitored.

Particulate monitoring is usually accomplished with a high-volume sampler, which is a vacuum-type device that draws air through a filter used for particulate analysis. This sampler provides average concentrations over a period of up to 24 hours. The total air flow is determined by the difference in rotameter readings at the start and conclusion of the test.

Monitoring for gaseous pollutants, such as sulfur dioxide, is accomplished with dynamic samplers for average concentrations over a 24-hour period and with static samplers for longer periods, e.g., 30 days. A dynamic sampling bubble train for sulfur dioxide monitoring is shown in Figure 4.9. A sulfation plate for static sampling of sulfur dioxide is presented in Figure 4.10.

RISK MANAGEMENT

Section 112(r) of the Clean Air Act and the Chemical Accident Prevention Provisions, 40 CFR 68, establish the legislative and regulation bases for risk management planning. The regulation identifies the substances to be regulated, presents a three-tier compliance approach (Programs 1, 2, and 3), and further defines specific requirements of the risk management plan.

The regulations apply only to stationary sources with processes that contain threshold quantities of the listed regulated substances. The list includes 77 toxic chemicals with threshold quantities in the range of 500 to 20,000 lbs (225 to 9070 kg) and 63 flammable substances of 10,000 lbs (4535 kg). For example, chlorine is regulated when more than 2,500 lbs (1135 kg) is on-line and stored on-site (114).

The risk management plan requirements increase in complexity with higher program levels. Program 1 applies when there has been no off-site accident history, there are no public receptors in the worst-case scenario, and emergency response is coordinated with local responders. The site/process is Program 2 when it is not eligible for Program 1 or 3. Distinctions of Program 3 are that the process is subject to the U.S. Occupational Safety and Health Administration, Process Safety Management standard (29 CFR 1910.19) and the process is in SIC code 2611, 2812, 2819, 2821, 2865, 2869, 2879, or 2911.

The risk management planning process begins with the determination that threshold quantities of regulated substances are on-site. If present, the next step is to perform a hazard assessment, which defines a five-year accident history and includes an off-site consequence analysis (115). In all cases, a worst-case release scenario is analyzed. For Programs 2 and 3, an alternate scenario also is analyzed. The consequence analyses use air dispersion modeling to define off-site impacts on the public and the environment. The required parameters for dispersion modeling are presented in Table 4.6. The hazard assessment establishes the basis for development of prevention and emergency response programs.

The prevention program for Program 1 is simply a certification that no additional measures are needed to prevent off-site consequences. Program 2 and 3 prevention programs address safety information, process hazardous analysis, operations procedures, training, maintenance, compliance audits, and incident investigations. In addition, Program 3 prevention programs include management of change, prestartup review, employee participation, and hot work permits.

For the emergency response program, Program 1 requires only a commitment to coordinate with local responders. Program 2 and 3 plans are detailed and site-specific to regulated substance release. Program provisions must address public notification, first-aid and emergency medical treatment, deployment and use of emergency response equipment, and training in a written plan.

The resulting risk management plan encompasses the hazard assessment, prevention program, emergency response program, and plan certification. The plan is required by June 21, 1999 and thereafter when a regulated substance is first present above a threshold quantity. When new substances are added to the regulated list, the plan must be complete within three years of the regulation. The regulation also specifies times for review and update of the plan.

PARTICULATE CONTROLS

Particulate matter is generated by a variety of physical and chemical mechanisms from numerous sources and is composed of finely dispersed liquids and solids. Control of particulate emissions is achieved either by

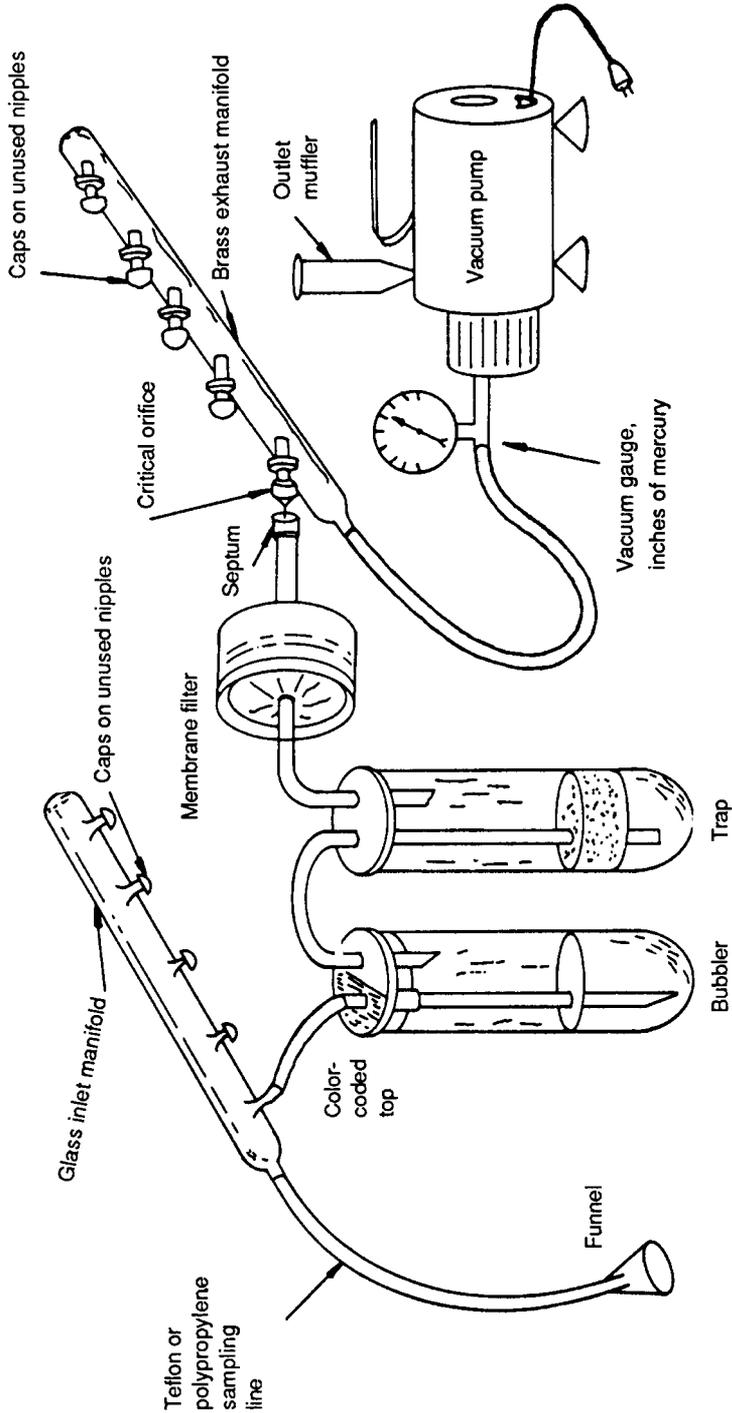


FIG. 4.9 Dynamic sulfur dioxide sampling unit (4, 7).

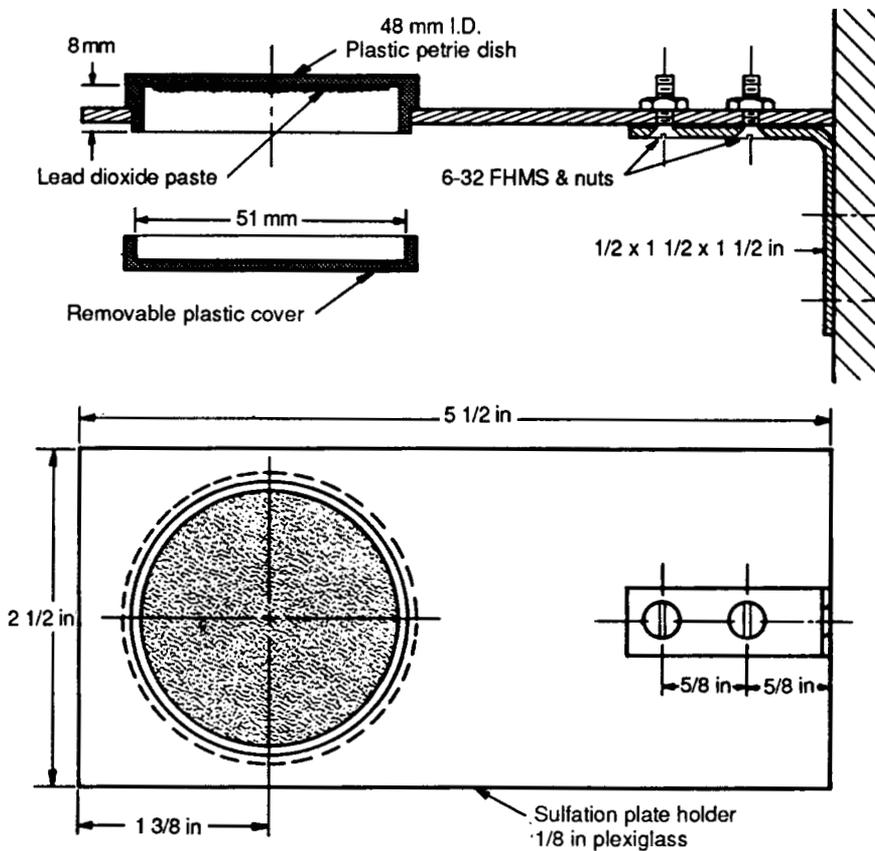


FIG. 4.10 Static sulfur dioxide sampling unit.

prevention of particle generation or by collection of entrained particles in the facility's gas emission. Common control devices include settling chambers, inertial separators, impingement separators, wet scrubbers, fabric filters, and electrostatic precipitators.

Source Controls

Alternatives for source control are limited, especially with respect to total prevention of particle generation, which normally relies on fuel switching. Thus, the more common strategy is process modification or optimization to reduce the emission quantity and/or to improve particle collectibility.

Fuel Substitution. Energy or fuel substitution can be an effective technique for reducing particulate emissions, but its use is contingent on fuel availability and the emission reduction objectives. This approach has been very useful for small and old sources, where new air pollution control expenses could have been prohibitive to continued operation.

Recognizing that there are site-specific combustion characteristics and variable fuel properties, Table 4.7 illustrates an emission factor analysis for a boiler fired by pulverized coal. In this case, natural gas reduces

TABLE 4.6 Required Parameters for Dispersion Modeling (115)

Worst-Case Scenario	Alternative Scenario
<p>Endpoints Endpoints for toxic substances are specified in reference guidance. For flammable substances, endpoint is overpressure of 1 psi for vapor cloud explosions,</p>	<p>Endpoints for toxic substances are specified in reference guidance. For flammable substances, endpoint is overpressure of 1 psi for vapor cloud explosions, or Radiant heat level of 5 kW/m² for 40 seconds for heat from fires (or equivalent dose), or Lower flammability limit (LFL) as specified in NFPA documents or other generally recognized sources</p>
<p>Wind speed/stability Use wind speed of 1.5 m/s and F stability class unless demonstrated that local meteorological data applicable to the site show a higher minimum wind speed or less stable atmosphere at all times during the previous three years. If demonstrated, these minimums may be used. Reference guidance assumes 1.5 m/s and F stability.</p>	<p>For site-specific modeling, use typical meteorological conditions for the site. If reference guidance is used, assume wind speed of 3 m/s and D stability.</p>
<p>Ambient temperature/humidity For toxic substances, use the highest daily maximum temperature and average humidity for the site during the past three years. Reference guidance assumes 25°C (77°F) and 50% humidity.</p>	<p>May use average temperature/humidity data gathered at the site or a local meteorological station. Reference guidance assumes 25°C (77°F) and 50% humidity.</p>
<p>Height of release For toxic substances, assume a ground-level release.</p>	<p>Release height may be determined by the release scenario. Reference guidance assumes a ground-level release.</p>
<p>Topography Use urban or rural topography, as appropriate</p>	<p>Use urban or rural topography, as appropriate</p>
<p>Dense or neutrally buoyant gases Tables or models used for dispersion of regulated toxic substances must appropriately account for gas density. Reference guidance provides tables for buoyant and dense gases.</p>	<p>Tables or models used for dispersion must appropriately account for gas density. Reference guidance provides tables for buoyant and dense gases.</p>
<p>Temperature of released substance Consider liquids (other than gases liquefied by refrigeration) to be released at the highest daily maximum temperature, based on data for the previous three years, or at process temperature, whichever is higher. Assume gases liquefied by refrigeration at atmospheric pressure are released at their boiling points. Reference guidance allows substitution of 25°C (77°F) or the boiling point of the released substance.</p>	<p>Substances may be considered to be released at a process or ambient temperature that is appropriate for the scenario. Reference guidance allows substitution of 25°C (77°F) or the boiling point of the released substance.</p>

TABLE 4.7 Particulate Emission Reduction Potential of Alternate Energy Sources (δ)

Energy source	Assumed particulate emission control, %	Particulate emission, ng/J (lb/10 ⁶ Btu) of delivered energy
Natural gas	0	21(0.048)*
No. 6 fuel oil	0	47(0.108)†
Bituminous coal	98.0	140(0.320)

*Based on an emission factor of 15 lb/10⁶ ft³ for gas with a heating value of 37,300 kJ/m³ and on an estimated conversion efficiency of 31.4%.

†Based on an emission factor of 5 lb/1000 gal for oil to 0.3% sulfur content with a heating value of 42,000 kJ/L and on an estimated conversion efficiency of 30.8%.

emissions by 55 and 85% with respect to No. 6 fuel oil and bituminous coal. The development of economical synthetic fuels will improve the availability of clean fuels.

Another alternative is to shift from on-site power generation to purchase of electricity. However, energy substitution is not normally a cost-effective alternative except in the case of smaller facilities where relatively high costs of particulate control would influence the analysis.

Process Modifications. Modifications of process feed materials, process unit functions, and process variables to eliminate or reduce particulates are key elements of optimization controls. These efforts may also identify changes to reduce the volume of exhaust gas and/or alter the particle size distribution. All of these actions may have the advantage of reducing treatment costs and providing a broader range of treatment devices to consider (δ).

The physical properties of feed materials, such as particle size, chemical composition, and moisture content, may have significant effects on particulate emissions, depending on the specific industrial process. In general, where heated air is used for drying, fine particles in the process feed material cause an increase in particulate emissions. Thus, screening or cleaning of raw feed material can reduce the emissions from the drying process.

In some cases, a process step(s) can be eliminated. For example, the transfer of material from one process operation to another may contribute to fugitive particulate losses. Process change options include longer conveyors, pneumatic instead of open conveyors, or combination of process steps.

Another alternative is to incorporate processes that wet or agglomerate the feed and/or process materials. The benefits are an increase in the effective particle size and in the efficiency of emission control devices.

Settling Chambers

Simple gravity separation is the basis of one of the oldest collection devices—the settling chamber (*1, 8, 9*). As illustrated in Figure 4.11, a settling chamber is a long, enlarged section in the exhaust system. As the cross-sectional area increases, there is a corresponding decrease in the exhaust gas velocity, which permits coarser particles to fall from the air suspension. These features are illustrated in Figure 4.12.

Settling chambers offer the advantages of modest construction, simple operation, dry collection, and small pressure drops. Structural space requirements are a significant disadvantage and are generally the limiting factor in use of settling chambers as primary collection methods.

The design of a settling chamber is based on the settling velocity of the particles to be removed. Generally, chambers will have flow-through velocities less than 3 m/s (10 ft/s) and will effectively remove particles greater than 50 μm in size. Efficiencies may be improved by use of horizontal plates or shelves (Figure

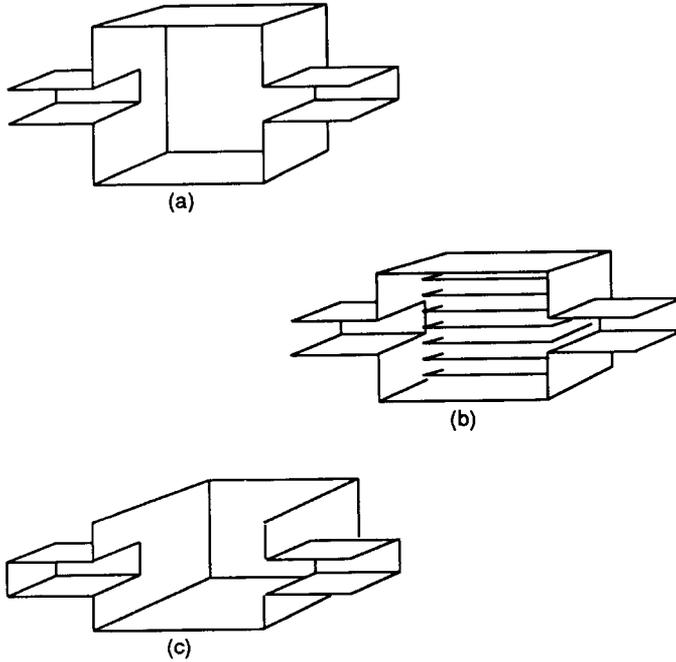


FIG. 4.11 Settling chamber configurations. (a) Boxlike; (b) plate box; (c) flat box.

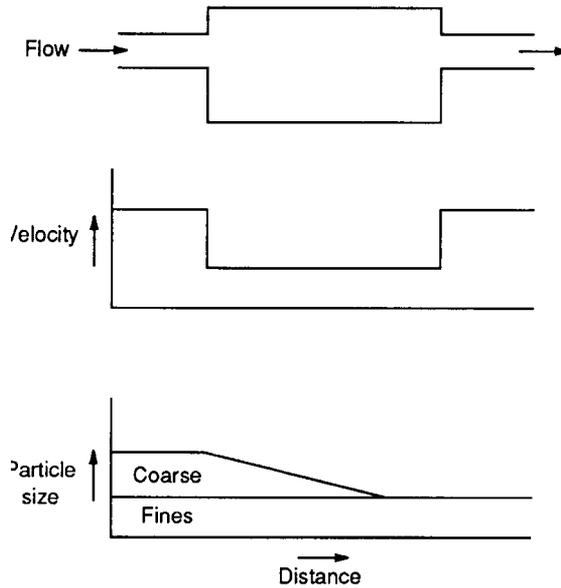


FIG. 4.12 Settling chamber performance features.

4.11b) or by increasing the width-to-depth ratio (Figure 4.11c); these approaches reduce the required particle settling distance. Dry collectors, such as scrappers and screw conveyors, are used to collect the settled particles.

The following equations are used to determine detention time, terminal velocity, pickup velocity, and efficiency. The detention (residence) time of the gas stream in the settling chamber is

$$\theta = \frac{WLH}{Q}$$

where θ = detention time, s
 W = chamber width, m
 L = chamber length, m
 H = chamber height, m
 Q = flow rate, m³/s

For the particle size of concern, the terminal settling velocity is

$$V_t = \frac{g\rho_p d^2}{18\mu}$$

where V_t = terminal velocity, m/s
 g = acceleration of gravity, 9.806 m/s²
 ρ_p = particle density, kg/m³
 d = particle diameter, in
 μ = gas viscosity, kg/m·s

Thus, the design particle will fall predicted distance h , equal to the product of the detention time and terminal velocity.

The design should also consider the limiting velocity which will pick up deposited particles and reentrain them into the gas stream. The following theoretical equation is used:

$$V_p = \frac{[0.4gd(\rho_p - \rho_g)]^{0.5}}{3\rho_g}$$

where V_p = pickup velocity, m/s
 ρ_g = gas density, kg/m³

The percent removal efficiency E of the settling chamber design for the referenced particle size is

$$E = \frac{h}{4} \times 100 = \frac{V_t WL}{Q} \times 100$$

Current practice tends to use settling chambers as a pretreatment device to remove coarse and abrasive particles for the protection and more effective use of more sophisticated collection equipment.

Inertial Separators

Inertial separators work by imparting centrifugal force to the particles to be removed. They are widely used collection devices for particles larger than 15 μm . Particles between 5 and 15 μm may be removed satis-

factorily using special systems. Conventional inertial separators are categorized as cyclone or mechanical units (1, 8).

Inertial separators are often used to remove heavier particles before further treatment. Due to the many varieties of units available, the design engineer should consult with equipment manufacturers during the design development.

Cyclone Separators. A cyclone is a versatile and low-cost separator that separates particles from the gas stream without the use of moving parts. A wide variety of units are available; Figure 4.13 illustrates four types based on tangential or axial gas inlet, gas outlet, and dust outlet configurations.

The vortex required for particle separation is created by injecting the gas stream into the cylinder section through tangential inlets or by use of stationary vanes with axial inlets. Most cyclones have a double vortex gas path that expels particles along the cylinder walls at the point at which the vortex changes direction.

The performance of cyclone separators is primarily dependent on particle size, and system design takes into consideration collection efficiency, pressure drop, and unit size. The collection efficiency increases with higher pressure drops associated with such factors as higher inlet gas velocities and larger unit features, such as cyclone body length.

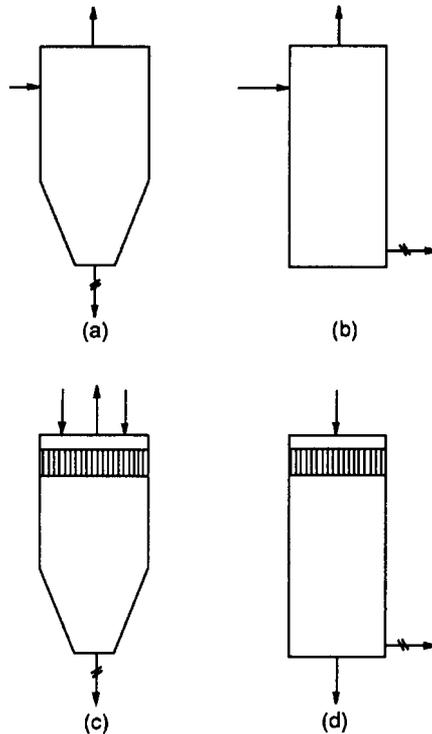


FIG. 4.13 Types of cyclone separators. (a) tangential inlet, axial outlet, axial dust outlet; (b) tangential inlet, axial outlet, tangential dust outlet; (c) axial inlet, axial outlet, axial dust outlet; (d) axial inlet, axial outlet, tangential dust outlet.

The cyclone efficiency also increases with increase in particle density, number of gas revolutions, ratio of body diameter to outlet diameter, and smoothness of the inner cyclone wall. Efficiency will increase with a decrease in gas viscosity, cylinder diameter, inlet duct width or area, outlet diameter, and gas density.

Single Cyclone Separators. The basic configuration for a single cyclone separator is the tangential inlet, axial outlet, and axial dust outlet type unit, as shown in Figure 4.14. However, the following discussions and relationships are fundamental to all cyclone separators.

The inlet velocity should be high enough to provide a high separation efficiency without creating excessive turbulence. Inlet velocities typically range from 10 to 25 m/s (30 to 85 ft/s) (9).

Cyclones are sized using a separation factor based on the premise that the centrifugal force applied to particles varies as the square of the inlet velocity and inversely as the radius of the cyclone. This relationship is expressed as (1):

$$S = \frac{V_i^2}{r_c g}$$

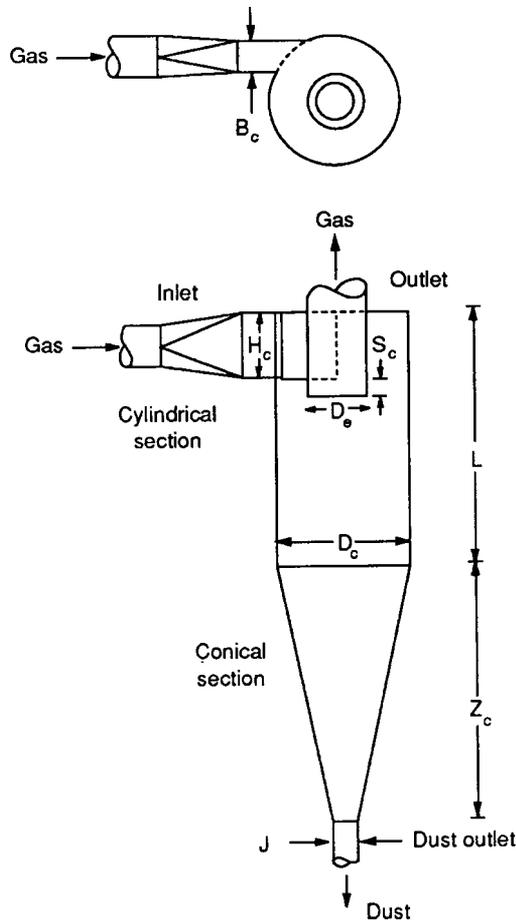


FIG. 4.14 Single cyclone separator.

where S = separation factor

V_i = gas inlet velocity, m/s

r_c = cylinder radius, m

g = acceleration of gravity, 9.806 m/s²

Though not expressed by a specific equation, collection efficiency generally varies directly with the separation factor.

The pressure drop across a cyclone is the result of several factors, including inlet and outlet losses, internal wall friction, and kinetic energy in the outlet gas. The most accurate determination is made through actual or prototype testing.

Pressure drop may be approximated by the equation (1):

$$F = \frac{KB_c H_c}{D_e^2}$$

where F = cyclone friction loss, number of cyclone inlet velocity heads

K = empirical constant

B_c = gas inlet width, m

H_c = gas inlet height, m

D_e = gas outlet diameter, m

Values of K range from 7.5 to 18.4, with 16 being one typical value.

Cyclone performance may be correlated with various parameters using numerous theoretical models (1, 8–15). Common practice is to use the concept of cut size d_c , which is defined as the particle diameter collected with 50% efficiency. Smaller particles will be less effectively collected, and larger-diameter particles will be collected more efficiently. The critical particle diameter is that size collected with 100% efficiency.

A popular cyclone performance model involves the calculation of the 50% particle cut diameter d_c as follows (1):

$$d_c = \left[\frac{9\mu B_c}{2\pi N V_i (\rho_p - \rho_g)} \right]^{0.5}$$

where d_c = Particle diameter collected with 50 percent efficiency, μm

μ = gas viscosity, kg/m·s

B_c = gas inlet width, m

N = effective number of gas turns

V_i = gas inlet velocity, m/s

ρ_p = particle density, kg/m³

ρ_g = gas density, kg/m³

The number of gas turns may range from 0.5 to 10 with a value of 5 expected for high-efficiency cyclones. The N value is approximated by the following equation:

$$N = \frac{\pi(2L + Z_c)}{H_c}$$

where L = cylinder length, in

Z_c = cone length, m

H_c = gas inlet height, m

High-Efficiency Cyclone Separator. By increasing the body length and decreasing the body diameter from that of a simple cyclone, high-efficiency cyclones are used for particles in the 5- to 10- μm size. These physical features, relative to the simple cyclone, tend to increase the cylinder length, the inlet velocity, the detention time, and the centrifugal force on the particles, and to decrease the cylinder diameter.

As was illustrated by Figure 4.14, a high-efficiency cyclone consists of a cylinder with a rectangular, tangential gas inlet; a tubular, axial gas outlet; and a conical lower section with an axial particle outlet. Typical dimensional ratios for this unit are (I):

Cylinder diameter	D_c
Cylinder length	$L = 2D_c$
Cone length	$Z_c = 2D_c$
Gas inlet width	$B_c = 0.25D_c$
Gas inlet height	$H_c = 0.5D_c$
Gas outlet length	$H_c + S_c = 0.625D_c$
Gas outlet diameter	$D_e = 0.5D_c$
Particle outlet	$J = 0.25D_c$

Mechanical Separators. Using centrifugal force supplied by a rotating vane or impeller, particulates move rapidly to the separator's perimeter for removal. Mechanical separators have somewhat higher efficiencies than simple gravity separators. Several types of separators are available in the marketplace.

The advantages of mechanical separators are their compactness, collection efficiency of smaller particles, and effectiveness when many units are used. However, they do not perform well when exposed to particles that tend to cake or otherwise accumulate on the rotating blade, and there is the disadvantage of energy use requirements. By operating at low speeds of 400 to 800 rpm, the effects of abrasion and material buildup are minimized.

Impingement Separators

Mists and dry particles will impinge on a surface due to their greater inertia when an exhaust stream is deflected around a surface. The simplest type of impingement separators are baffled chambers, as illustrated in Figure 4.15. Various devices with simple to complex patterns use plates, cylinders, or other surfaces to collect particles.

The collection of mists is the best application of impingement separators. The collected liquid forms a film on the surface and then drips into the collection chamber. Since dry particles may be reentrained as they fall from the collecting surface, liquid sprays are used to wash the surface. The wetted surface also improves particle collection.

Impingement separators offer numerous options to the design engineer. They have potential for use as pretreatment devices or as the primary collection equipment.

Wet Scrubbers

Using a variety of methods, wet scrubbers are collection devices that wet particles in order to remove them from the gas stream. They utilize inertial impaction and/or Brownian diffusion as the particle collection mechanism, and droplets, sheets, and jets as the liquid collection mechanism. Table 4.8 lists primary advantages and disadvantages of wet scrubbers with respect to fabric filters and electrostatic precipitators.

In Table 4.9, major categories of wet scrubbers are listed in order of increasing performance capabilities and energy requirements for nonmechanical and then for mechanically aided units. Discussions will follow on the more popular types of scrubbers.

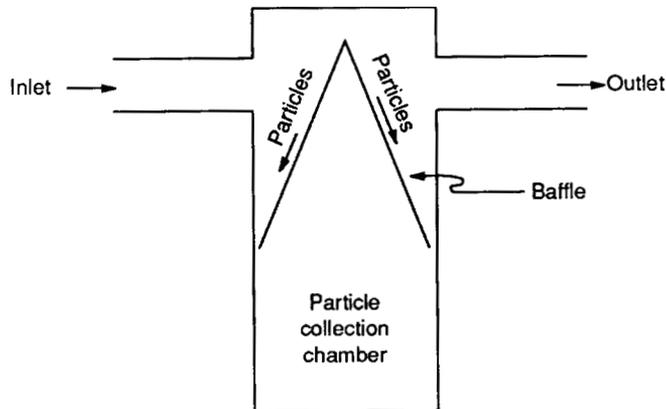


FIG. 4.15 Baffled chamber impingement separator.

The primary design features of wet scrubbers are penetration (efficiency) and pressure drop across the device, which is a function of the liquid-to-gas ratio. Tables 4.10 and 4.11 show typical ranges of the liquid-to-gas ratio and pressure drop for selected scrubber types.

A secondary design feature is the use of a demister to remove liquid droplets, wetted solids, and/or condensed mist. The entrained mist is collected by features that make the gas stream change its flow direction, resulting in coalescence and gravity separation. Figure 4.16 illustrates alternate features of a baffle-type impingement demister, which is a very common and simple method.

Operation and maintenance practices for wet scrubber installations are essential to satisfy not only design objectives but also other site specifics, such as air quality control, employee health, and general safety. Most wet scrubber component failures (such as clogged spray nozzles) result from abrasion, corrosion, chemical scaling, sedimentation, and wear or alignment on moving parts (8).

Wet scrubbers typically use water as the cleaning liquid. Water usage and wastewater disposal requirements are important factors in the evaluation of a scrubber alternative, and both are influenced by the quantity of particles collected, the size distribution of the particles, and the presence of dissolved contaminants in the wastewater. Many applications recirculate the scrubber liquid, which reduces consumption demand but will lead to higher particulate and dissolved solids concentrations, requiring treatment and disposal.

Liquid recirculating scrubber systems usually include a makeup water source and pump, a scrubber efflu-

TABLE 4.8 Advantages and Disadvantages of Wet Scrubbers*

Advantages	Disadvantages
Absorbs gas phase emissions	Condensate plume suggests pollution
Compact size	Corrosion
Efficient through wide loading range	Inefficient with high-temperature gases
Insensitive to moisture content	Requires high power input
Low capital cost	Waste scrubber liquid handling required
Low operating and maintenance cost	
Reentrainment rare	
Versatility for hazardous emissions	

*Prepared with respect to fabric filters and electrostatic precipitators

TABLE 4.9 Major Types of Wet Scrubbers (δ)

Category	Particle capture mechanism	Liquid collection mechanism	Types of scrubbers
Preformed spray scrubbers	Inertial impaction	Droplets	Spray towers Cyclonic spray towers Vane-type cyclonic towers Multiple-tube cyclones
Packed-bed scrubbers	Inertial impaction	Sheets, droplets	Standard packed-bed scrubbers Fiber-bed scrubbers Moving-bed scrubbers Crossflow scrubbers Grid-packed scrubbers
Plate scrubbers	Inertial impaction Diffusional impaction	Droplets, jets, and sheets	Perforated-plate scrubbers Impingement-plate scrubbers Horizontal impingement-plate (baffle) scrubbers
Venturi scrubbers	Inertial impaction Diffusional impaction	Droplets	Standard venturi scrubbers Variable-throat venturi scrubbers: flooded disk, plumb bob, movable blade, radial flow, variable rod
Orifice scrubbers	Inertial impaction Diffusional impaction	Droplets	Orifice scrubbers
Mechanical scrubbers	Inertial interception	Droplets and sheets	Wet fans Disintegrator scrubbers

ent pump, a settling basin or pond, and a pump for recirculating treated water. Some installations require multiple settling units with sludge collectors and sludge disposal facilities. Other system features may include chemical treatment for solids precipitation, pH control, and gravity or pressure filtration.

Preformed Spray Scrubbers. The principal features of preformed spray scrubbers are their low energy requirements and their low efficiency for removing particles less than $5\ \mu\text{m}$ in diameter.

Spray Tower. A spray tower or chamber is the simplest type of wet scrubber in that it consists of a shell wherein liquid droplets fall by gravity through the gas stream, flowing toward a demister and the top gas outlet. This system is illustrated in Figure 4.17.

TABLE 4.10 Typical Liquid-to-Gas Ratios for Wet Scrubbers (δ)

Scrubber type	Liquid-to-gas ratio	
	L/m ³	Gal/1000
Spray tower	1.30–2.70	9.7–20
Cyclone	0.70–1.30	5.2–9.7
Packed bed	0.10–0.50	0.75–3.7
Moving bed	1.30–2.70	9.7–20
Impingement plate	0.40–0.70	3.0–5.2
Venturi	0.70–1.00	5.2–7.5

TABLE 4.11 Typical Wet Scrubber Pressure Drop (δ)

Scrubber type	Pressure drop	
	kPa	in H ₂ O
Spray tower	0.25–0.5	1.0–2.0
Cyclone	0.25–0.8	1.0–3.2
Packed bed	0.25–2.0	1.0–8.1
Impingement plate	0.25–2.0	1.0–8.1
Venturi	1.5–18	6.1–73
Orifice	0.5–4.0	2.0–16

Cyclone Scrubber. A number of cyclone scrubber devices are available, ranging from a simple cyclonic spray tower to specially fabricated multistage units. Common features include a tangential gas inlet to a cylindrical vessel and a lower axial particle and/or liquid discharge outlet.

A cyclonic spray tower uses the features of a cyclone separator and simple spray tower, as shown in Figure 4.18. A modification is the vane-type cyclonic tower, which uses internal vanes to impact the cyclonic motion. Both of these systems function with an upward gas flow. A downward gas flow is used in a multiple-tube cyclone scrubber, which consists of multiple minitubes with separate liquid supply.

Packed-Bed Scrubbers. The characteristics of packed bed scrubbers include introduction of liquid near the top of the unit followed by a trickling down of the liquid to the packed bed. The liquid and gas flows may be concurrent, crosscurrent, or countercurrent. Also, there may be one or more layers (stages) of packing.

The collection efficiency depends upon the contact time of the gas stream on the packed bed. Coarsely packed beds are used to remove large, coarse dusts and mists; the converse is true for finely packed beds, which are also more prone to clogging with higher particulate concentrations in the gas stream. Packing includes coke, crushed rock, raschig or other rings, ben or other saddles, and other materials and shapes.

Concurrent packed-bed scrubbers are usually the most efficient for smaller particles, but they tend to have higher pressure drops. The cross-current scrubber requires less liquid flow, usually has a lower pressure drop, and tends not to clog. The countercurrent type requires the most liquid flow and is best suited for higher loadings. Figure 4.19 illustrates a two-stage countercurrent scrubber.

Plate Scrubbers. A plate (tray) scrubber consists of a tower containing one or more perforated plates; flow is countercurrent with the gas entering near the bottom of the tower. Generally, the plates have impingement

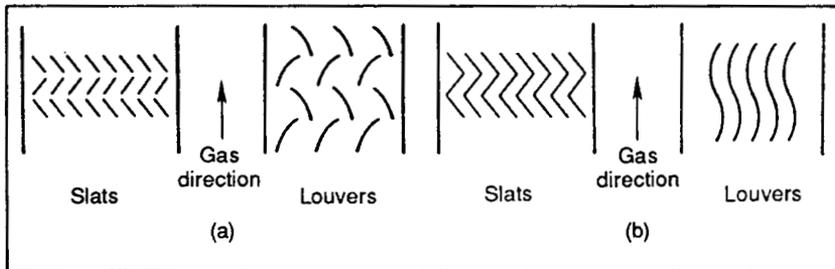


FIG. 4.16 Baffle-type demisters. (a) Open-vane design; (b) closed-vane design (16).

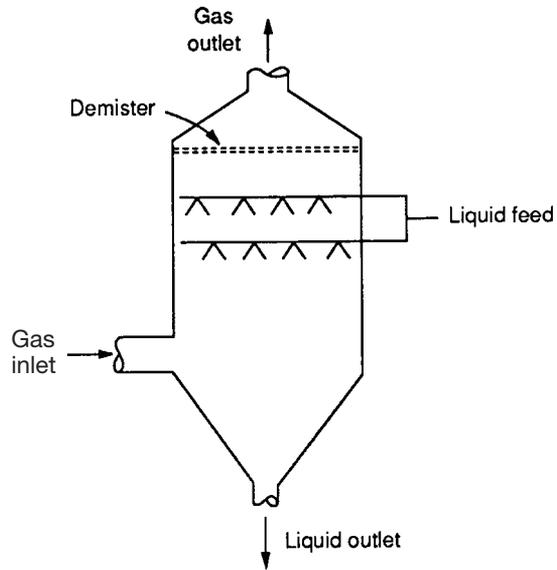


FIG. 4.17 Spray tower scrubber.

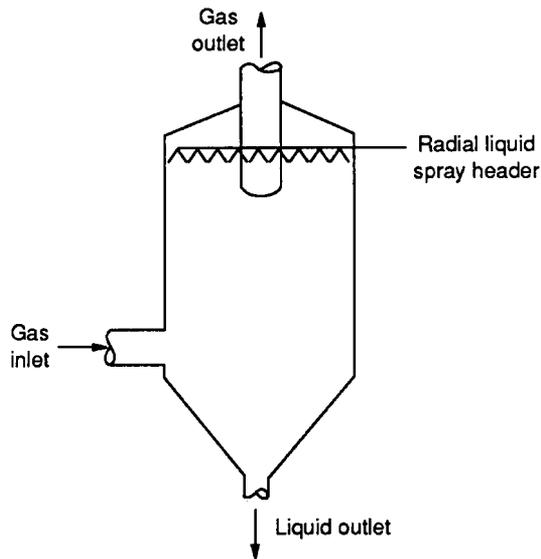


FIG. 4.18 Cyclonic spray tower scrubber.

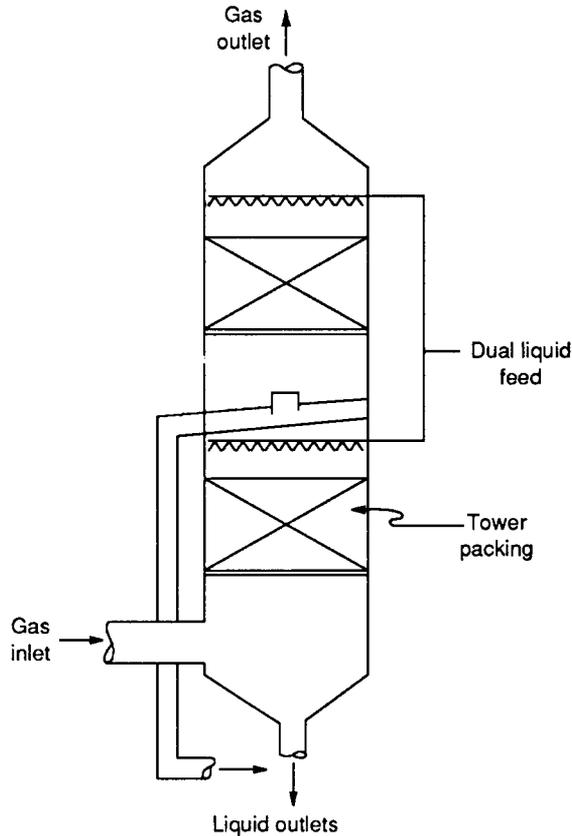


FIG. 4.19 Two-stage packed bed scrubber.

baffles over the perforation to force the rising gas to turn into the liquid level on the plates. These devices are useful for collection of particles over $1\ \mu\text{m}$ in diameter.

Venturi Scrubbers. One of the most common particulate-removal devices is the venturi scrubber, because of its simplicity and relatively high collection efficiency of particles in the 0.5- to $5\text{-}\mu\text{m}$ range. The gas stream is passed through a venturi section to which, or before which, a low-pressure liquid (generally water) is added at the throat. The system is unique in that it not only collects fine particles but also absorbs some gas-phase emissions.

The liquid may be injected in the throat (wetted approach) of the venturi scrubber when the gas stream is close to saturation. In many instances, especially for hot gases, the liquid is injected in the converging section (nonwetted approach) of the scrubber. In both cases, the liquid is atomized by the turbulence in the throat and begins to collect particles impacting the liquid as a result of differing velocities for the gas stream and atomized droplets. Downstream in the diverging section, the mixture decelerates, and further impacts occur, causing the droplets to agglomerate. Often, a centrifugal fan is installed upstream (forced draft) or downstream (induced draft) from the scrubber; the fan provides a motive force to the gas stream.

The liquid droplets are normally atomized by impacting the liquid spray with a high-velocity gas stream.

The alternate approach is to feed the liquid through small, high-pressure orifices in the spray nozzles. This system has the disadvantage of requiring a clean liquid feed stream for protection against clogging.

Once the particles have been captured by the liquid, a separator is used to remove the particle or liquid from the gas stream. Figure 4.20 illustrates the application of a venturi scrubber preceding a simple cyclone separator.

The design of a venturi scrubber may be based on previous experience with an analogous application, field tests of a scrubber on the source, or collection of sufficient data to characterize the gas stream. The latter data include particle size distribution, flow rate, and temperature needed to utilize performance curves for a given unit. Table 4.12 presents the typical range of design data for a venturi scrubber application.

The most important design consideration is the pressure drop across the venturi (Figure 4.21), which can be derived by equations (17) or use of performance curves. A simple approach for calculation of pressure drop is (18):

$$\Delta P = V_t^2 L \times 10^{-6}$$

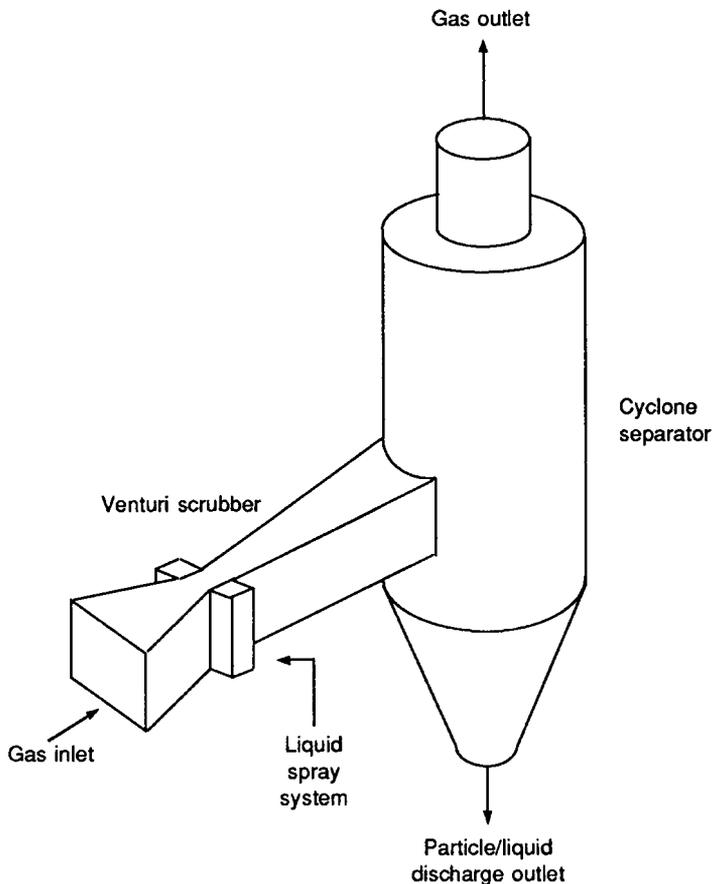


FIG. 4.20 Venturi scrubber system.

TABLE 4.12 Venturi Scrubber Design Data (*I, 8*)

Parameter	Value
Pressure drop	1–20 kPa (5–80 in H ₂ O)
Gas velocity at throat	75–100 m/s (15,000–20,000 ft/mm)
Liquid application rate	0.4–1 L/m ³ (3–7.5 gal/1000 ft ³)
Recirculation	Possible

where ΔP = pressure drop across venturi, cm H₂O

V_t = throat gas velocity, cm/s

L = water to gas volume ratio, L/m³

In U.S. customary units, the equation becomes

$$\Delta P = 5V_t^2L \times 10^{-5}$$

where ΔP , V_t , and L are expressed in terms of inch of water, ft/s, and gallons per 1000 ft³, respectively.

A typical venturi scrubber performance curve includes logarithmic relationships of pressure drop, collection efficiency, and particle size (*17–21*). Figure 4.22 is representative of plots likely to be supplied by a manufacturer; the relationships vary from unit to unit. By knowing the particle size and collection efficiency

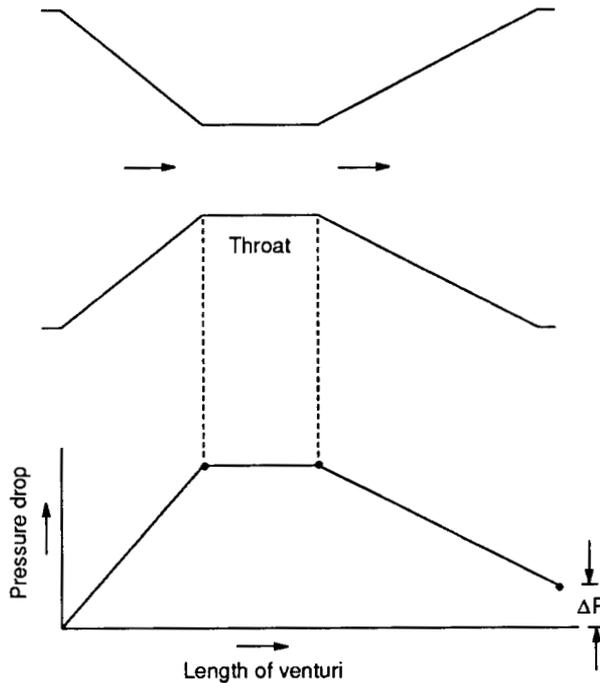


FIG. 4.21 Venturi scrubber pressure profile.

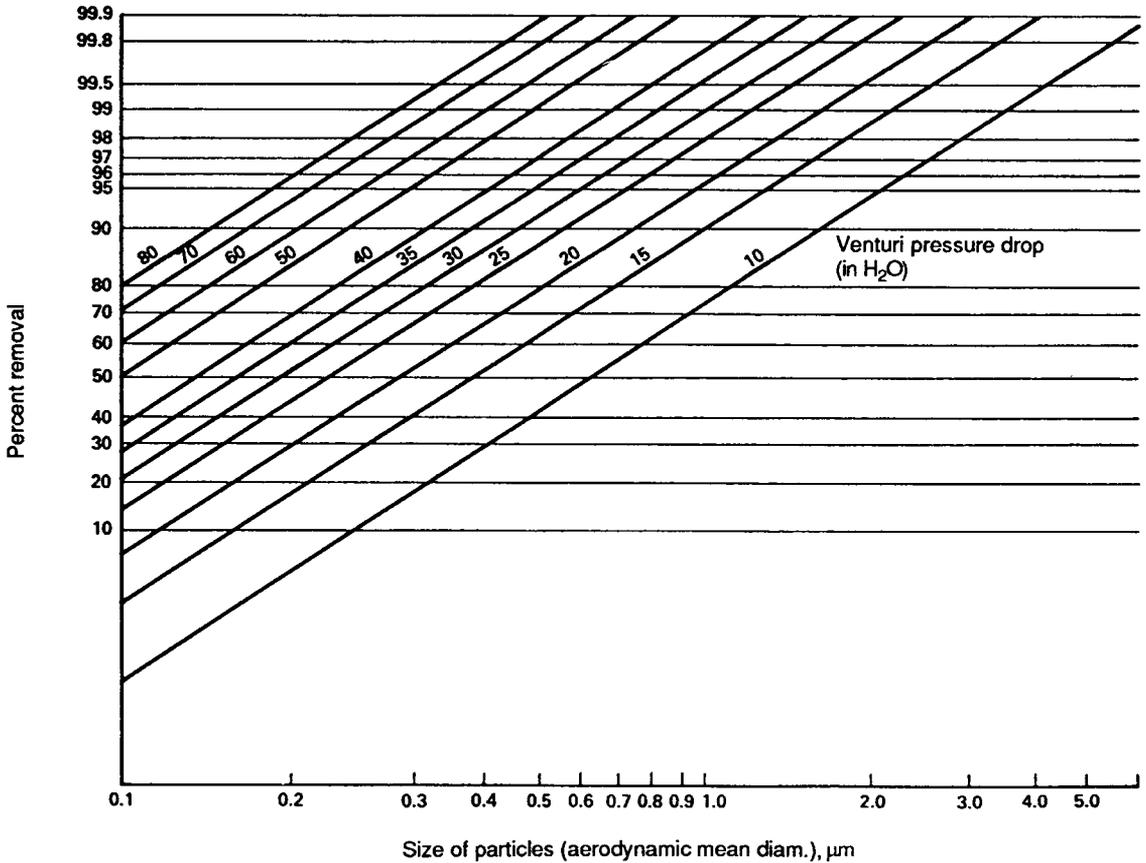


FIG. 4.22 Venturi scrubber performance curves (19).

required, the pressure drop is estimated by the performance curve. If the pressure drop is too high, the designer needs to develop an alternative to venturi scrubber treatment.

The range of pressure drops noted in Table 4.11 is indicative of the variety of applications for venturi scrubbers. Low-range pressure drops apply to bark boilers, solid waste incinerators, and mineral crushers. Midrange sources include coal boilers, kilns, and dryers. Liquid waste incinerators and metal smelters normally exhibit the higher pressure drops (22).

The design also considers the corrosive and erosive nature of the gas stream during the selection of construction materials for the venturi scrubber. Manufacturers should be consulted during this process. Generally, the scrubber will be constructed of carbon or stainless steel. Ceramic or other liners may also be used.

Venturi scrubbers can be sized using either gas flow rate at inlet conditions, $Q_{e,a}$, or the saturation flow rate, $Q_{e,s}$ (23). The calculation involves

$$Q_{e,s} = Q_{e,a} \frac{(T_{e,s} + 273)}{(T_e + 273)}$$

in SI units, or

$$Q_{e,s} = Q_{e,a} \frac{(T_{e,s} + 460)}{(T_e + 460)}$$

in U.S. customary units

where $Q_{e,s}$ = saturated gas stream flow rate, am^3/s (aft^3/min)

$Q_{e,a}$ = inlet gas stream flow rate, am^3/s (aft^3/min)

$T_{e,s}$ = temperature of the saturated gas stream, $^{\circ}\text{C}$ ($^{\circ}\text{F}$)

T_e = temperature of the gas stream, $^{\circ}\text{C}$ ($^{\circ}\text{F}$)

The saturated gas temperature is determined using a psychrometric chart, Figure 4.23. The measured moisture content M_e of the gas stream in percent by volume is converted to units of kg (lb) $\text{H}_2\text{O}/\text{kg}$ (lb) dry air by multiplying the percent value by 0.0062 to obtain the humidity value for the psychrometric chart.

Orifice Scrubbers. An orifice scrubber is also known as an entrainment or self-induced spray scrubber. In this device, the gas stream passes over a pool of liquid at a high velocity before entering an orifice. The high velocity results in centrifugal forces, impingement, and turbulence, which cause wetting of the particles for their separation and collection from the gas stream. Orifice scrubbers have low penetration of particles around $2 \mu\text{m}$ in diameter and larger.

Mechanical Scrubbers. Those devices in which the liquid spray is generated by a rotating element, such as a drum, disk, or fan, are referred to as mechanical scrubbers. This approach produces very finely divided droplets, which results in effective capture of fine particles at the expense of higher energy costs.

Fabric Filters

One of the most efficient devices for removal of particulates is the fabric filter collector. Fabric filters have the capability of maintaining collection efficiencies above 99% for particle size down to $0.3 \mu\text{m}$ (24–28).

The basic features of a fabric filter unit consist of woven or felted fabric, usually in the form of tubes (bags) that are suspended in a housing structure (baghouse). The emission stream is distributed by means of specially designed entry and exit plenum chambers, providing equal gas flow through the filtration medium.

The particle collection mechanisms for fabric filters include inertial impaction, Brownian diffusion, interception, gravity settling, and electrostatic attraction. The particles are collected in dry form either on a dust cake supported by the fabric (most efficient) or on the fabric itself (8, 27, 28). The process occurs with a relatively low pressure drop requirement. Periodically, most of the dust cake is removed for disposal; the residual dust serves as an initial filtration aid while the new dust cake develops.

Fabric filter systems are presented with respect to the three primary cleaning methods. Discussions follow on considerations of important design features of the fabric filter itself (1, 8, 27, 28).

Fabric Filter Systems. The particle collection mechanisms used in various fabric filter collectors are relatively the same. However, the equipment geometry, mode of cleaning, and supplier modifications are quite diverse. This diversity is due to the many applications of fabric filtration with specific performance requirements and physical characteristics.

The most common way to classify fabric filters is by the fabric cleaning method. The three major methods are mechanical shaking, reverse air cleaning, and pulse jet cleaning (8). Other cleaning methods include manual cleaning, pneumatic shakers, bag collapse, and sonic cleaning (1).

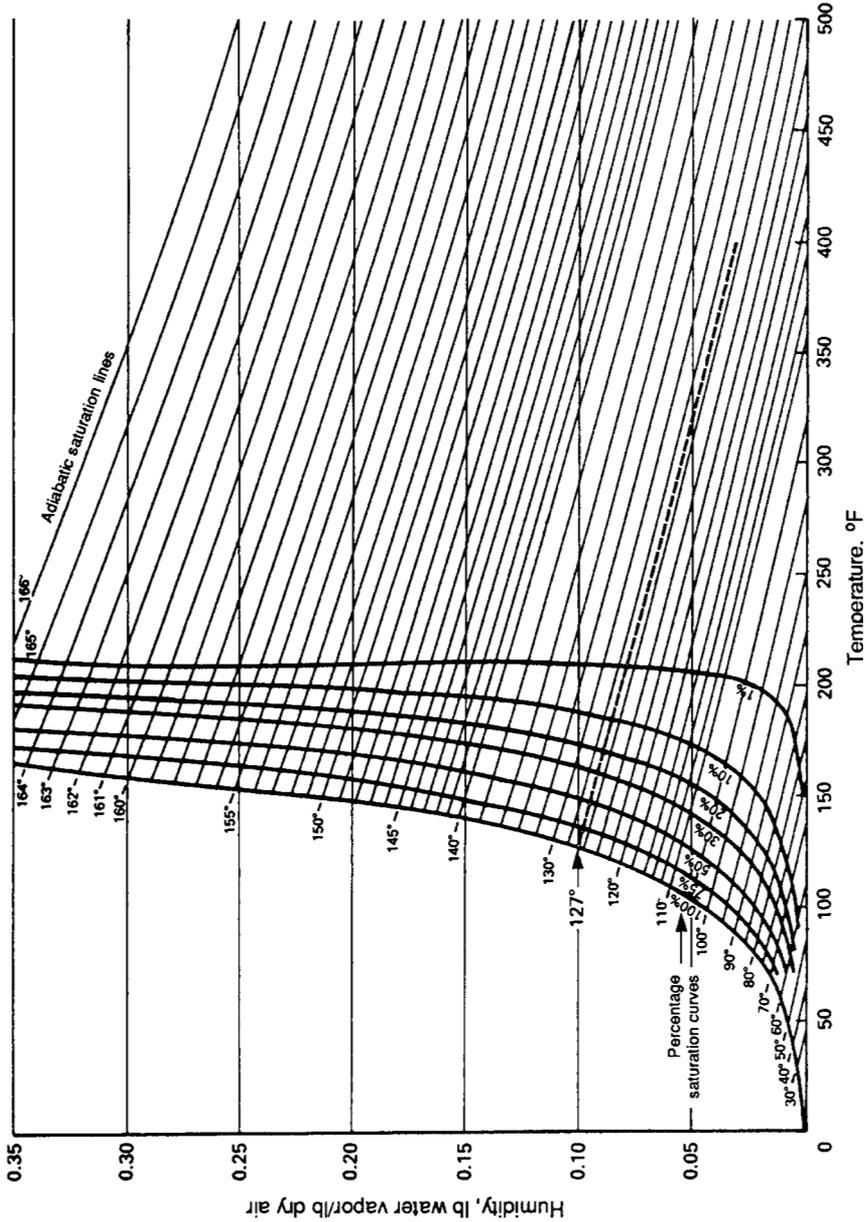


FIG. 4.23 Psychrometric chart (19).

Mechanical Shakers. Many baghouses use some type of mechanical shaking, usually driven by an electric motor. The rotary motion of the motor is translated into an oscillation, and the bags are shaken horizontally (typically) or vertically. The cleaning process progresses through individual compartments containing several bags.

There should be no pressure inside the filter bag during the shaking cycle. Unless the blower is off during the cleaning cycle, it is common to use a small amount of reverse airflow to ensure complete bag collapse during the cycle. When the unit serves a high-temperature source, such as a furnace, the thermal drive may be sufficient to interfere with cleaning even if the blower is off.

As dust accumulates on the fabric filter, the pressure loss through the unit increases until a prescribed maximum pressure is reached and cleaning is necessary to reduce the pressure loss. Manual, semiautomatic, or automatic cleaning cycles are used. Sufficient time is needed to ensure a thorough cleaning once the cycle is initiated. Automatic cleaning cycles are started on a time-cycle basis or are keyed to a maximum allowable pressure drop. Some reverse jet systems operate with continuous cleaning.

The shaking action creates a standing wave in the filter bag and causes flexing of the fabric. This causes the dust cake to crack and release portions of the dust; some dust remains on the fabric and in the fabric itself. The cleaning intensity is controlled by the tension on the filter bag and by the amplitude, frequency, and duration of the shaking process.

Reverse Air. For fabric filters using reverse airflow cleaning, a large volume of low-pressure air is directed countercurrent to the normal gas stream flow during filtration. The reversal of gas flow causes the caked dust to crack and assists in the removal of the cake.

Usually, the cleaning process occurs off-line. This is especially true for inside filter collectors in which cleaning is done by isolated units or compartments. Also, in this case, anticollapse rings may be needed to prevent closure of the collector and dust bridging.

A separate blower may be used to provide the reverse air for cleaning. The air supply may be either cleaned exhaust gas or outside air. The flow rate varies with the characteristics of the dust being filtered.

Pulse Jet. In systems using filtration on the exterior of the fabric collector, pulse jet cleaning may be used. The process involves the application of a high-pressure pulse of compressed air inside of the top of the collector for a short time (a fraction of a second). This action tends to form a wave action down the collector while rapidly expanding the fabric and quickly dislodging all collected dust. Thus, vibration from the wave action and fabric flexing from the pressure increase are the primary cleaning mechanisms.

The cleaning air pulse is normally up to 700 to 825 kPa (100 to 120 lb/in²) for a duration less than 0.1 to 0.2 s. These characteristics normally permit cleaning to be accomplished on-line. A disadvantage occurs in that the high-pressure air pulse typically increases particulate emissions during the on-line cleaning process. Also, since the pulse removes all of the filter cake, filtering actions by any residual dust is lost. Because of this, woven fabrics are favored over felted fabrics.

Fabric Filter Design. Characterization of the emission stream is important in the design of a fabric filter system. Data needs include gas flow rates, temperature ranges, acid dew point, moisture content, and particulate mass loading. The analysis should identify the presence of large particulate matter, sticky particulate matter, and potentially flammable or explosive gases or particulate matter. With this data base, an appropriate collector can be designed for the required level of particulate control.

The selection of the filter fabric and sizing of the collector normally are accomplished in conjunction with the design of the cleaning system. The size of the fabric filter is related to the necessary air-to-cloth ratio and the expected number of units to be out of service for cleaning and/or maintenance at any given time. Additional considerations are operating condition, system pressure drop, safety, compartment construction, instrumentation, solids removal equipment, and need for good maintenance (8).

Fabric. A number of natural and synthetic fabrics are used in fabric filter collectors. Cotton and wool are the most common natural fibers, while the synthetics include dacron, fiberglass, nylon, and others. Often, the fabrics are coated with various substances to prolong service life and/or enhance particle collection. For

example, graphite finishes are used to reduce static charge buildup and to smooth the fabric yarn to reduce abrasion. Another example is the use of a silicone treatment to improve resistance to abrasion.

In many cases, the final fabric selection will be the result of prior experience with a similar application and the associated selection of cleaning method and air-to-cloth ratio. However, important factors to consider include (8):

- Dust penetration
- Temperature—operating and maximum
- Chemical degradation—acid and alkaline
- Abrasion resistance
- Cake release
- Pressure drop
- Cleaning method
- Fabric construction—woven or felted
- Cost—initial and replacement

A comparison of leading fabric characteristics is presented in Table 4.13.

Air-to-Cloth Ratio. Filtration velocity is often referenced as the air-to-cloth (A/C) ratio, which is defined as the emission gas stream flow rate at actual conditions divided by the net fabric (cloth) area active for filtration. The units are expressed as $\text{m}^3/\text{min}/\text{m}^2$ ($\text{ft}^3/\text{min}/\text{ft}^2$) or simply as m/min (ft/min). Table 4.14 provides factors to apply to net cloth area to obtain gross cloth area for the physical sizing and costing of units.

The A/C ratio suggests the amount of air filtered over a period of time without specific regard to the cross-sectional area occupied by the fabric filter cloth. As a result, theoretical or empirical relationships are

TABLE 4.13 Characteristics of Leading Filter Fabrics^a

Fiber type ^b	Maximum continuous operating temperature °F	Resistance ^c				
		Abrasion	Mineral acids	Organic acids	Alkalies	Solvent
Cotton ^d	180	G	p	G	G	E
Wool ^e	200	FIG	F	F	P/F	G
Modacrylic ^e (Dynel)	175	F	VG	VG	G	G
Polypropylene ^e	200	E	E	E	E	G
Nylon polyamide ^e (Nylon 6 & 66)	220		P	F	VG	E
Acrylic ^e (Orion)	260	6	G	G	F	E
Polyester ^e (Dacron, Creslan)	275	E	G	G	G	E
Nylon aromatic ^e (Nomex)	450	E	F	G	VG	E
Fluorocarbon ^e (Teflon, TFE)	500	F/G	E ^g	E ^g	E ^g	E ^g
Fiberglass ^d	550	P,G ^h	VG	E	P	E

^aReferences 1, 8, 19, 24, 29–31. Where data differed, a representative category was chosen.

^bRepresents the major categories of filtration fibers. Names in parentheses indicate some principal trade names.

^cP = poor resistance, F = fair resistance, G = good resistance, VG = very good resistance, and E = excellent resistance.

^dWoven fabrics only.

^eWoven or felted fabrics.

^fConsidered to surpass all other fibers in abrasion resistance.

^gThe most chemically resistant of all these fibers.

^hAfter treatment with a lubricant coating.

TABLE 4.14 Net to Gross Cloth Area Conversion Factors (26)

Net cloth area, A_{nc} , ft ²	Factor to obtain gross cloth area, A_{tc} , ft ²
1–4,000	Multiply by 2
4,001–12,000	Multiply by 1.5
12,001–24,000	Multiply by 1.25
24,001–36,000	Multiply by 1.17
36,001–48,000	Multiply by 1.125
48,001–60,000	Multiply by 1.11
60,001–72,000	Multiply by 1.10
72,001–84,000	Multiply by 1.09
84,001–96,000	Multiply by 1.08
96,001–108,000	Multiply by 1.07
108,001–132,000	Multiply by 1.06
132,001–180,000	Multiply by 1.05
180,001+	Multiply by 1.04

not used to size units. Sizing is based on demonstrated experience of manufacturers for various applications and is recommended for a specific dust or fume and cleaning method. Table 4.15 illustrates A/C ratios for a variety of applications.

The factors which influence the selected A/C ratio include the cleaning method, filter fabric, and particle size and concentration characteristics. Without cost-effectiveness considerations, low A/C ratios will tend to improve particle removal efficiencies by lessening the chance of creating pinhole leaks in the dust cake where aerodynamic effects are important.

Operating Condition. As illustrated in Figure 4.24, for a coal-fired industrial boiler with an air heater, the fabric filter system may operate under negative or positive pressure, depending on the fan placement, which involves consideration of the emission's characteristics. The negative-pressure systems tend to have the most applications (1, 27, 32).

When the emission gases are pushed through the system by a fan (forced-draft fan) located upstream, the system unit is a positive-pressure baghouse. This approach eliminates the need for ductwork and, in many cases, a stack downstream of the fabric filter, which reduces needs for space and other materials but increases difficulties for monitoring. Leaks from the fabric filter enter the surrounding air and, thus, increase the potential for fugitive emissions. Furthermore, the fan is subjected to the full dust (and fume) load and the associated wear from abrasive actions.

A negative-pressure baghouse is one where the fan (induced-draft fan) is located on the downstream, clean side of the fabric filter. The filter housing needs to be sealed to avoid the intake of outside air that would normally cool the emission stream. Cooling may reduce the gas temperature below its dew point and cause damaging condensation inside the unit. In some situations, the presence of outside air could produce a risk of fire and/or explosion. On the other hand, leaks in negative-pressure systems will not cause fugitive dust emissions.

Differential Pressure. The pressure drop across a fabric filter is the sum of the static pressure drop across the cleaning fabric and that developed across the accumulated cake. The latter increases over time from the last to the next cleaning cycle. Many units operate with a differential pressure of 0.5 to 2 kPa (2 to 8 in H₂O); however, some units operate at differential pressures up to 2.5 or 3 kPa (10 to 12 in H₂O).

Safety. When handling flammable or explosive contaminants, special safety provisions are needed to minimize damage in the event of a fire or explosion. Some basic precautions include:

TABLE 4.15 Air-to-Cloth (A/C) Ratios for Various Emissions (25, 28)

Dust or fume	A/C ratios recommended for cleaning method, ft/mm		
	Shaker	Reverse air	Pulse jet
Abrasives	2.0–3.0	*	9
Alumina	2.25–3.0	*	*
Aluminum	3.0	*	16
Aluminum oxide	2.0	*	*
Asbestos	2.5–4.0	*	9–16
Bauxite	2.25–3.2	*	8–10
Blast cleaning	30–35	*	*
Carbon	1.2–2.5	*	57
Carbon black	1.5–2.5	1.1–1.5	8–12
Chrome	1.5–2.5	*	9–12
Coal	2.0–3.0	*	12–16
Coke	2.5	*	9–12
Dyes	2.0	*	10
Fertilizer	2.0–3.5	1.8–2.0	8–10
Flint	2.5	*	*
Fly ash	2.0	2.1–2.3	9–10
Foundry	*	*	8–12
Glass	2.5	*	*
Graphite	1.5–3.0	1.5–2.0	7–9
Gypsum	2.0–3.5	1.8–2.0	10–16
Iron ore	2.0–3.5	*	11–12
Iron oxide	2.0–3.0	1.5–2.0	8–16
Iron sulfate	2.0–2.5	1.5–2.0	6–8
Lead oxide	2.0–2.5	1.5–1.8	6–9
Leather	3.5–4.0	*	15–20
Lime	2.0–3.0	1.5–2.0	10–16
Limestone	2.0–3.3	*	8–12
Machining	3.0	*	16
Manganese	2.25	*	*
Metal fumes	1.5	1.5–1.8	6–9
Metal powders	2.0	*	9–10
Mica	2.25–3.3	1.8–2.0	9–11
Paint pigments	2.0	*	*
Paper	3.5–4.0	*	10–12
Perchlorates	*	*	10
Plastics	2.0–3.0	*	7–10
Polyethylene	*	*	10
PVC	*	*	7
Resin	2.0	*	8–10
Silica	2.25–2.8	1.2–1.5	7–12
Silica flour	2.0–2.5	*	*
Silicates	*	*	9–10
Silicon carbide	*	*	10
Slate	2.5–4.0	*	12–14
Starch	2.25	*	*
Talc	2.25	*	*

*No information available.

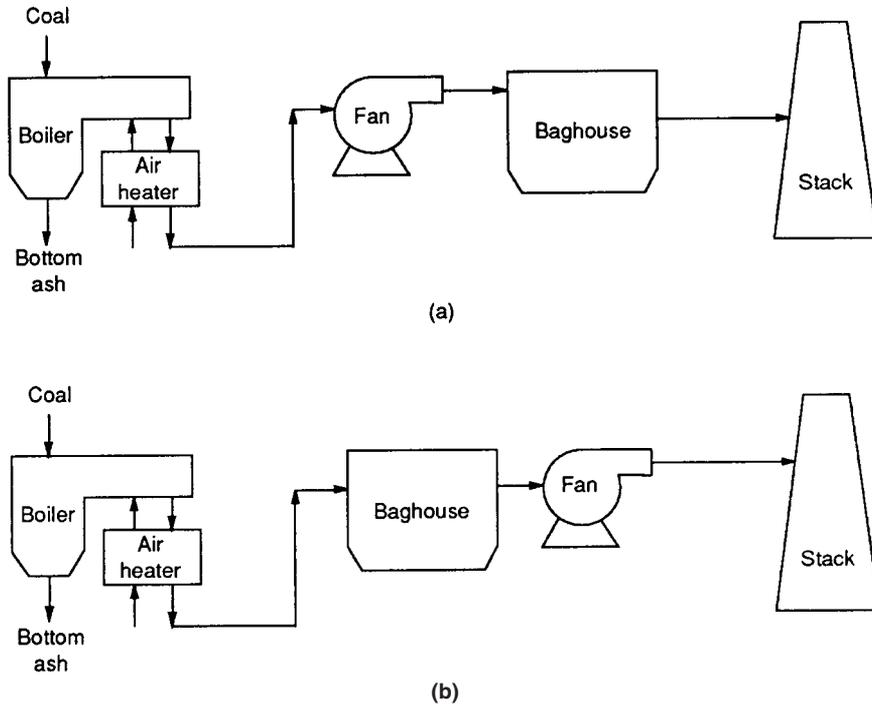


FIG. 4.24 (a) Positive- and (b) negative-pressure baghouse systems.

- The fabric filter should be located out-of-doors. If this is not possible, it should be located near an exterior wall as remote as possible from personnel.
- An explosion relief hatch should be provided on the unit and sized with respect to volume and maximum pressures from an event.
- The construction of the unit shell should be adequately reinforced to withstand the initial impacts of a fire or explosion until secondary systems respond.
- Sprinklers or other means should be provided inside the unit. The system should be interlocked with process operations so as to stop inlet flow and outlet exhaust in the event of a fire.
- In the ductwork, fire and smoke dampers should be installed to protect personnel and, to the extent practical, equipment and structures.
- Fabric filters should be of the static-dissipative type to minimize the possibility of ignition from a static discharge when particles are released from the filter. Also, the fabric should be fire-resistant.

Electrostatic Precipitators

Electrostatic precipitators use electrical energy to charge and collect particles. They have high removal efficiencies; in addition, they are effective for a variety of source categories and emission gas characteristics. These features are outlined in Table 4.16, which lists advantages and disadvantages of electrostatic precipitation. Discussions follow on the basic system approaches, system components, and design considerations.

TABLE 4.16 Advantages and Disadvantages of Electrostatic Precipitation*Advantages:*

1. Highly effective collection with efficiencies exceeding 99% in some cases.
2. Low power requirements and associated low operating power costs.
3. Capable of removing very small particles, even those not removable by other treatment schemes.
4. Dry dust collection often used, which may be useful for product and by-product recovery.
5. Pressure drops are small.
6. Temperature changes are small with dry systems; most impacted in spray-wet systems.
7. Low maintenance requirements due to few or no moving parts.
8. System tolerant to high temperatures, with no special provisions below 535°C (1000°F).
9. Collection efficiency and system capacity can be easily expanded with larger or additional units.
10. Some pollutants, such as acid and tar mists, are effectively collected by other schemes.

Disadvantages:

1. Capital investment is high.
2. Space requirements are relatively large.
3. Very low or high resistivity particles are difficult to remove.
4. Relatively constant operating conditions are needed for efficient collection.
5. Gaseous wastes are not affected.
6. Safety is a concern due to high voltage.
7. Wet systems produce sludges that may require dewatering before disposal.

Electrostatic Precipitation Systems. The classification of electrostatic precipitators may be as dry or wet systems and/or as single- or two-stage systems (1, 8, 33).

Dry Systems. The dry precipitator with plate-type collection electrodes is the predominant type of system in industrial applications and is the process most discussed in this section.

Wet Systems. The primary difference in wet and dry systems is the method by which collected particles are cleaned from the collection electrode. Three common wet precipitator configurations are the plate type with horizontal gas flow, the concentric plate type with vertical gas flow, and the conventional pipe type with reversing vertical flow.

Single-Stage Systems. Most applications of electrostatic precipitators use the single-stage mode of operation.

Two-Stage Systems. The two-stage precipitator unit was developed to clean air in conjunction with institutional, commercial, and industrial air conditioning systems. For emission control, these units usually are restricted to applications requiring collection of liquid particles that drain quickly from the collection electrode.

Component Characteristics. The principal features of an electrostatic precipitator are the housing, discharge and collection electrodes, power source, cleaning mechanism, and solids-handling system. Other features include the gas distribution system and instrumentation. Figure 4.25 illustrates most of these features for a dry precipitator (1, 8, 19, 33, 34).

Housing Materials. Carbon steel is the most common material for the electrostatic precipitator housing. In cases where the gas stream contains high concentrations of sulfur oxides or where there is liquid-gas contact, stainless steel may be needed. An alternate to the latter is to maintain the gas stream temperature above the dew point (1, 26, 33, 34).

The nature of the housing is that it is gas-tight and weatherproof and that it is grounded to ensure personnel safety. The principal features include the housing proper with its shell and insulated housing as well as the inlet and outlet connections, the dust hopper, and inspection ports.

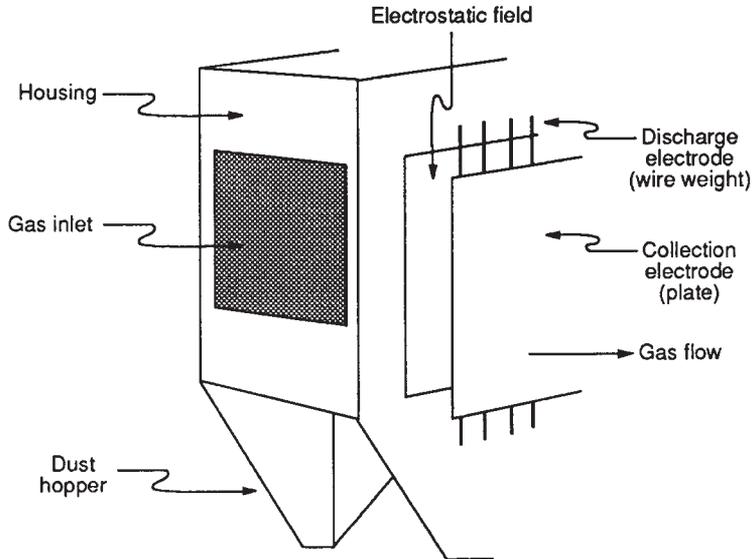


FIG. 4.25 Dry electrostatic precipitator schematic.

Discharge Electrode. Design of the discharge electrode is influenced by the composition of the emission stream. Physically, the electrode may take a variety of shapes, such as cylindrical [0.16 to 0.32 cm ($1/16$ to $1/8$ in) diameter] or square [0.48 cm ($3/16$ in) typical] wire or stamped or formed metal strips. The shape influences the current and voltage characteristics. For example, the smaller the wire or the more pointed its surface, the greater should be the current for a specific voltage.

The mounting of the discharge electrode is accomplished in one of two ways. The wire-weight approach uses weights at the bottom to hold in place a suspended electrode. The rigid-frame system has the electrode rigidly attached to masts or frames. Figure 4.26 illustrates these types of electrodes.

The dust particles are charged by ions from the discharge electrode, and the electric field strength governs the amount of charge on the particles. Field strength is based on system voltage and the distance between the discharge and collection electrode.

Generally, electrostatic precipitators are operated with as high a secondary voltage as possible, which will limit sparking. Sparking reduces collection efficiencies as a result of an instantaneous voltage drop and suspension of the electrostatic collection field. In some cases, automatic voltage controls are used to control sparking within a specified range.

Collection Electrode. Dust is collected on the grounded electrodes of the electrostatic precipitator. These electrodes, also called plates, come in a variety of shapes for flat units, and, in some cases, the precipitator is equipped with cylindrical collection surfaces (8, 33).

Generally, the collection electrodes are available in lengths of 1 to 4 m (3 to 12 ft), wire-weight, or 2 m (6 ft) to greater than 5 m (15 ft), rigid-frame. Also, they come in heights of 3 to 12 m (9 to 36 ft) for wire-weight and as high as 15 m (50 ft) for rigid-frame designs. The electrodes are grouped to form independently rapped collecting modules for assembly in straight and parallel alignment with the discharge electrodes.

The electrode support system design must consider cleaning methods; in particular, rappers for dry systems will impact their energy to the electrode in the form of vibration and impact loadings.

Transformer-Rectifier. Electrical power supplied to the electrostatic precipitator comes from a high-

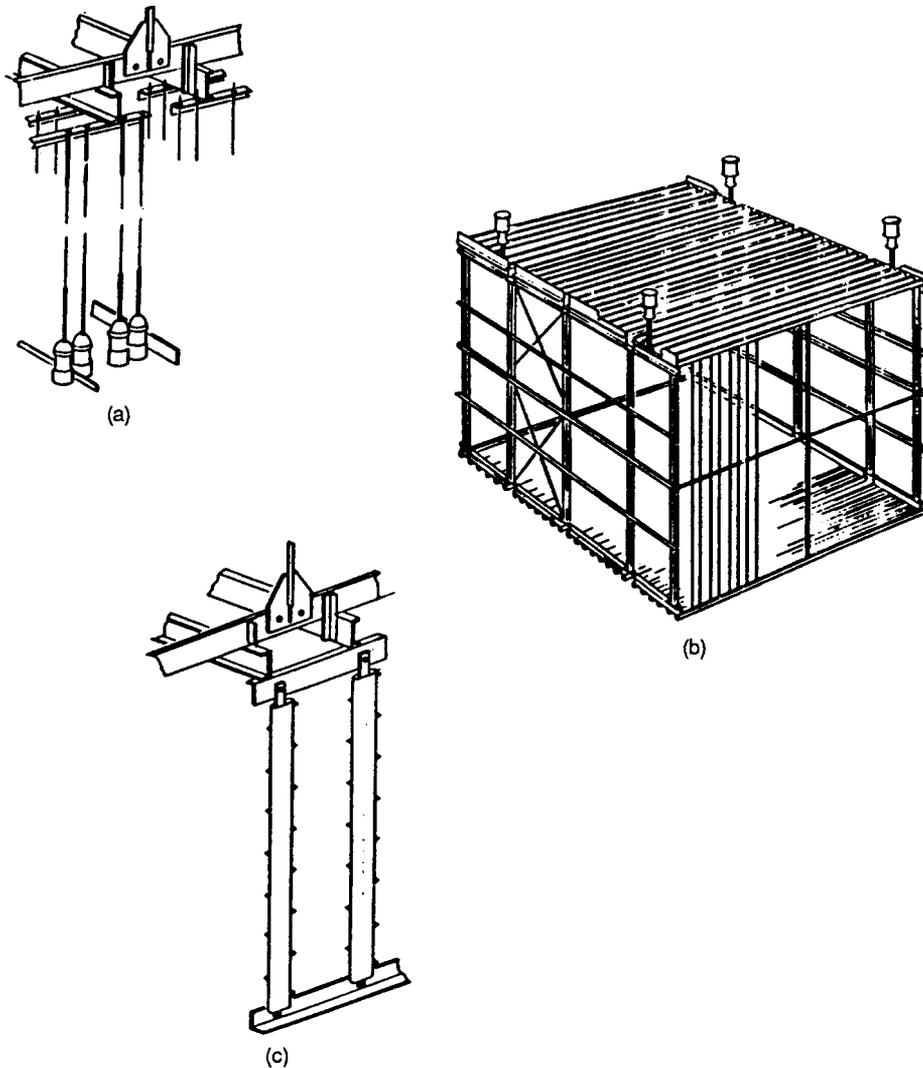


FIG. 4.26 Discharge electrodes. (a) Weighted wire; (b) rigid frame; (c) rigid discharge electrode (RDE) (33).

voltage transformer–rectifier, which converts low-voltage ac power to high-voltage dc (normally in the negative form). Generally, multiple units are used.

Voltage and current values are related to such factors as electrode geometry and gas and particle characteristics. Typically, input voltage is 460 V, three-phase, 60 Hz (ac) with an output voltage between 45 and 70 kV (dc). The maximum rated current output is usually in the range of 250 to 1500 mA (8).

Cleaning Mechanism. Rappers and liquid washdown are the cleaning mechanisms for dry and wet electrostatic precipitators, respectively. In both cases, the frequency and intensity of cleaning requires case-by-case design and operating flexibility.

Wire-weight and rigid-frame designs are used to categorize rappers for dry electrostatic precipitator applications. Cleaning and rapping are very important in the overall performance of the system. Improperly installed or operating rappers may result in particle reentrainment and/or sparking as a result of particle buildup on the collection electrode.

For wire-weight systems, the rapping impulses are provided by either single-impulse or vibratory rappers, which are either electrically or pneumatically activated. In some cases, the rapper is used to clean a system's discharge electrodes. Recognizing the many variables of the rappers' physical characteristics and of individual emission (particle) streams, Table 4.17 illustrates typical ranges of rapper system characteristics.

Mechanical hammer rappers are used for rigid-frame installations. Each frame is served by a hammer driven by a low-speed motor. The rapping intensity is determined by the hammer weight, and the frequency is controlled by the shaft rotation speed.

For wet precipitators, cleaning is performed by the flow of a thin film of liquid over the collection electrode. In some cases, sprays are used to cool, condition, and/or scrub the gas stream. Since the spray is precipitated with the particles, a secondary wetting of the collection electrode is provided.

Solids Handling. Dust hoppers collect the precipitated particles from a dry electrostatic precipitator. Typically, the hopper is like an inverted pyramid converged to a round or square discharge. For best results, particles are removed frequently, or continuously, from the hopper, which is not normally designed as a storage facility.

Either pressure or vacuum systems are used to remove particles from larger hoppers. In smaller installations, a screw conveyor may be used. Wet sluicing is another approach for removal of the solids. In the former cases, the solids can be disposed of in a dry form. Sluiced solids require a pond or other means to remove solids from the liquid stream.

Design Considerations. Important gas stream and particle properties that affect the effectiveness of electrostatic precipitation of particulate matter include particle size distribution, gas flow rate, resistivity, and temperature. The design must also consider how these characteristics affect the corrosiveness of the particles and the removability of particles from the collection electrodes.

For the electrostatic precipitator itself, manufacturers are a primary source for design data. An important design determination is the area of the collection electrode. Typical design data for other parameters are shown in Table 4.18 (1, 8, 33).

Particle Size. Electrostatic precipitation is most effective with coarser, larger particles. Small particles in the 0.2- to 0.4- μm diameter range are most difficult to remove because the field-charging mechanism is replaced by diffusion charging by thermal ions (random collisions) as a charging mechanism for very small particles. If there is a large percentage of small particles less than 1 μm , they can inhibit the generation of the charging corona in the inlet field, and, as a result, the effectiveness of electrostatic precipitation may be unsatisfactory.

TABLE 4.17 Wire-Weight Rapper Characteristics (33)

Parameter	Typical value
Collection area per rapper unit	110–150 m ² (1200–1600 ft ²)
Discharge electrode length served per rapper unit	300–2150 m (1000–7000 ft)
Rapping intensity	3–35 rn/kg (5–50 ft/lb)
Rapping interval range	0.5–10 min

TABLE 4.18 Electrostatic Precipitator Design Data (I , δ)

Parameter	Typical range
Horizontal length of collection plates	0.5–1.0 times height
Vertical height of collection plates	3.7–7.3 m (12–24 ft)
Spacing of collection plates	20–28 cm (8–11 in)
Treating time	2–10 s
Velocity through precipitator	1–2 m/s (3–6 ft/s)
Particle drift velocity (W)	0.03–0.18 m/s (0.1–0.6 ft/s)
Applied voltage	30–75 kV
Corona current	0.03–3.3 mA/m (0.01–1 mA/ft)
Field strength	2.7–5.9 kV/cm (7–15 kV/in)
Treatment time	2–10 s
Gas temperature, standard	up to 370°C (up to 700°F)
Efficiency	90–99.9%

Computer models, supplier experience, owner testing, and/or historic applications should be used to predict the impact of particle size distribution on the system removal efficiency. To the extent possible, size distribution should be determined or projected under the full range of expected operating conditions.

Gas Flow. The design needs to interact the gas volume with the unit to maintain a velocity in the range of 1 to 2 m/s (3 to 6 ft/s).

Low velocity, below 0.6 to 0.9 m/s (2 to 3 ft/s), can result in poor distribution of the gas and even dropout of particles en route to the unit. Flexibility to bypass portions of the electrostatic precipitator can be obtained with dampers and is one means to control variable gas flow rates.

High velocities may cause rapping and reentrainment of particles because of the aerodynamic forces on the particles. The critical velocity will vary from project to project due to the influences of gas flow, plate configuration, unit size, resistivity, and other factors and properties.

Resistivity. Resistivity considerations apply only to dry electrostatic precipitators. Resistivity is a measure of a particle's ease in conducting electricity, and higher values signify less ability of the particle to transfer the charge.

The preferred range of resistivity is usually taken as 10^8 to 10^{10} Ω -cm; Table 4.19 depicts electrostatic precipitator characteristics associated with several levels of resistivity. Factors influencing resistivity include chemical composition, moisture content, and temperature of the gas stream and/or particulates to be removed.

Particles with low resistivity are difficult to collect since they tend to lose their charge and drop off the collection electrode to become reentrained in the gas stream. Special design features for collection electrodes or coatings are needed to reduce reentrainment.

On the other hand, particles with high resistivity create problems by blinding the collection electrode with an accumulation of particles. Conditioning agents for the particles, oversized units, and/or frequent cleaning are used to improve the performance of systems collecting high-resistivity particles.

Temperature. Temperature effects on resistivity can be significant, and the designer should be aware of the relationship for individual process applications. Gas temperatures up to 370°C (700°F) are considered normal and at 535°C (1000°F) are high. Special conditions are needed for very high temperatures, such as those around 700°C (1300°F).

Collection Area. The area of the collection plate (electrode) is an important design consideration and is a function of the gas stream flow rate, the particle drift velocity, and the design system removal efficiency. The Deutsch–Anderson relationship for these variables is often used and may be expressed as

TABLE 4.19 Characteristics Associated with Different Resistivity Levels

Resistivity level, Ω -cm	ESP characteristics
Less than 10^8	<ol style="list-style-type: none"> 1. Normal operating voltage and current levels unless dust layer is thick enough to reduce plate clearances and cause higher current levels 2. Reduced electrical force component retaining collected dust, vulnerable to high reentrainment losses 3. Negligible voltage drop across dust layer 4. Reduced collection performance due to 2
10^8 – 10^{10}	<ol style="list-style-type: none"> 1. Normal operating voltage and current levels 2. Negligible voltage drop across dust layer 3. Sufficient electrical force component retaining collected dust 4. High collection performance due to 1, 2, and 3
10^{11}	<ol style="list-style-type: none"> 1. Reduced operating voltage and current levels with high spark rates 2. Significant voltage loss across dust layer 3. Moderate electrical force component retaining collected dust 4. Reduced collection performance due to 1 and 2
Greater than 10^{12}	<ol style="list-style-type: none"> 1. Reduced operating voltage levels; high operating current levels if power supply controller is not operating properly 2. Very significant voltage loss across dust layer 3. High electrical force component retaining collected dust 4. Seriously reduced collection performance due to 1, 2, and probably back corona

Typical values. Operating voltage: 30–70 kV, dependent on design factors; operating current density: 5–50 nA/cm² Dust layer thickness: ¼ to 1 in.

$$A = \frac{-Q}{tW} \ln(1 - E)$$

where A = collection plate area, m² (ft²)

Q = gas stream flow rate at actual conditions, m³/h (ft³/min)

t = time conversion factor, 3600 (60)

W = particle drift velocity, m/s (ft/s)

E = desired collection efficiency expressed as a decimal

Typical values for drift velocity are presented in Table 4.20; manufacturers and further literature review are additional sources. Otherwise, a value of 0.09 m/s (0.3 ft/s) may be used for particles with a resistivity in the range of 10^7 to 10^{10} Ω -cm or 0.03 m/s (0.1 ft/s) for particles with a higher resistivity in the range of 10^{11} to 10^{13} Ω -cm (I).

GAS CONTROLS

Gaseous pollutants originate from many industrial processes and may be either organic (i.e., volatile organic compounds), or inorganic (i.e., nitrogen oxides or sulfur oxides) in composition. Following characterization of the emission stream, the first priority in pollution control is to investigate source and process reduction measures. Conventional gas emission treatment techniques include absorption, adsorption, condensation, flaring, and incineration. Also, particulate control systems (especially wet scrubbers) may substantially reduce gaseous pollutants as a secondary process.

TABLE 4.20 Typical Values for Drift Velocity for Various Applications (1)

Application	Drift velocity, ft/s
Pulverized coal	0.33–0.44
Paper mills	0.25
Open-hearth furnace	0.19
Secondary blast furnace (80% foundry iron)	0.41
Gypsum	0.52–0.64
Hot phosphorus	0.09
Acid mist (H ₂ SO ₄)	0.19–0.25
Acid mist (TiO ₂)	0.19–0.25
Flash roaster	0.25
Multiple-hearth roaster	0.26
Portland cement (wet manufacturing)	0.33–0.37
Portland cement (dry manufacturing)	0.19–0.23
Catalyst dust	0.25
Gray-iron cupola (iron–coke ratio = 10)	0.10–0.12

Note: m/s = 0.305 ft/s

Source Controls

In this context, source controls include not only consideration of raw products but also effectiveness of process operations. The source controls previously discussed for particulates have secondary impacts on gaseous contaminants; this is also true for some of the following impact particulate emissions. Often, the source control techniques, such as fuel switching, reduce both types of emissions (8, 36, 37).

Fuel Substitution. Historically, fuel substitution is used to reduce sulfur oxide emissions and, in more recent times, to reduce nitrogen oxides. Alternate fuel sources and alternate fuels, such as natural gas and fuel oil, should be considered. In the future, more use of synthetic fuels and, perhaps, nuclear power will be more viable alternatives. Trends in other modes of power generation and energy use may provide cost-effective, nonpolluting use of hydropower, solar energy, geothermal energy, and wind power.

Fuel Cleaning. Desulfurization and denitrification of fuels are emerging as source controls. The greatest potential for fuel cleaning has been demonstrated for coal. The current findings show minimal benefits from cleaning other fuels such as natural gas and fuel oil (36, 37).

Desulfurization. The most effective desulfurization process is coal cleaning. Sulfur occurs in coal primarily as organic sulfur, which is chemically bound to the coal, and also as inorganic sulfur, which is found in discrete particles of pyrite in the coal.

Physical coal cleaning removes ash-forming impurities along with pyrite. Generally, the process involves crushing the coal to a size whereby mineral and coal particles may be separated using differences in density or surface properties. Most separation techniques use density as the design parameter; the actual separation process may be a hydraulic jig, hydroclone, froth flotation system, concentrating table, dense-medium vessel, or classifier (38–41).

Chemical coal cleaning is being developed to complement the desulfurization gains by physical cleaning methods. In contrast to physical cleaning, the developing technology and associated costs will likely limit the use and/or effectiveness of chemical coal cleaning (36, 40, 41).

Denitrification. Seemingly, fuel denitrification of coal and heavy oils could be used to control nitrogen oxide emissions, but current technology is limited. Some marginal secondary benefits have been observed as a result of desulfurization processes (39).

Process Modifications. Since most gaseous emissions are the result of combustion processes, the system should be thoroughly evaluated for opportunities to improve efficiency. Increased efficiency reduces fuel costs and the quantity of fuel-derived pollutants. Also, more complete use of fuel leads to less volatile emissions.

Sulfur and, in particular, nitrogen oxide emissions are sensitive to combustion operating conditions and equipment design. For example, modified operating conditions for nitrogen oxide control include low excess air, off-stoichiometric or staged combustion, flue gas recirculation, reduced air preheat, reduced firing rate, and steam or water injection. Equipment modifications include burner design and spacing (36, 37).

Fluidized bed combustion is a primary alternative to conventional practice. Table 4.21 provides a comparison of advantages and disadvantages of fluidized bed combustion compared with conventional boilers. Research and development efforts are addressing atmospheric and pressurized systems.

Absorption

Air pollution control by absorption is the process whereby one or more gaseous contaminants are selectively dissolved into a relatively nonvolatile liquid. The liquid contains less than the equilibrium concentration of the gaseous component. Absorption is a major chemical engineering unit operation and is covered in many technical references.

In some cases, the effluent from an absorber can be reprocessed to recover a usable by-product; however, more often it is simply discharged to a wastewater treatment system. Absorption is commonly used to control emissions of sulfur dioxide, hydrogen sulfide, hydrogen chloride, chlorine, ammonia, and nitrogen oxides. Organics may also be absorbed, but their removal is more dependent on liquid solvent selection and subsequent wastewater treatment capabilities.

Absorption is categorized as chemical or physical. Chemical absorption occurs when there is a reaction between the absorbed gas and the liquid solvent. Physical absorption occurs when the absorbed gas merely dissolves in the liquid solvent. Typical liquid solvents include water, mineral oils, and specially prepared aqueous solutions.

Flue gas desulfurization is an important application of absorption processes. Eight specific absorption processes are summarized in Table 4.22 and illustrated by referenced schematic diagrams in Figures 4.27 to 4.33.

Absorber Systems. Gas absorption systems are designed to maximize contact between the gas and liquid solvent in order to permit interphase diffusion of the materials. Absorbers found to adequately disperse the liquid include packed towers, plate or tray towers, spray chambers, and venturi scrubbers.

TABLE 4.21 Advantages and Disadvantages of Fluidized Bed Combustion (36, 37)

Advantages:

1. Compact size yielding low capital cost, modular construction, factory assembly, and low heat transfer area.
2. Higher thermal efficiency including higher heat release and heat transfer.
3. Potentially efficient control of sulfur oxides by direct contact of coal with a SO_x absorber.
4. Lower combustion temperature resulting in less fouling and corrosion and reduced NO_x formations.
5. Fuel versatility including a wide range of low-grade fuels, such as char from synthetic fuels processing.
6. Adaptable to a high-efficiency gas-steam turbine combined power generation cycle.
7. Waste solids are in dry form.

Disadvantages:

1. Large particulate loading in the flue gas.
2. Potentially large amounts of solid waste, SO_x absorbent.
3. Limited applied technology beyond that of the petroleum industry.

TABLE 4.22 Absorption Processes for Sulfur Oxide Control

Process	Description	Illustration
Lime	Nonregenerable process with lime slurry. Calcium sulfite and sulfate formed. Sludge landfillable (16, 36, 42–45).	Figure 4.27
Limestone	Nonregenerable process with limestone slurry. Calcium sulfite and sulfate formed. Sludge landfillable (16, 36, 44–46).	Figure 4.28
Dual alkali	Regenerable process with sodium solution. Calcium sulfite and sulfate formed by lime or limestone regeneration. Sludge landfillable (36, 47, 48).	Figure 4.29
Sodium solution	Nonregenerable process with sodium hydroxide or carbonate. Spent absorbent of sodium sulfate requires treatment (36, 50).	Figure 4.30
Ammonia-based	Nonregenerable process with ammonium hydroxide. Ammonium sulfate salable by-product. Alternate processes available (36, 51, 52).	Figure 4.31
Wellman–Lord	Regenerable process with aqueous sulfite. Liquid SO ₂ , liquid SO ₃ , sulfuric acid, or elemental sulfur recoverable by-products (36, 51, 52).	Figure 4.32
Citrate	Regenerable process with water enhanced by citric acid and sodium hydroxide or carbonate. Liquid SO ₂ or elemental sulfur recoverable by-products (36, 53).	Figure 4.33
Magnesium oxide	Regenerable process with magnesium oxide. SO ₂ , liquid SO ₃ , or elemental sulfur recoverable by-products (36, 52, 54).	

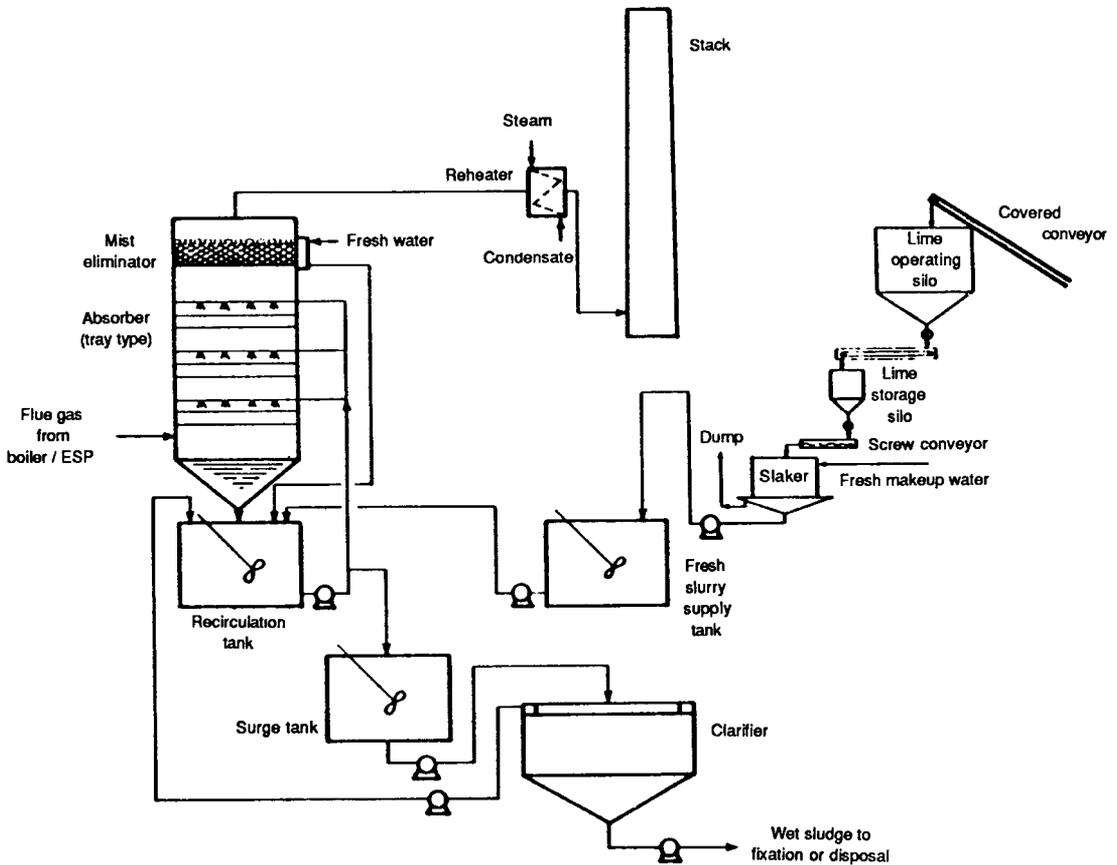


FIG. 4.27 Lime flue gas desulfurization process (36).

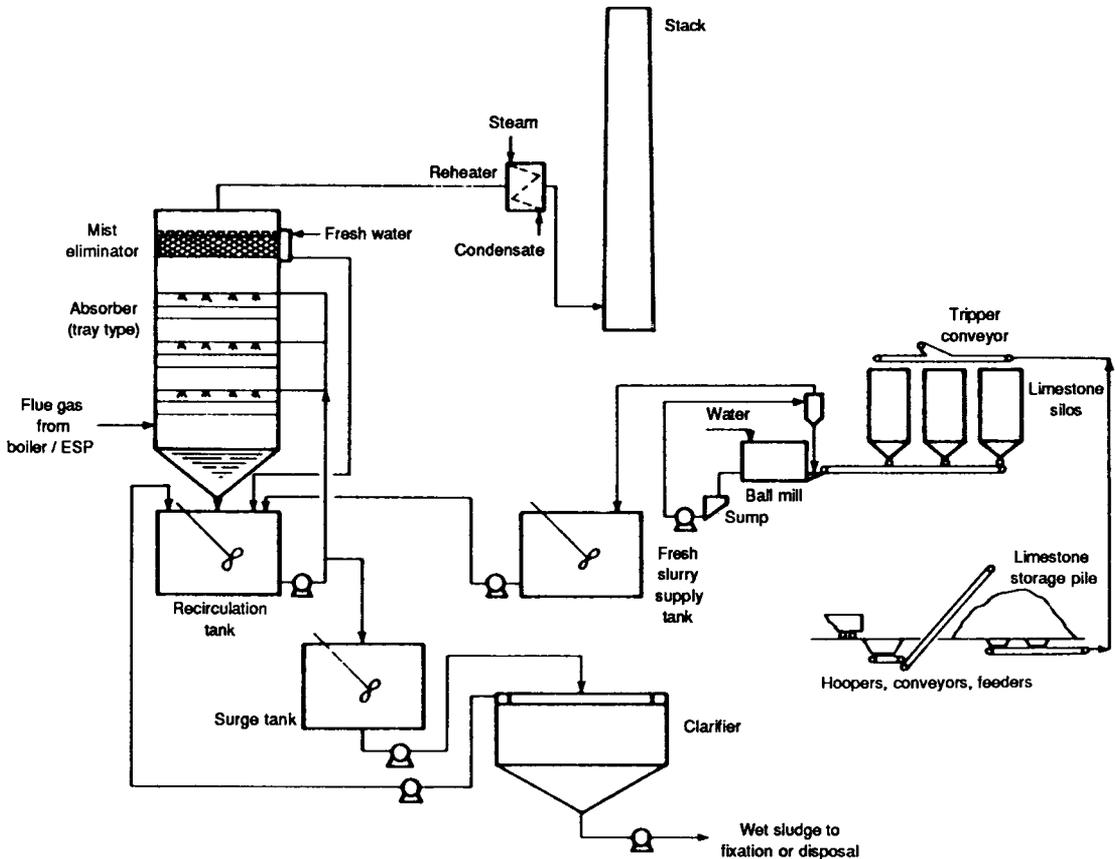


FIG. 4.28 Limestone flue gas desulfurization process (36).

When water is the liquid solvent, absorbers are analogous to wet scrubbers used for reduction of particulate emissions. Thus, this type of equipment can serve to reduce both types of contaminants. However, this basis of design will be more specifically directed to one or the other.

Packed Tower Absorber. A packed tower uses any of a variety of packing materials to expose a large (wetted) surface area to the gas stream. The most common and most efficient flow pattern is countercurrent, with the gas stream entering near the bottom of the tower and the liquid solvent entering near the top. Figure 4.34 (page 4.34) illustrates this system.

In practice, the packing tends to be manufactured shapes from materials that are structurally sound, chemically inert, and relatively inexpensive. Raschig rings are used most often, but other shapes include Berl saddles, Intalox saddles, Lessing rings, cross-partition rings, and spiral-type rings.

Plate Tower Absorber: Using a countercurrent flow pattern, plate towers utilize multiple plates to produce stepwise contact for the absorption process. The number of plates is a function of the mass transfer operation, the degree of removal required, and the desired separation.

The predominant type of plate used is the bubble cap type, which is illustrated in Figure 4.35 (page 4.59).

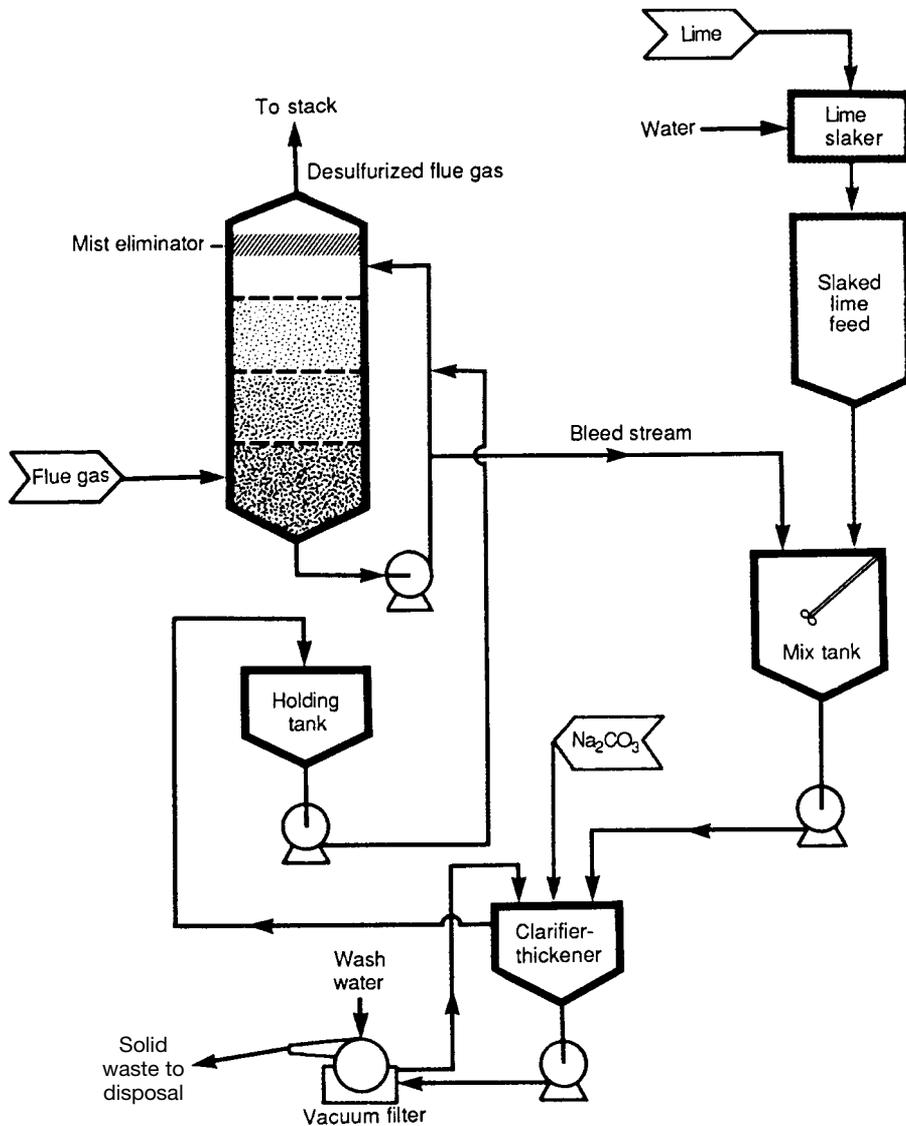


FIG. 4.29 Dual alkali flue gas desulfurization process (65).

As the liquid solvent flows across the plate at a controlled depth, the gas stream flows through the periphery slots in the bubble cap and into the solvent.

Spray Chamber Absorber. The spray-type absorber also uses a countercurrent flow pattern where the gas stream flows upward through a fine spray of the liquid solvent. The surface area for absorption is simply the surface of the spray droplets. Since fine droplets are required for surface contact, the solvent is susceptible to entrainment in the gas stream. Perhaps the best application of spray chamber absorbers is as a pretreatment for other more effective (and expensive) systems.

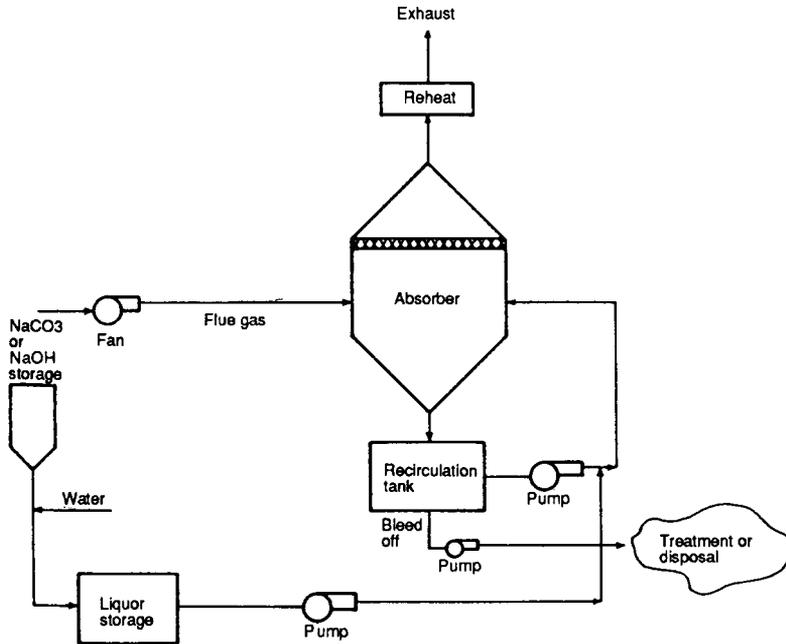


FIG. 4.30 Sodium solution flue gas desulfurization process (36).

Venturi Absorber. Dispersion and mixing of the gas stream and liquid solvent by a venturi absorber is achieved by injecting the solvent into the gas stream as it passes through the venturi or the converse. The relatively short contact time limits the use of venturi absorbers to pretreatment for gaseous contaminant control.

Absorber Design. Removal of gaseous pollutants by absorption involves three operations.

1. The pollutant is diffused through the gas stream to the surface of the liquid solvent.
2. The pollutant is dissolved into the liquid solvent at the gas–solvent interface.
3. The dissolved pollutant is diffused from the interface into the mass of liquid solvent.

The basic relationship of the absorption process is expressed as:

$$N_A = K_G(p - p_i) K_S(C_i - C)$$

where N_A = mass transfer coefficient

K_G = reaction coefficient for gas

p = partial pressure of gas

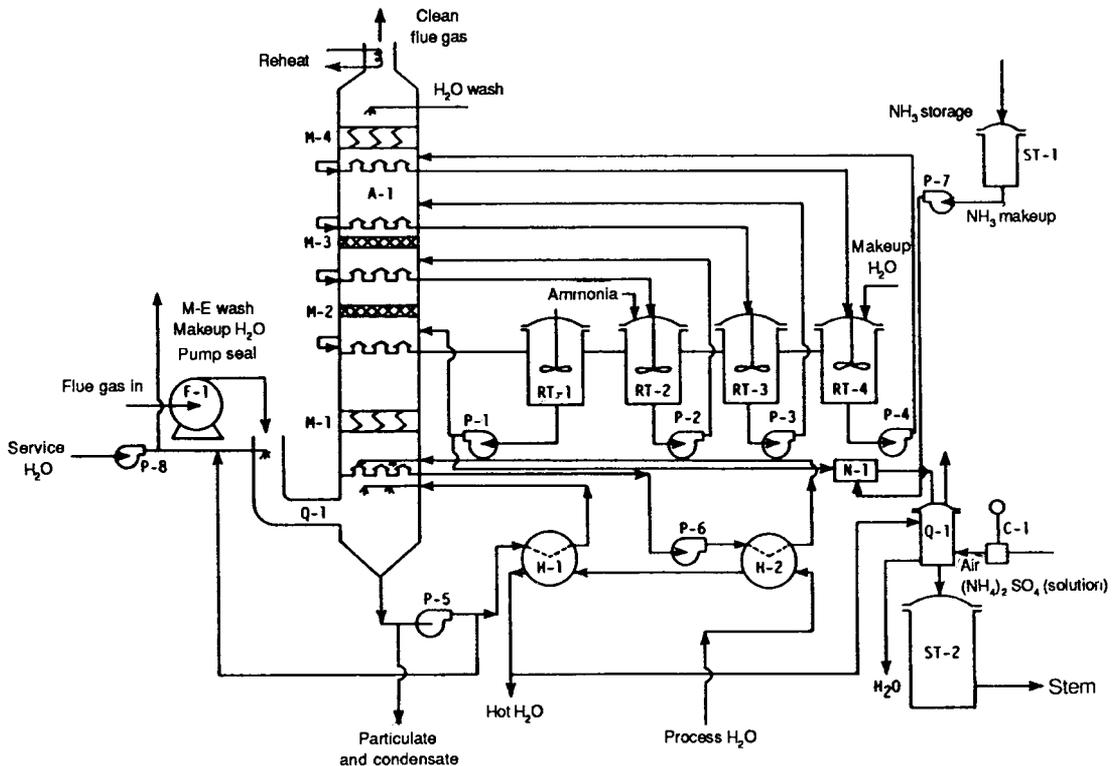
p_i = partial pressure of gas at interface

K_S = reaction coefficient for solvent

C_i = concentration in solvent at interface

C = concentration in solvent

The equilibrium condition occurs at the gas–solvent interface. One approach, the two-film concept, considers that a hypothetical film exists on each side of the interface. Figure 4.36 (page 4.60) illustrates the two-film model.



Legend:

A-1: Absorber	C-1: Compressor	F-1: FD fan	N-1: Neutralizer
Q-L: Quencher	H-1,2: Heat exchangers	ST-1,2: Storage tanks	O-1: Oxidizer
P-1 to 8: Pumps	RT-1 to 4: Recycle tanks	M-1 to 4: Mist eliminators	

FIG. 4.31 Ammonia-based flue gas desulfurization process (36).

Detailed design information is available from suppliers and mass transfer technical references. The following addresses more general design considerations. For example, Table 4.23 (page 4.60) lists design considerations for the more commonly used packed tower and bubble cap plate absorbers.

The absorption rate is normally improved by greater surface contact, higher liquid-to-gas ratios, higher gas concentrations, and lower temperatures. The physical properties of the reaction are affected by common characteristics such as diffusion, viscosity, and density. Respective flow rates and temperatures are important operating conditions.

Adsorption

The phenomenon whereby a gas molecule is retained by either chemical or physical forces is known as *adsorption*. The mechanism of adsorption is complex and is discussed in numerous references and elsewhere in this handbook with respect to water and wastewater treatment (1, 19, 36, 55).

The adsorbing material, or *adsorbent*, is naturally most important, and its primary characteristics are a

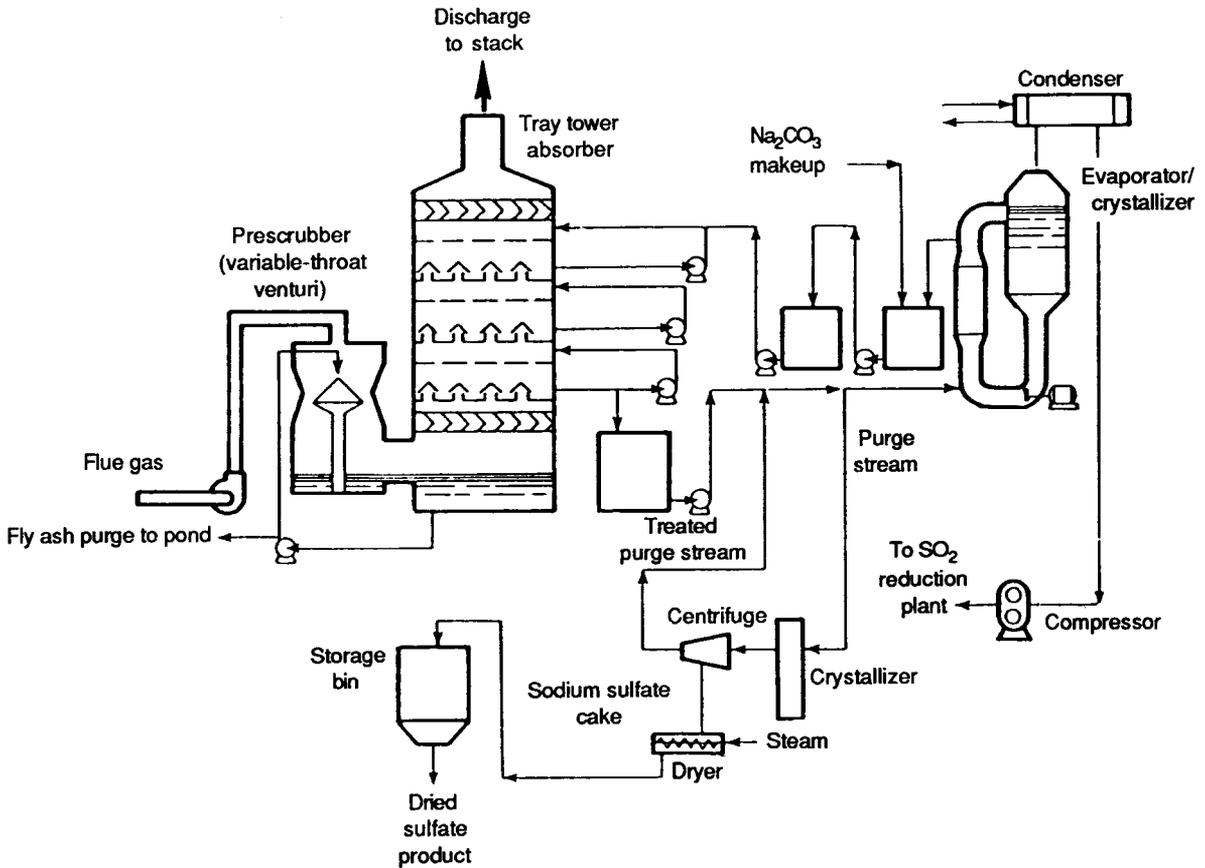


FIG. 4.32 Wellman-Lord flue gas desulfurization process (71).

preferential affinity for the contaminant(s) of concern and a large ratio of surface area to volume derived from a highly porous composition. Activated carbon is the most commonly used adsorbent, and alumina, bauxite, and silica gel are examples of other adsorbents.

Activated carbon adsorbs organic gases and vapors, with and without the presence of water. In the absence of water vapor, silica gel adsorbs organic and inorganic gases. When water vapor is present, alumina, bauxite, and silica gel remove water from the gas and, thus, perform a drying function. Gas adsorption practice normally focuses on the removal of volatile organic compounds using activated carbon.

The adsorptive capacity of the activated carbon system is favored by the following relative conditions

- Higher-molecular-weight compounds
- Unsaturated compounds
- Cyclical structured compounds
- Lower operating temperature
- Higher compound concentrations
- Lower vapor pressures

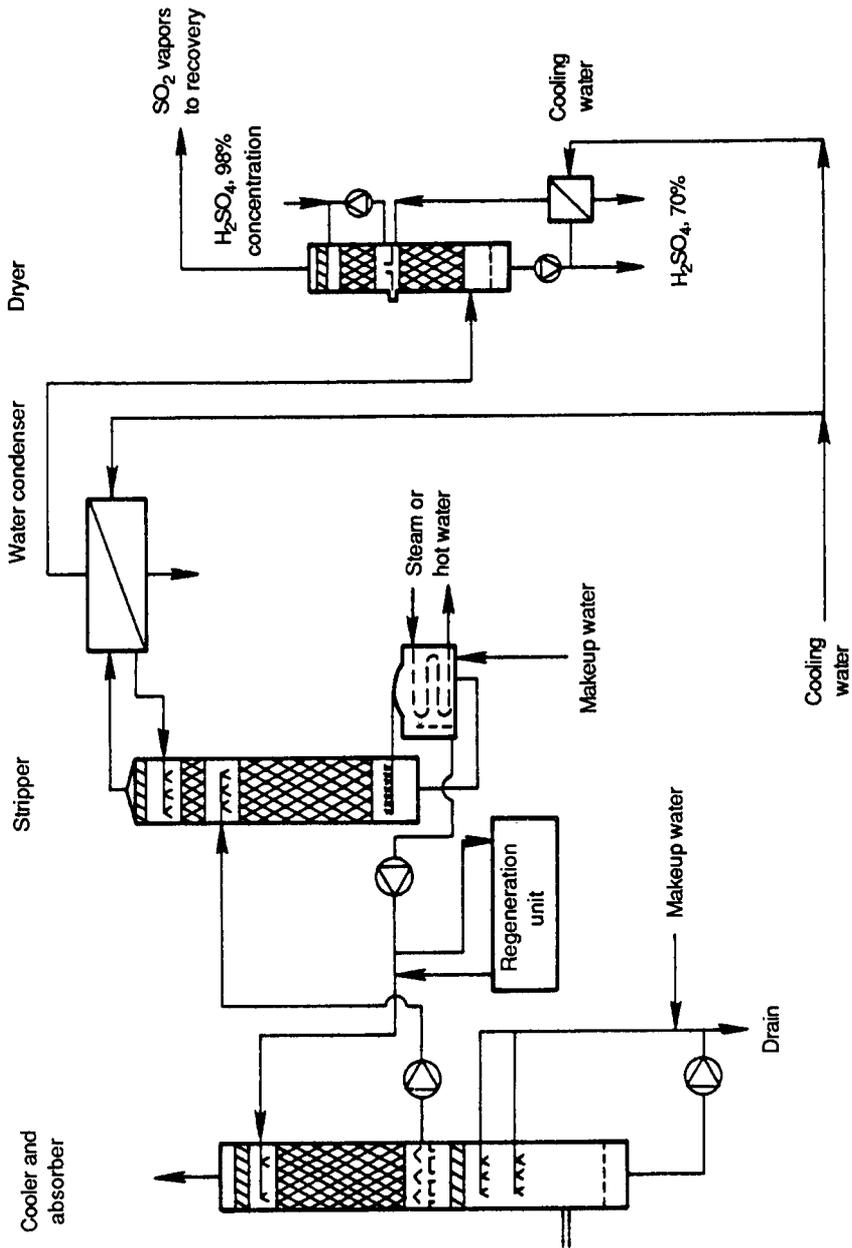


FIG. 4.33 Citrate flue gas desulfurization process (36).

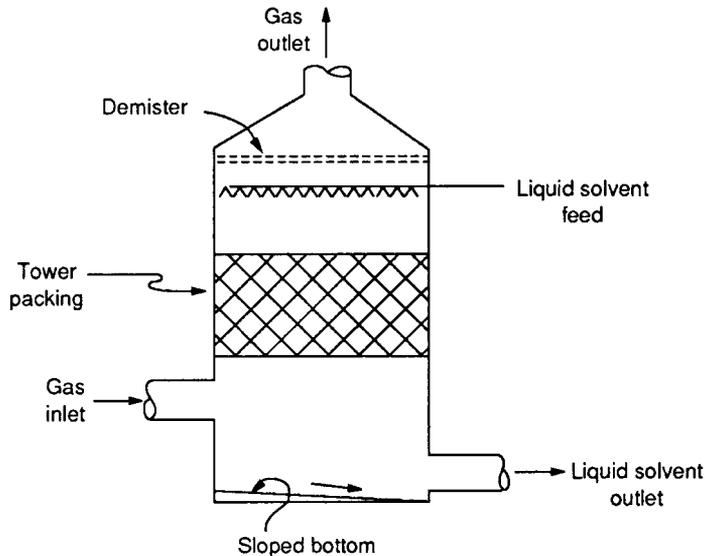


FIG. 4.34 Countercurrent packed tower absorber.

Adsorption isotherms are used to express a specific abatement scheme by showing the amount of contaminant adsorbed with respect to equilibrium pressure or concentration at a constant temperature.

Adsorber Systems. The two most common adsorber systems use fixed bed and fluidized bed units. Other types and configurations are available as supplier options.

Fixed-Bed Adsorber. In the simplest case, a fixed-bed adsorber works as a batch operation using a single adsorber unit. As a minimum, the system must have two units so that one is operable during the time required to regenerate the parallel unit. Figure 4.37 (page 4.61) illustrates this configuration. Multiple adsorbers may be used in series, provided a “standby” unit is available.

The progression of adsorption through the unit begins near the inlet and moves toward the outlet. *Breakthrough* is the term used to describe when traces of contaminants are leaving the adsorber unit and signifying that the parallel standby unit should be placed in service.

Fluidized-Bed Adsorbers. Continuous adsorption is obtained by use of fluidized-bed adsorbers with countercurrent flow of the gas stream and adsorbent. Gas enters the bottom of the adsorber unit and flows axially through the unit. Fresh or regenerated adsorbent is added at the top of the unit while there is a continuous removal of spent or saturated adsorbent removed at the bottom. Adsorbent regeneration occurs off-line.

Adsorber Design. For a specific application, adsorbent suppliers and equipment manufacturers are often a good source of design data related to performance of similar facilities. Technical literature may be used, but adsorption references are generally more focused on adsorption from liquids than from gas streams.

Even though there are a variety of adsorbents used to remove a very wide range of vapor and gaseous compounds, adsorber design may still be discussed with respect to three steps: (1) pretreatment, (2) adsorption, and (3) regeneration.

Pretreatment. Adsorber units are not suited for treatment of gas streams with appreciable amounts of particulates. Alternate systems, such as scrubbers, may be used to remove particulates, and organics may be removed with fabric filters, or by pretreatment before going through an adsorber unit.

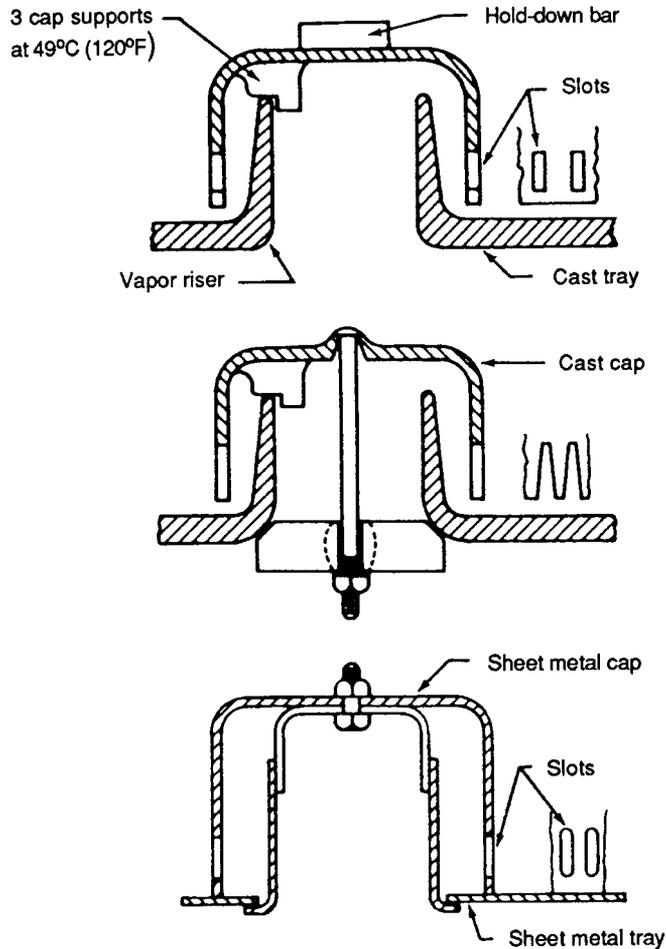


FIG. 4.35 Typical bubble caps (1).

Since the adsorption process favors lower temperatures, heat exchangers are needed if the temperature significantly exceeds about 40°C (100°F).

Dehumidification is a pretreatment process that is normally utilized to maintain relative humidity below 50% so as to minimize the competition of water vapor for adsorption sites in activated carbon systems. Other systems, such as one using silica gel, require total removal of water vapor before pollutant adsorption begins.

When dealing with volatile organic compounds, the design should evaluate flammable or explosive potential in the gas stream and there should be consultation with the insurance carrier if there may be a problem. Lower explosive limit (LEL) data are published for many individual compounds (56). A rule of thumb is to limit volatile organic compound concentrations to 10,000 ppmv (19).

Treatment. The primary treatment process requires definition of the adsorbent best suited for the gas stream, the quantity of the adsorbent, the sizing of the adsorber units, and the piping system and other appurtenances.

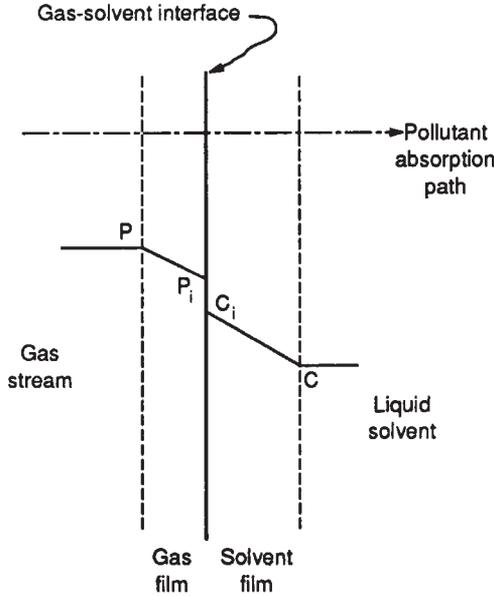


FIG. 4.36 Two-film gas absorption model.

The quantity of adsorbent is based on the adsorption isotherm and pollutant concentration. The number of adsorber units, cycle time, emission flow rate, and molecular weight also enter the calculations. Typically, a safety factor is applied to compensate for fluctuations in the emission stream characteristics and adsorption capacity reductions following repeated regeneration.

Regeneration. Normally, regeneration is accomplished by exposing the adsorbent to high temperatures in the adsorber unit or a special off-line system. For activated carbon, steam regeneration is common practice, and a downstream condenser is used to capture the desorbed pollutants.

The design should use a cost-effective analysis to develop the regeneration system, if, in fact, one is eco-

TABLE 4.23 Design Considerations for Gas Absorbers

Packed tower absorber	Bubble cap plate absorber
Packing material	Plate spacing
Packing volume	Number of plates
Packing placement	Liquid flow rate
Liquid distribution	Gas flow rate
Liquid flow rate	Superficial slot velocity
Gas flow rate	Bubble cap spacing
Tower diameter	Type of bubble cap
Tower height	Weir lengths
Pressure drop	Pressure drop
Column weight	Liquid disposition
Liquid disposition	

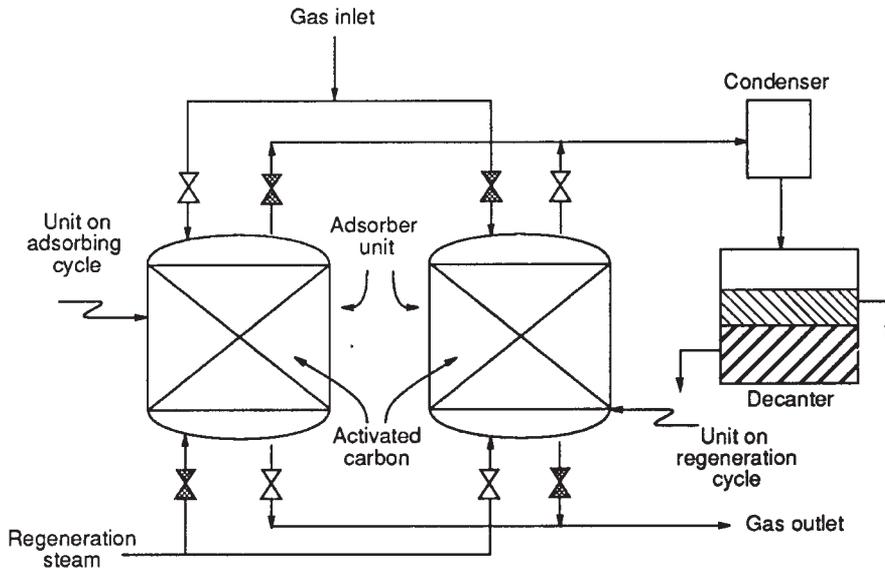


FIG. 4.37 Fixed-bed carbon adsorption system.

nominally favored. For smaller units, the economics may favor replacement of adsorbent with new product. For other cases, only partial regeneration may be justified.

Condensation

In specific cases, contaminants in the form of gases and vapors can be effectively controlled by condensation. In other cases, a condenser is used for pretreatment to reduce the load or potential operational problems with other (often more expensive) air pollution control equipment.

Vapors are condensed to a liquid phase by either increasing the system pressure without a change in temperature or by decreasing the system temperature without a pressure change. Air pollution control condensers typically employ the approach of removing heat from the vapor emission.

Condenser Systems. Condensers are categorized as being a contact or surface type. In a contact condenser, the vapors, coolant, and condensate are all mixed inside the unit's shell. On the other hand, surface condensers operate without the coolant coming in direct contact with the vapor or condensate.

Contact Condensers. Three common types of contact condensers are barometric, spray, and jet units. Each is relatively simple to build and operate, and the coolant is typically water.

The barometric condenser works on a countercurrent flow with the vapor emission entering near the bottom of the unit with a cascading water coolant flow from the upper reaches of the unit. The spray condenser functions with an upflow of the vapor emission through a group of spray nozzles; generally, this unit is similar to a wet scrubber design for removal of particulate matter. A high-velocity jet condenser uses a vapor to enhance mixing of the water coolant with the emission stream.

Despite many advantages of contact condensers over surface condensers, there are two significant disadvantages. One is that contact units require a large volume of coolant, perhaps 10 to 20 times more than a surface unit. The other is that correction of an air pollution problem has now created water pollution problems.

Due to the volume of condensate requiring treatment, it is often too expensive to justify continued use of contact versus surface condensers.

Surface Condensers. Most surface condensers are of the shell-and-tube type. Manufacturers provide units with tube-bundle configurations, which may be horizontal and/or vertical in alignment. Figure 4.38 illustrates a shell-and-tube condenser with a vertical tube-bundle alignment.

Cooling waters flow inside the tubes while vapors condense on the shell side. For surface condensers, the cooling water or other coolant is often reused, since it has not been contaminated by the condensate. A cooling tower or other heat dissipater is needed to dissipate heat gained in the condenser.

In addition to the surface condenser's advantage of producing a relatively small volume of condensate for wastewater treatment, the surface condenser may be recovering a condensate that can be recycled to the source as a recovered by-product.

Condenser Design. The design approach and complexity depends on the number and nature of condensible pollutants in the emission stream. Detailed design information is available from suppliers and technical references (1, 19, 55, 57).

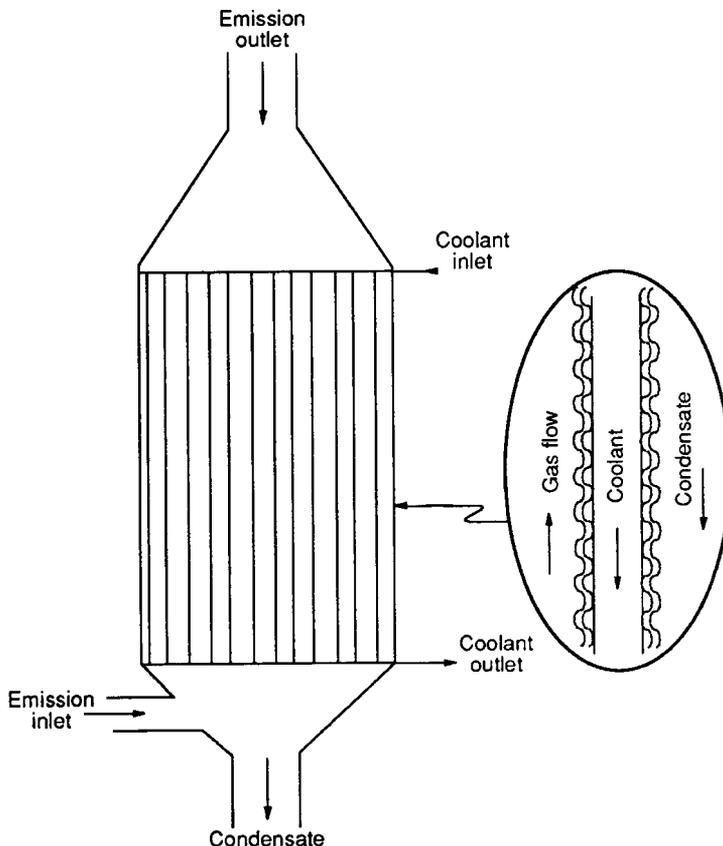


FIG. 4.38 Shell-and-tube condenser.

The design process involves determination of the condensation temperature required, coolant selection, condenser sizing, and coolant requirements. If water vapor is present in the emission stream, pretreatment is needed for dehumidification, such as by heat exchanger, so that water (or, in some cases, ice) will not interfere with the condenser's purpose of removing vapor contaminants.

Flaring

For some time, flares (open flames) have been used to burn waste gases that are not economical to recover or are the results of intermittent, uncertain, or emergency process operations (19, 58).

Flare Systems. The more common types of flares are steam-assisted, air-assisted, and pressure head. Typically, the flare operation is classified as smokeless, nonsmokeless, or fired.

Smokeless operations are needed for destruction of organics that are heavier than methane, and they normally use steam or air to provide efficient mixing to complete the combustion process. The nonsmokeless operation is simply the burning of material that produces smoke. Fired (also called endothermic) flares are those that require additional energy to provide complete combustion. Steam-assisted, smokeless flares are the most frequently used and are the focus of this section. Figure 4.39 illustrates a typical system.

The first treatment process component in the typical flare system is a knockout drum to collect water

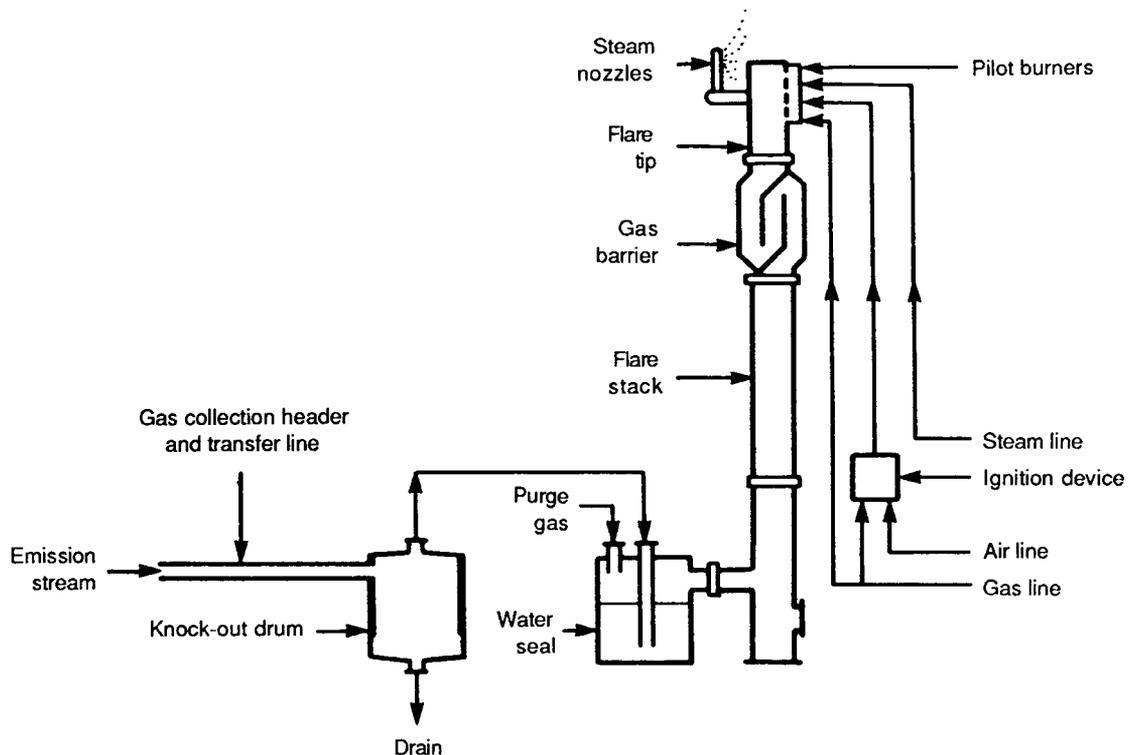


FIG. 4.39 Steam-assisted flare system schematic (19).

droplets, which may extinguish the flare, and organic droplets, which can produce burning particles in the final emission. Next, a water seal, stack seal, or other means is used to prevent flame flashback during low emission flow rates.

Flare Design. Under ideal conditions, complete combustion of volatile organic compounds could occur. However, in practice, the designer's goal is to achieve 98% or greater efficiency. Studies have shown that a minimum of 11,200 kJ/m³ (300 Btu/ft³) of net heating value is required for steam- (and air-) assisted flares. (For unassisted flares, multiply by 0.667.) Often, an active or standby supplemental fuel, such as natural gas, is necessary to maintain heating values in the emission stream.

For design of steam-assisted flares, calculations are needed to determine supplementary fuel requirements, flare gas flow rate, flare gas heat content, flare gas exit velocity, and steam requirements. Equations are presented based on U.S. Customary units; conversion factors are 1.06 kJ/Btu, 0.0283 m³/ft³, and 1.61 (N-m³/h)/(scfm).

Supplemental Fuel Requirements. When a supplemental fuel supply is needed, the following equation may be used to calculate the natural gas flow rate:

$$Q_f = \frac{(300 - h_e)Q_e}{582}$$

where Q_f = natural gas flow rate, scfm
 h_e = emission heat content, Btu/scf
 Q_e = emission flow rate, scfm

If h_e is greater than or equal to 300 Btu/scf, Q_f equals zero.

Flare Gas Flow Rate. The flare gas flow rate, Q_{fg} , is equal to the sum of the emission flow rate Q_e and the flow rate of any required supplemental fuel Q_f .

Flare Gas Heat Content. When the emission heat content h_e is 300 Btu/scf (design minimum) or more, the flare gas heat content h_{fg} equals h_e . Otherwise, supplemental fuel is added to increase the heat content such that h_e equals h_{fg} equals 300 Btu/scf.

Flare Gas Exit Velocity. In order to maintain flame stability, very low flare gas exit velocities should be avoided; a minimum recommended velocity is 0.03 ft/s (59). Using flare exit velocity as a function of heat content, Table 4.24 provides guidance on maximum flare gas exit velocities, U_{max} .

The exit velocity calculation begins with

$$U_{fg} = \frac{3.06Q_{fg,a}}{D_{tip}^2}$$

TABLE 4.24 Flare Gas Exit Velocities (19)

Flare gas heat content* h_{fg} , Btu/scf	Maximum exit velocity, U_{max} , ft/s
$h_{fg} < 300$	— [†]
$300 \leq h_{fg} < 1,000$	$3.28 [10^{(0.0118h_{fg}+0.908)}]$
$h_{fg} \geq 1,000$	400

*If no supplementary fuel is used, $h_{fg} = h_e$.

[†]Based on studies conducted by EPA, waste gases having heating values less than 300 Btu/scf are not assured of achieving 98% destruction efficiency when they are flared in steam-assisted flares.

where U_{fg} = flare gas exit velocity, ft/s
 $Q_{\text{fg,a}}$ = flare gas flow rate at actual conditions, acfm
 D_{tip} = flare tip diameter, in

Knowing the flare gas temperature T_{fg} (°F), the actual flow rate condition is

$$Q_{\text{fg,a}} = \frac{Q_{\text{fg}}(T_{\text{fg}} + 460)}{530}$$

Steam Requirements. The mass-to-mass ratio of steam to gas may range from 0.15 to 0.50. In order to achieve the desired 98% combustion, a ratio of 0.4 lb steam per pound of flare gas is recommended (60). The steam flow equation is

$$Q_s = 1.03 \times 10^{-3} \times Q_{\text{fg}} \times MW_{\text{fg}}$$

or

$$Q_s = 1.03 \times 10^{-3}(16.7Q_f + Q_e MW_e)$$

where Q_s = steam requirement, lb/min
 MW = mean molecular weight, lb-mole

Incineration

Incinerators, also called afterburners, use combustion to convert combustible (volatile) gases, vapors, and/or particulates to carbon dioxide, water, and ash. Primary applications are for control of volatile organic compounds, including hazardous wastes, and odorous compounds, but the applications are sensitive to sources that do not have low gas flow rates and low particulate concentrations. Generally, incineration is not cost- and energy-effective for the sole removal of particulate matter. Also, emissions containing significant quantities of such pollutants as halogens and sulfur will require scrubbers or other control devices for treatment of flue gases (1, 8, 19).

Incinerator Systems. Two basic types of incinerators are used as air pollution control devices: thermal (direct-fired) and catalytic. In both cases, the process requires supplementary fuel, since the gas stream volatiles are almost always below concentrations required to sustain combustion. In addition, due to operating problems, catalytic incinerators are severely limited in applications with a very high concentration of particulates.

Thermal Incinerators. Thermal incinerators operate at high temperatures and are widely used. The combustion temperature and residence time are the primary variables affecting the incinerator's destruction efficiency. Figure 4.40 is a schematic diagram of a typical thermal incinerator system with a heat exchanger. Dilution air, combustion air, and flue gas treatment features may or may not be required, depending on the specific application.

Gas characteristics are also important. For example, halogenated compounds require higher temperatures and longer residence times for complete oxidation than do unsubstituted organics. Table 4.25 presents design variables for these two conditions.

The flue gas from a thermal incinerator is at a high temperature and contains economically recoverable heat energy. Typical installations use heat exchangers that provide preheating of the gas emission stream before it enters the combustion chamber.

Catalytic Incinerators. Catalytic incineration operates by passing a preheated gas stream through a cata-

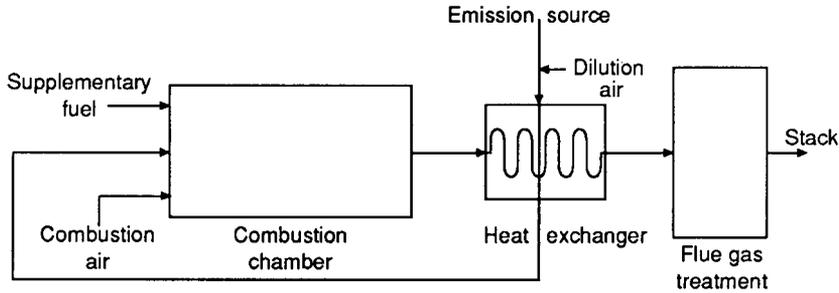


FIG. 4.40 Thermal incinerator system schematic.

lyst bed to oxidize the combustible emissions. The catalyst is used to initiate and promote combustion at much lower temperatures than those required for thermal incineration.

A schematic diagram of a catalytic incinerator system is presented in Figure 4.41. A heat exchanger is an option for systems with recuperative heat exchange, i.e., heat transfer between two gas streams. Site-specific conditions define any needs or requirements for dilution air, combustion air, and/or flue gas treatment.

Normally, catalysts are metals of the platinum family that are thinly coated on an inert support material. The catalyst bed may be a metal mesh mat, ceramic honeycomb, or other configurations designed to maximize surface area. In some cases, the catalysts may be in sphere or pellet form. Designers are cautioned in the use of catalytic incinerators for (organic) particulate removal because particulate residue can coat the catalyst, which reduces the surface area and inhibits the process performance.

In addition to gas characteristics, the performance of a catalytic incinerator is a function of catalyst properties, operating temperature in and out of the catalyst bed, and space velocity, which is the total volumetric flow rate entering the catalyst bed divided by the bed's volume.

Particularly for volatile organic compounds, catalytic incinerators are capable of essentially complete destruction of the pollutants. But, cost-effective destruction efficiency values have an upper range of 90 to 95%.

Suggested values and limits for design variables for a fixed-bed catalytic incinerator are presented in Table 4.26. The minimum bed inlet temperature is designated as 315°C (600°F) to ensure an adequate initial reaction rate. The minimum temperature at the bed outlet is a function of combustible material concentration (or heat content); a conservative value of 540°C (1000°F) is designated to ensure an adequate overall reaction rate to achieve the desired destruction efficiency. The maximum outlet temperature is set at 650°C (1200°F) to prevent catalyst deactivation by overheating. For the 90 and 95% destruction efficiency, the

TABLE 4.25 Thermal Incinerator System Design Variables (59, 60)

Required destruction efficiency, %	Nonhalogenated stream		Halogenated stream	
	Combustion temperature, °C (°F)	Residence time, s	Combustion temperature, °C (°F)	Residence time, s
98	870 (1,600)	0.75	1100 (2,000)	1.0
99	980 (1,800)	0.75	1200 (2,200)	1.0

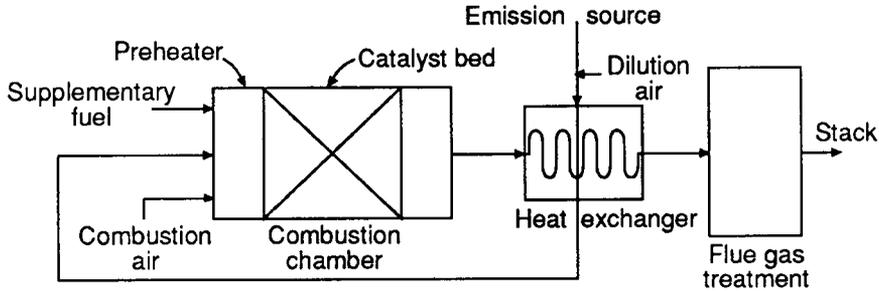


FIG. 4.41 Catalytic incinerator system schematic.

space velocities correspond to 0.042 m³ (1.5 ft³) and 0.057 m³ (2 ft³), respectively, per 1610 m³/h (1000 ft³/min) of combined gas flow.

Incinerator Design. Some design features, such as operating temperatures, were discussed in the presentation on thermal and catalytic incinerators. Discussions follow on fuel (heat) requirements and heat recovery. Other design features include incinerator sizing, burner selection, protection systems, construction material, and instrumentation.

Supplementary Fuel Requirements. Fuel requirements are determined by a heat balance, using the heat of fuel combustion and the sensible heat needed to raise the temperature of the gas and its combustibles up to the required temperature for the desired level of destruction efficiency. The heating value of the combustible emissions (and recovered waste heat) is deducted to determine the net supplementary fuel requirements.

Heat Recovery. Generally, heat recovery from the flue gas is cost-effective. Heat recovery equipment is classified as either recuperative or regenerative. Recuperative systems recover heat continuously and operate in a concurrent, countercurrent, or crosscurrent mode. Fixed- or moving-bed regenerative systems provide heat recovery on an intermittent basis through alternate heating and cooling of a solid. Thus, air and flue gas flows are periodically reversed.

TABLE 4.26 Fixed-Bed Catalytic Incinerator System Design Variables

Required destruction efficiency, %	Temperature at the catalyst bed inlet ^a , °C (°F)	Temperature at the catalyst bed outlet ^b , °C (°F)	Space velocity, SV, h ⁻¹
90	315 (600)	535–630 (1000–1200)	40,000 ^c
95	315 (600)	535–630 (1000–1200)	30,000 ^d

^aMinimum temperature of combined gas stream (emission stream + supplementary fuel combustion products) entering the catalyst bed is designated as 315°C (600°F) to ensure an adequate initial reaction rate.

^bMinimum temperature of flue gas leaving the catalyst bed is designated as 535°C (1000°F) to ensure an adequate overall reaction rate to achieve the required destruction efficiency. Note that this is a conservative value; it is in general a function of the HAP concentration (or heat content) and a temperature lower than 535°C (1000°F) may be sufficient to achieve the required destruction level. Maximum temperature of flue gas leaving the catalyst bed is limited to 630°C (120°F) to prevent catalyst deactivation by overheating.

^cCorresponds to 0.04 m³ (1.5 ft³) of catalyst per 1610 N-m³/h (1000 scf/min) of combined gas stream.

^dCorresponds to 0.06 m³ (2.0 ft³) of catalyst per 1610 N-m³/h (1000 scf/min) of combined gas stream.

Source: Refs. 59, 61, 62.

FUGITIVE EMISSIONS

In individual cases, fugitive emissions may be sufficient to cause violations of ambient air quality standards even after implementation of source controls. This is particularly likely with respect to emissions of particulate matter. The sources of fugitive emissions are categorized as fugitive industrial process (particulate and chemical) emissions or as fugitive dust (8, 63–68).

Fugitive Industrial Particulate Emissions

Particulates from an industry-related activity that are not collected through a primary exhaust or control system are known as industrial process fugitive emissions. The primary control techniques are ventilation systems, process optimization, and suppression.

Fugitive Particulate Sources. Industrial particulate fugitive emissions may originate from indoor or outdoor operations. The indoor sources include materials handling, storage activities, and other industrial operations, and outdoor sources include mining operations, rock crushing, sandblasting, and other activities. Table 4.27 lists a number of emission sources and shows applicable control techniques.

Emission Controls. As shown in Table 4.27, ventilation and collection, process optimization, and wet suppression are the primary controls for industrial process fugitive emissions. The evaluation of control alterna-

TABLE 4.27 Industrial Process Fugitive Emission Sources and Applicable Control Techniques (63)

Source	Ventilation and collection	Process optimization	Wet suppression
Material handling			
Transferring or conveying	x	x	x
Loading–unloading	x	x	x
Storage	x	x	x
Bagging or packaging	x		x
Material crushing and screening	x	x	x
Mineral mining			
Drilling	x		
Blasting		x	
Extraction		x	x
Waste disposal (tailings)			x
Metallurgical operations			
Furnace charging	x	x	
Furnace tapping (product and slag)	x	x	
Casting	x	x	
Mold preparation	x	x	x
Casting shakeout	x	x	
Slag disposal		x	x
Sintering	x	x	
Coke oven charging		x	
Coking (leaks)		x	
Coke pushing	x	x	
Coke quenching		x	

tives is site-specific and involves consideration of the industrial process and facilities, the characteristics of the emission(s), and the secondary impacts of the control techniques.

Ventilation Systems. Localized enclosures and hoods and building ventilation are two techniques to control sources of industrial process fugitive emissions. The former is generally most desirable and cost-effective.

Common sources amenable to localized controls include processes such as (8, 63, 64, 69–71):

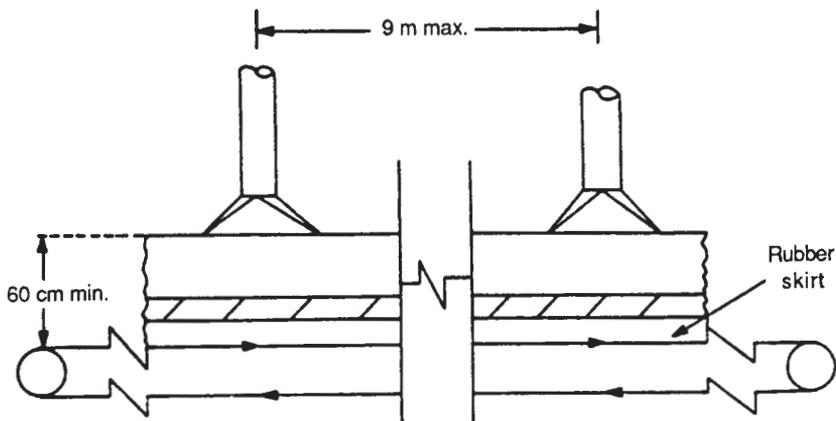
- *Materials handling:* Conveyors, elevators, feeders, loading and unloading, product bagging, and storage silos
- *Solids beneficiation:* Crushing, screening, and other classifying operations

These and other sources are included in Table 4.27.

Several examples are used to illustrate localized controls. Figure 4.42 shows a covered conveyor with exhaust hoods; production transfer points require special attention. In Figure 4.43, alternate systems for a conveyor loading a hopper with a storage bin are presented. A related case of a chute loading a storage bin is illustrated in Figure 4.44. The last control example is the use of a hood at a bag-filling operation, as shown in Figure 4.45.

In practice, the closer the hood is to the dust source, the better the collection efficiency and the less air is needed for the collection process. Enclosures and associated exhaust hoods should be designed to provide a smooth flow of air at an appropriate control velocity.

The capture velocity is usually based on published data for velocities of released particles and on the velocity of ambient air. The potential impact of capture velocity on uptake of valuable process products should be made.



Cover conveyor between transfer points.
 Exhaust at transfer points as required.
 Exhaust minimum additional $0.5 \text{ m}^3/\text{s}$ per inch of
 belt width at a maximum of 9-m intervals.
 Entry loss = entry loss factor for tapered
 hood X duct velocity pressure.
 Duct velocity = 1100 m/min minimum.

FIG. 4.42 Belt conveyor ventilation for fugitive emissions control (72).

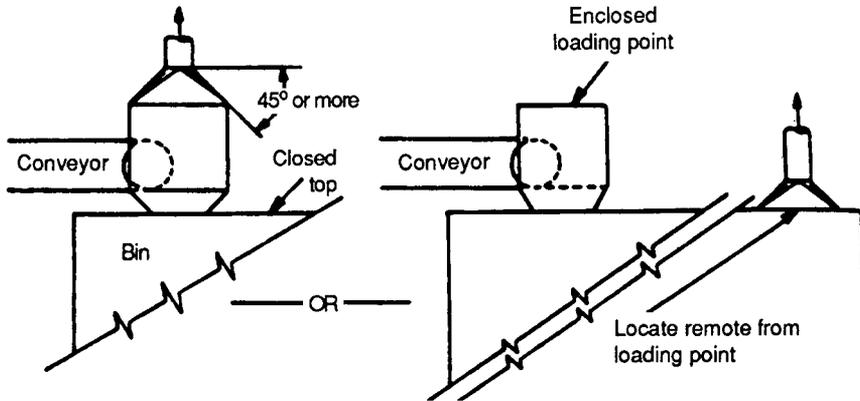


FIG. 4.43 Conveyor-hopper-bin ventilation for fugitive emissions control (72).

The ventilation system design considers the relationship of velocity and volume. When collection is near the source, systems using high velocity and low volume are generally preferred, since they have minimal requirements for ductwork, pollution control equipment, and makeup air. Requirements for higher air volumes are increased when the collection point is more remote from the source.

The more localized ventilation systems will generally use dry dust collectors, such as cyclones and fabric filters. Dry collectors have the advantage of relatively low life-cycle costs and ease of collected dust disposal.

Ventilation controls for an entire building, or segregated portion thereof, may be the only feasible approach when there are numerous minor sources combining to cause a problem situation. Fabric filters are normally used for removing particulates. Due to the large airflow rates required and a potential for unsafe occupational exposure, general area ventilation systems are not a first choice alternative.

Process Optimization. Many options are available to industry on a case-by-case basis to control fugitive emissions. Generalized approaches include proper operation and maintenance of plant equipment, prompt

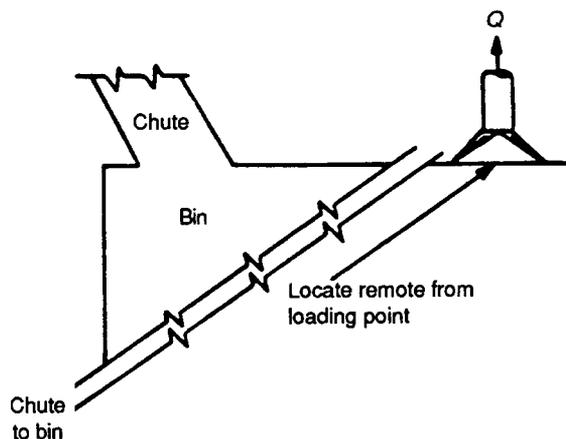


FIG. 4.44 Chute-bin ventilation for fugitive emissions control (72).

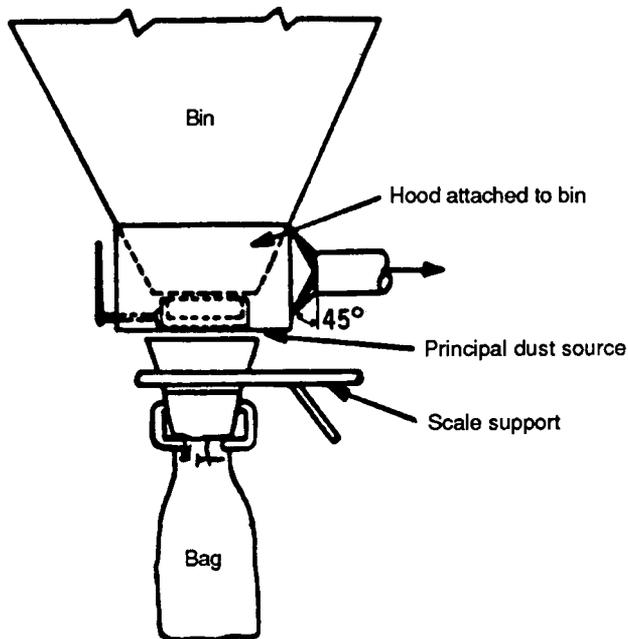


FIG. 4.45 Bag-filling ventilation for fugitive emissions control (72).

cleanup of spillage before it becomes airborne, and partial or complete seclusion and/or shielding of fugitive dust sources.

Suppression. Conventional systems using water, chemicals, or foam and special systems, such as electrostatically charged fog, are used to suppress fugitive dust in material handling and beneficiation processes. Chemical additives to water and foam systems are more expensive than water systems but offer the advantage of being more long lasting (63–65, 69, 73, 74).

Fugitive Industrial Chemical Emissions

As with particulates, fugitive chemical emissions, as gas or as liquid emitting gas, may occur as an uncontrolled release from an industrial process, and similar control approaches to those used for particulates may apply. For instance, emissions from a plating tank may be controlled with more effective use of hoods and local ventilation techniques.

Fugitive emissions may also come from leaking equipment. This requires a more unique control technology, such as containment or source control. This is the focus of the following discussion (67, 68, 75).

Fugitive Chemical Sources. There are many potential sources of fugitive emissions in the chemical process and similar industries have many potential sources of fugitive emissions. Categorically, the pollutants of most concern are volatile organic compounds.

A problem arises when a number of relatively small sources from equipment leaks and other nonprocess sources accumulate to an undesirable level. Typical sources include leaks from

Valves
Pumps
Compressors
Pressure-relief valves
Sampling connections
Open-ended lines
Flanges

Table 4.28 provides a comparison of emissions from these sources in terms of individual source emission factors and of total source contribution for volatile organic compounds (VOCs).

Source Controls. Since emission control of these equipment-related sources is unit-specific, control techniques are discussed by source category.

Valves. With the exception of check and relief valves, a stem is used to operate an industrial process valve. A source of a fugitive gas emission comes from a failure of the stem seal. The failure may be caused by improper installation, deterioration over time, and/or wear from use.

Typically, packing glands are used to seal the stem. A wide variety of packing materials are available for site-specific applications. An alternative approach is to use O-rings, which are adaptable to a smaller range of conditions.

Repair varies from simple to complex techniques used on-line, to off-line repair, to complete replacement of the valve and sealant system. Figure 4.46 illustrates primary valve maintenance points for a stemmed valve.

Special valves are available that are designed for little or no leak potential. Bellows sealed and diaphragm valves are two examples and are illustrated in Figures 4.47 and 4.48, respectively. The bellows sealed valves are the least prone to leaking.

Pumps. The typical fugitive gas emissions from pumps occur as a result of sealant failure where the moving shaft meets the stationary casing. The common sealants are packed and mechanical seals.

Reciprocating and rotating shafts successfully use packed seals. Figure 4.49 illustrates a typical design of a packed seal. Lubricants are required between the moving shaft and the stationary packing to control friction-generated heat between the two materials.

The potential for leaks is further reduced by use of mechanical seals; however, these seals are used only

TABLE 4.28 Comparison of Fugitive Emissions from Equipment Types (68)

Equipment type	Process fluid	Emission factor, kg/h	Percent of total VOC fugitive emissions
Valve	Gas/vapor	0.0056	47
	Light liquid	0.0071	
	Heavy liquid	0.00023	
Pump	Light liquid	0.0494	16
	Heavy liquid	0.0214	
Compressor	—	0.228	4
Pressure-relief valve	Gas/vapor	0.1040	9
Sampling connection	—	0.0150	3
Open-ended line	—	0.0017	6
Flange	—	0.00083	15
			100

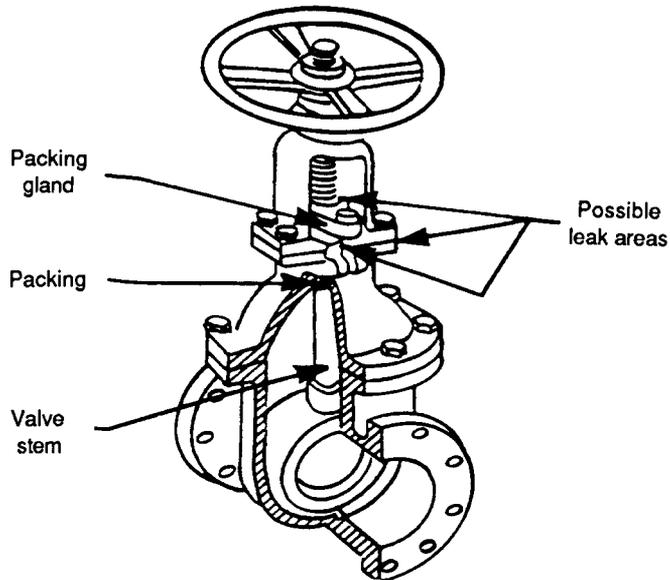


FIG. 4.46 Primary maintenance points for a valve stem (72).

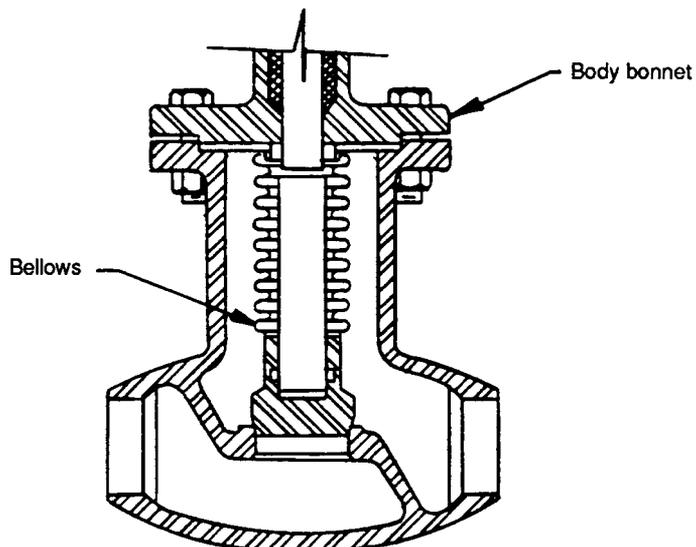


FIG. 4.47 Typical design of a bellows sealed valve (72).

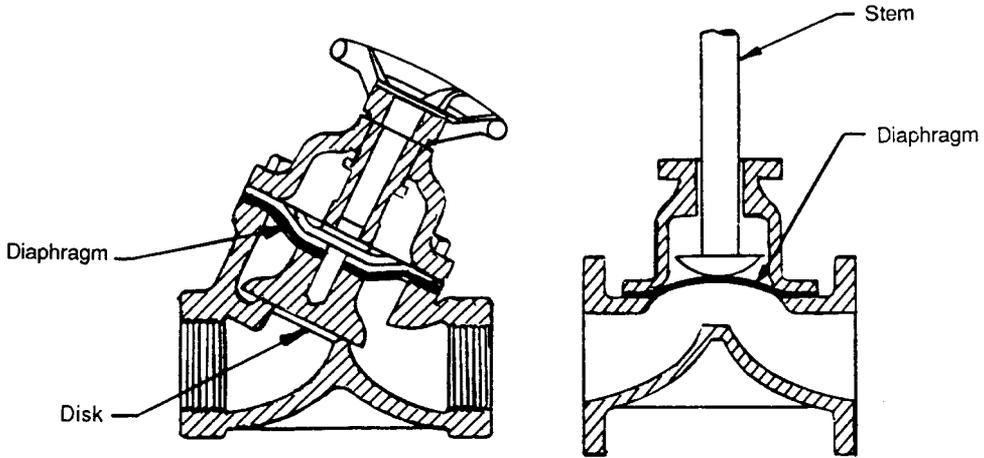


FIG. 4.48 Typical design of diaphragm valve (67).

with rotating shafts. Leak potential may be further reduced by utilizing dual redundant seals. Mechanical seal designs incorporate a lapped seal face between a stationary element and a rotating seal ring. Figures 4.50 and 4.51 show key elements of a basic design for a single pump seal and of alternate designs for dual seals. Close tolerances are a must for effective use of mechanical seals.

Canned-motor, diaphragm, and magnetic-drive pumps have unique operating characteristics that do not require a sealant to control leakage.

Compressors. Most concepts for fugitive emission control from pumps apply to compressors. Typical

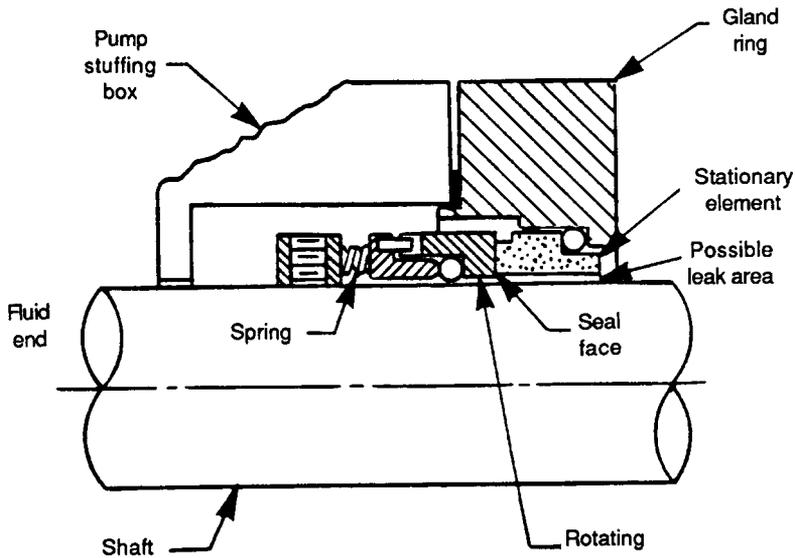


FIG. 4.49 Typical design of a packed sealed pump shaft (67).

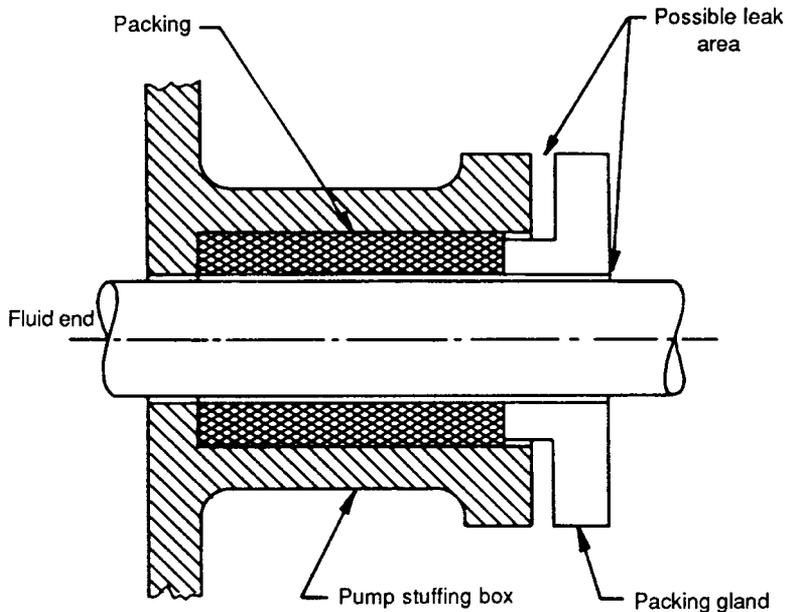


FIG. 4.50 Typical design of a single mechanical pump seal (67).

control applications are packed seals for reciprocating compressors and mechanical seals for rotating (shaft) compressors. Some newer sealing systems include vents to a control device.

Pressure-Relief Devices. A spring-loaded pressure-relief valve is the most common device used to alleviate excessive operating pressures. The valve opens at a set pressure and then reseats when the operating pressure returns to below the set value. Reseating is often a source of fugitive emissions, and the source may vary from a single improper reseat to continuous failure due to a degraded seating element. Figure 4.52 illustrates a pressure-relief valve and rupture disk device to control fugitive gas emissions.

A rupture disk pressure-relief device does not allow any emissions until the pressure is so large as to cause the disk to rupture. Another fugitive emission control scheme is to route any discharge from a pressure-relief device to an appropriate control system or device.

Sampling Connections. Since purging of sampling lines is a source of fugitive emissions, closed-purge sampling systems should be used. Two such systems are illustrated in Figure 4.53.

Open-Ended Lines. Particularly in the chemical process industry, a plant may have many open-ended valves and lines to drain, purge, or vent a process fluid. Fugitive gas emission control options include routing to a control device and/or use of plugs, caps, and blind flanges.

Flanges. Generally, flanges and other connectors are the largest category and most widespread source of fugitive emissions in a plant. Problems may arise due to use of an improper flange gasket, to assembly deficiencies among connecting parts, or to inadequate tightening of the flange bolts. The flange gasket must be resistant to expected chemical (corrosion) and physical (thermal) conditions.

Fugitive Dust

Particulates from open sources that become airborne through wind and/or human activity are referred to as fugitive dust. The primary control techniques include wet suppression, stabilization, and specialized tech-

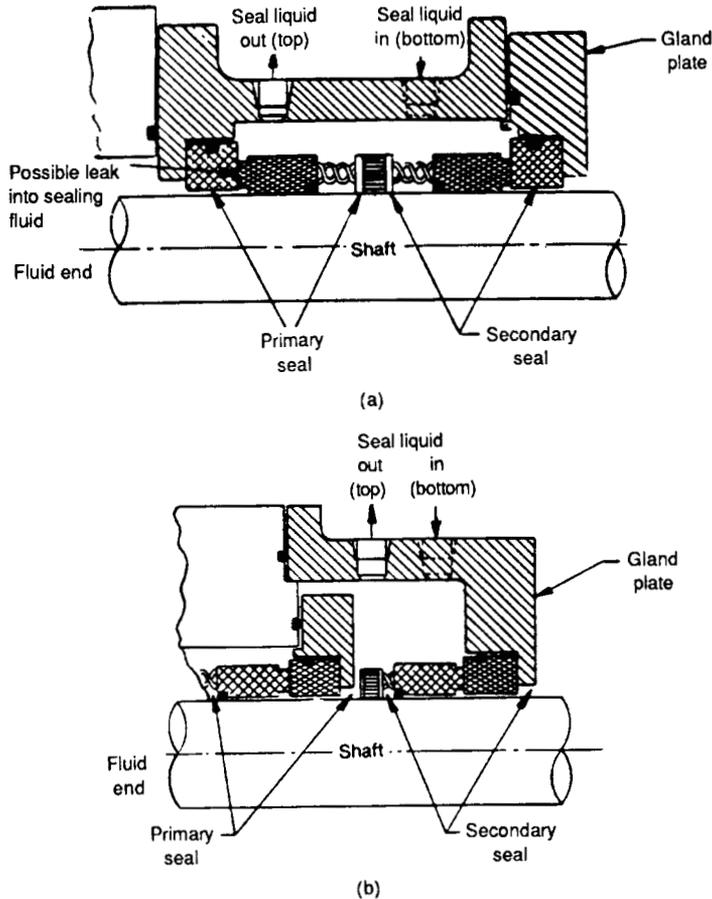


FIG. 4.51 Two arrangements of dual mechanical pump seals. (a) Back-to-back arrangement; (b) tandem arrangement (46).

niques. The special case of fugitive dust control at hazardous waste remedial action sites is discussed in Chapter 9.

Fugitive Dust Sources. Unpaved roads and construction activities are the primary sources of fugitive dust that are of concern. These and other sources are listed in Table 4.29, along with common control techniques for each source.

Dust Controls. In evaluating and selecting the dust control scheme, the designer needs to consider cost effectiveness and pollutant reduction as well as any possible negative impacts on the area's environment. The latter is especially true with respect to applications of chemicals that may degrade ground and/or surface waters.

Wet Suppression. Plain water has been used for many years to suppress fugitive dust. However, this approach is only temporary and must be repeated at regular intervals. Other shortcomings include potential un-

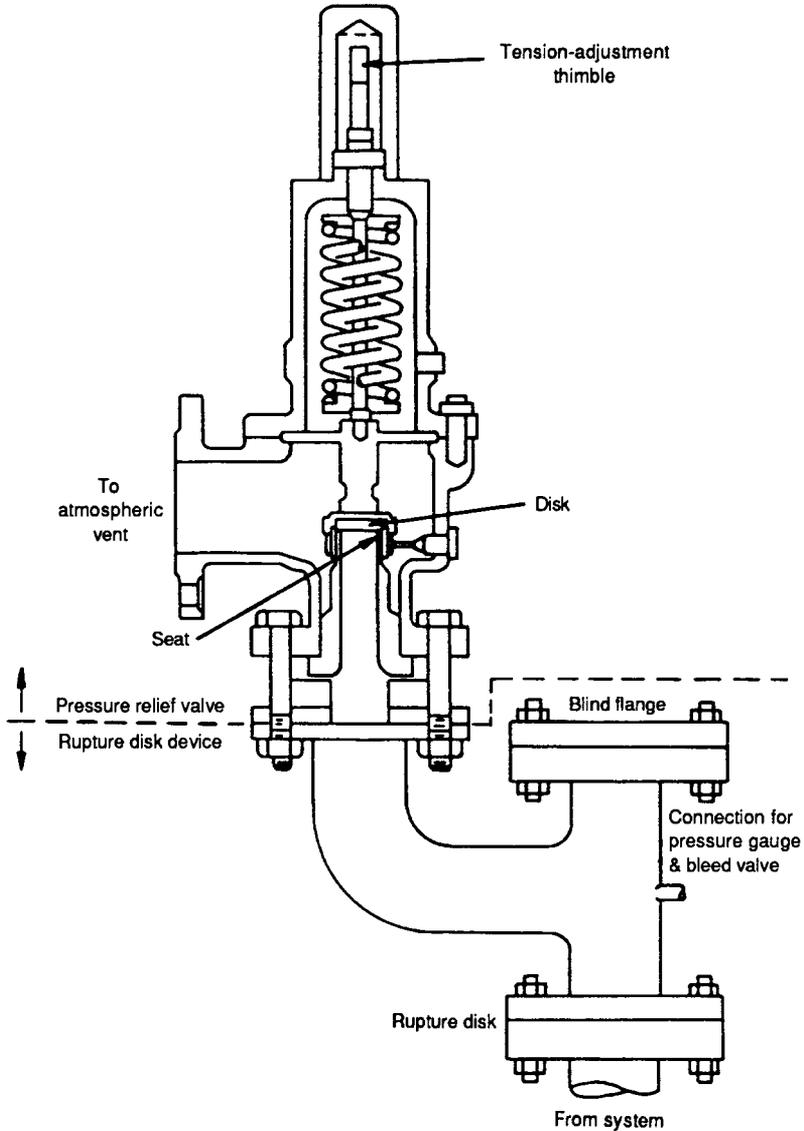


FIG. 4.52 Pressure-relief valve mounted on a rupture disk device (67).

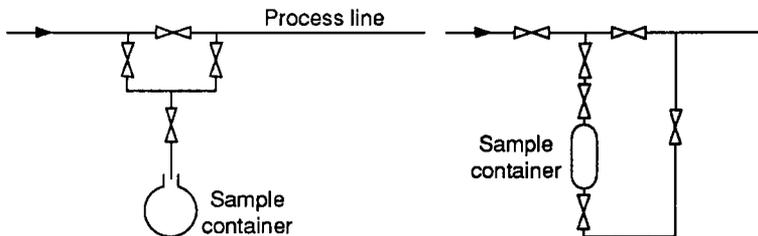


FIG. 4.53 Two arrangements of closed-purge sampling systems (67).

TABLE 4.29 Fugitive Dust Emission Sources and Applicable Control Techniques (63, 67)

Source	Wet suppression	Stabilization	Speed reduction	Surface cleaning transportation controls	Windbreaks	Good operating practices
Unpaved roads	x	x	x			
Construction activities	x			x		x
Dust from paved roads				x		
Off-road motor vehicles			x			
Overburden removal/storage	x					x
Reclamation efforts		x				x
Inactive tailings piles		x			x	
Disturbed soil surfaces		x			x	x
Agricultural tilling					x	

desirable freezing problems in colder areas and inadequate water supplies or sources in arid regions. Wetting agent additions will increase the effective period of wet suppression and, thus, reduce water demands.

Chemical Stabilization. Particulate release from unpaved roads or other exposed surfaces can be reduced or prevented by chemical stabilization that binds dust and surface particles to form a protective crust. Chemical suppressants can be categorized as salts, lignin sulfonate, wetting agents, petroleum derivatives, and special mixtures. Trade names of some tested fugitive dust suppressants are shown in Table 4.30.

Salts, such as calcium chloride, magnesium chloride, and trade name mixtures absorb and retain moisture in the surface layer. Stabilization from wetting agents and surfactants occurs due to lowering of the surface tension of water and the resulting rapid penetration into the surface layer. The other dust suppressants tend to bind fines to larger particles in the surface layer.

Water solution is the normal means of chemical application. The design considerations are

- Application intensity, volume per unit area
- Application frequency, number of applications per unit of time

TABLE 4.30 Classification of Tested Chemical Suppressants (66)

Dust suppressant category	Trade name
Salts	Peladow
	LiquiDow
	Dustgard
	Oil well brine
Lignosulfonates	Lignosite
	Trex
	Biocat
Surfactants Petroleum-based	Petro Tac
	Coherex
	Arco 2200
	Arco 2400
	Generic 2 (QS)
Mixtures	Arcote 220/Flambinder
	Soil Sement

- Dilution ratio, volume of concentrated chemical to volume of water
- Application procedure, truck-mounted sprays, fixed-spray system, or other means

The implementation alternatives for a chemical stabilization system (for unpaved roads) is presented in Table 4.31.

The longevity of a chemically stabilized surface is a function of many factors. For example, heavy traffic (weight, volume, and speed) negatively impacts unpaved roads. Freeze–thaw cycles break down the crust formed by many binding agents. Heavy rains will remove soluble chemicals from treated surfaces.

Physical Stabilization. In physical stabilization, a cover is placed on exposed surfaces to prevent particles from becoming airborne from wind or machinery action. For example, rock, soil, slag, straw, or other material may be placed over stockpiles or steep slopes (77).

For unpaved roads, ditches, and the like, special fabrics have been developed to stabilize the surface. These fabrics are also referred to as “road carpet,” and are also used to prevent erosion by runoff.

In lieu of road carpet, gravel is an alternative for stabilizing unpaved dirt roads. Of course, paving is the most effective control for fugitive road dust (66, 78).

Vegetative Stabilization. The techniques of deploying vegetation to control (water) erosion are also effective in the stabilization of exposed surfaces to control fugitive dust emissions. In both cases, the area should not be mechanically disturbed after planting (8, 74, 76).

TABLE 4.31 Implementation Alternatives for Chemical Stabilization of an Unpaved Road (66)

Cost elements	Implementation alternatives
Purchase and ship chemical	Ship in railcar tanker (11,000–22,000 gal/tanker) Ship in truck tanker (4000–6000 gal/tanker) Ship in drums via truck (55 gal/drum)
Store chemical	Store on plant property In new storage tank In existing storage tank Needs refurbishing Needs no refurbishing In railcar tanker Own railcar Pay demurrage In truck tanker Own truck Pay demurrage In drums Store in contractor tanks
Prepare road	Use plant-owned grader to minimize ruts and low spots Rent contractor grader Perform no road preparation
Mix chemical and water in application truck	Put chemical in spray truck Pump chemical from storage tank or drums into application truck Pour chemical from drums into application truck, generally using forklift Put water in application truck Pump from river or lake Take from city water line
Apply chemical solution via surface spraying	Use plant-owned application truck Rent contractor application truck

Specialized Techniques. Some fugitive dust controls are relatively specific to the site and/or situation. Examples include

- Vehicle speed reduction—less turbulence
- Surface (street) cleaning—reduces reentrainment
- Traffic control—reduces vehicle access
- Windbreaks—lessens surface wind speed
- Good operating practices

VOLATILE ORGANIC COMPOUND CONTROL

The Clean Air Act Amendments (CAAA) of 1990 increased regulations of volatile organic compounds (VOCs) and hazardous air pollutants (HAPs). In particular, Section 112 deals with the control of HAPs, by industry, and lists 188 chemical compounds and groups that must use Maximum Achievable Control Technology (MACT). Because of groups of compounds in this list, there are actually about 400 regulated HAPs. Generally, the regulations establish stringent standards that require reductions of emissions by 95–98% and are loaded with complex record keeping. These regulations include construction and operating permit provisions and allow for emissions averaging.

VOCs: Sources and Effects

Volatile organic compounds refers to emissions that readily vaporize at relatively low temperatures. Standards have been promulgated for more than twenty commercial source categories. A distribution of VOC sources in the United States is shown in Table 4.32, for all sources and for stationary sources.

On a mass emission basis, the top fourteen VOCs and HAPs in the United States are (116):

Toluene	<i>m</i> -Xylene	Chlorobenzene
Formaldehyde	Benzene	Acetic acid
Methylene chloride	<i>o</i> -Xylene	Trichlorotrifluoroethane
Methyl chloroform	Perchloroethylene	Trichloroethylene
Ethylene	<i>p</i> -Xylene	

TABLE 4.32 Sources of VOCs in the United States (116)

Source	All Sources, %	Stationary Sources, %
Mobile sources	32	—
Miscellaneous solvent uses	16	23
Surface coating	14	20
Miscellaneous sources	13	20
Petroleum marketing	10	14
Hazardous waste TDSF	8	12
Petroleum refining	3	5
Chemical manufacture	2	3
Industrial processes	2	2

Note: TDSF is treatment, storage, and disposal facilities.

Mass of emissions alone does not evidence the relative severity of VOCs or other toxic pollutants. The toxicity of a compound and the degree of exposure are equally important and must be considered together. For example, toluene is the most emitted compound but is less toxic than benzene, a carcinogen linked to leukemia. Many VOC compounds are suspected carcinogens.

Characterization

All regulations that aim to limit emissions must define the regulated quantities, whether by quality or concentration. Therefore, it is extremely important to consider the testing methods used to arrive at the estimates of emissions before and after the use of controls (117).

Since all methods result in estimates, with variations due to the test method, statistics should be applied to interpret data for environmental compliance. The engineer is urged not only to think in terms of production process and control technologies, but also the measurement of contamination levels, its estimates and variations. It is important to arrive at an understanding with regulators on the sampling and testing methods used, and the way data will be analyzed and conclusions drawn.

Emission Modeling

There are two fundamental aspects of emission modeling. First, an estimation of VOC emissions from a source is necessary. Second, the dispersion of that emission into the atmosphere must be evaluated with respect to potential impacts on both the public and the environment (117).

There are many cases in which it is difficult to measure VOC emissions from a source, such as from open basins. Computer models are often used to estimate emissions. However, there is a wide variation in the results obtained from differing models, and the selection of models should be critically assessed.

Dispersion of emissions into the atmosphere is especially important with hazardous and toxic materials, as with odors. Since controls aim to be risk-based, these dispersion calculations should become a part of the evaluation of any emission control strategy. Similarly, when applied to odors, these dispersion calculations can be varied to allow a determination of the limits of emissions to avoid an unwarranted impact on a neighbor.

VOC Control Methods

This section deals with emission control systems applicable to VOC removals, and comments are made on control technologies applied to VOC emissions. The control technologies for VOC and odor emissions often are the same (117).

Before selecting any emission control system, the engineer should be aware of the fundamentals concerning the VOC to be removed, and the performance requirements of the control device. To remove any compound, the engineer should consider a compound's vapor pressure, solubility in water or other solvents, reactivity, ability to be adsorbed, and its biodegradability. Through a review of these basic characteristics, a good choice can be made of the logical approach to be used for control.

Source Controls. Since the best approach to pollution control is not to pollute in the first place, each process emission should be reviewed for controls or formulation changes that can limit VOC emissions. Pollution prevention using source controls has been very effective in decreasing VOC emissions, such as in the paint and coating, metal finishing, printing, and textile industries.

Collection. Fugitive emissions represent a significant problem in some industries. Careful process design is required to contain emissions and careful collection system design is required to transport these emissions

to treatment and control devices. Improperly designed collection systems will result in unnecessary, untreated emissions. Design of effective collection systems is straightforward, having been dealt with in many textbooks on ventilation and air flow. The engineer is urged to pay attention to the basics and not to compromise duct and fan design, which may lead to inadequate collection, excessive leaks, high power consumption, or system fouling.

Condensation. VOC emissions often can be controlled by condensation when present at an adequate concentration. Provisions have to be made for removal of particulates and moisture. The selection of the applicable condensation temperature will depend on the VOC vapor pressure and the percentage to be removed. A wide range of condensation temperatures are obtainable from various media, such as air, water, chilled water, brines, chilled heat transfer fluids such as glycol, “freon”-type compounds, and liquid nitrogen.

Thermal Oxidation. Thermal oxidation is applicable to VOC removal, particularly where emissions are at higher concentrations. Most VOCs are flammable and, thus, decrease the need for added heat to achieve the required thermal oxidation temperatures. Operating costs can be high, and system design should consider various methods of heat reduction and recovery.

There are numerous types of thermal oxidizers, such as catalytic recuperative, thermal recuperative, catalytic regenerative, and thermal regenerative; flares and rotary concentrators may be used as supplements. In general, when the concentration of the stream is 15–20% of LEL (lower explosive limit), then both catalytic recuperative or thermal recuperative are applicable. Above that level, thermal recuperative is preferred. A rotary concentrator expands the range of applicability of a thermal oxidizer by concentrating the contaminant, which decreases the size and operating cost of the thermal oxidation equipment.

Adsorption. Carbon adsorption has wide application in the control of VOCs. However, it has special operating requirements and high costs, and each application should be carefully examined. Carbon adsorption is usually a batch operation with multiple beds, allowing for some beds to be regenerated while others are still on-line. Spent carbon is normally regenerated with steam, and the volatiles are subsequently condensed and separated.

The advantage of carbon adsorption is that it is versatile and can recover solvents for reuse. Carbon adsorbers are used to treat lower concentrations of emissions than condensers. Also, carbon adsorption is frequently used in hybrid systems for its “polishing” capabilities, after prior stages, such as condensation or scrubbing, have removed the major contaminants.

Particulates can cause high pressure drops, and heat of absorption must be controlled to avoid ignition. Moisture greatly limits absorption capacity. Regenerated carbon still retains some residual contaminants, and therefore, some concentration of contaminants will always pass through.

Carbon absorption is not applicable to compounds with high volatility that boil below 0°C. Less volatile compounds may have strong attractions to the carbon and are difficult to remove. Care must be taken in design to avoid exothermic reactions, which can cause hot spots or bed fires.

Wet Scrubbers. There are hundreds of different designs of wet scrubbers. This section will specifically comment on the applicability of major designs of wet scrubbers to VOC removal.

Recirculating Scrubbers. The most common recirculating scrubbers are packed columns or coarse spray designs, either in a vertical or horizontal configuration. Vertical packed columns are the most common wet scrubbers in use. The engineer should avoid conditions that will plug the packing.

Virtually all recirculating wet scrubbers fail at removing VOCs, because whatever VOCs are removed are subsequently revolatilized. The only way VOCs could be effectively removed would be to use a once-through operation. In almost all cases for recirculating scrubbers, this is not practical, since the liquid-to-gas ratio is large.

Possible applications for recirculating scrubbers are: where the VOC is at high temperatures and some re-

moval can be achieved by the scrubber acting as a direct contact condenser; where the VOC has high water solubility; and where the VOC can be chemically reacted and bound so as not to revolatize.

Nonrecirculating Scrubbers. Nonrecirculating or fog scrubbing overcomes problems of high liquid-to-gas ratios by generating an enormous amount of surface area with fine droplets of less than 10 microns. The amount of liquid required for effective scrubbing is greatly decreased, often to 1 gal/min per 10,000 cfm (0.063 L/s per 4.7 m³/s), and this makes a once-through operation practical. This scrubbing technique has been found to be effective both for removal of VOCs by adsorption to the surface of the droplet and for diffusion into the droplet for solution and chemical reaction.

Biological Systems. A number of VOCs are biodegradable through proper selection of biota, temperature, moisture, pH, and nutrients, so that the biological medium is active. There are practical limitations of biological systems because of the space required, maximum limits of contaminant concentrations, limits of air flow quantities, and power consumption. Biological media decompose and must be replaced, which adds to the costs. Nevertheless, biofilters have found applications for VOC control, most often to treat wastewater odors.

Hybrid Systems. There are many combinations of technologies, called hybrid systems, which are more beneficial than single systems. Examples are: absorption and recycling (where the solvent is water); absorption in water (using fog scrubbing) and biotreatment; combustion air supplementation; fuel oil absorption before burning; and concentration/condensation.

ODOR CONTROL

Compounds which produce offensive odors are discussed in this section, with emphasis on alternative control methods. Odor abatement techniques are similar to those used to reduce gas and vapor pollutant emissions, except that adjustments are needed to accommodate the lower concentrations of odorous compounds.

As a result of the numerous and often interacting aesthetic, analytical, and legal aspects of odor control, project- and site-specific investigations are vital to determine the level of odor control needed. Then, a design development phase can be conducted to determine the most cost-effective technique to satisfy the project requirements (79–84, 118–122).

Odor: Sources and Effects

Potential sources of odors are as vast as the combination of odorous compounds possible in industrial, commercial, and residential applications. Furthermore, the concentration required for the identification of an odor is quite varied. (For selected compounds, Table 4.33 illustrates typical odor thresholds.) For example, at wastewater treatment facilities, hydrogen sulfide (H₂S) is the dominant odor. Other forms of sulfur, nitrogen, and organic compounds contribute odors, which are released from the water surface in areas of increased turbulence.

In the plant influent, odors emanate from reduced sulfur compounds produced under anaerobic conditions in the sewage collection system or treatment process recycles. Oxidized aldehydes, ketones, and carboxylic acids, the result of degradation of carbohydrates, fats, and proteins, also contribute odors. In the preliminary and primary treatment process areas, reduced sulfur compounds are the dominant odor producers.

Screening, grit removal, hydraulic drops across weirs, and turbulence in channels are the primary odor emission points in wastewater treatment works. Also, sludge digestion and biosolids processing can create conditions that release odorous compounds during mixing and transfer operations. As air is introduced during secondary treatment, oxidized compounds create an earthy, musty odor considerably less offensive than the reduced sulfur compounds. As a result, secondary treatment processes require less odor control.

TABLE 4.33 Typical Odor Recognition Thresholds

Compound	ppm by volume
Acetaldehyde	0.2
Acetic acid	
Acetone	100
Acrolein	0.2
Acrylonitrile	20
Allyl chloride	0.5
Ammonia	50
Aniline	1
Benzene	5
Benzyl chloride	0.05
Benzyl sulfide	0.002
Bromine	0.05
Butyric acid	0.001
Carbon disulfide	0.2
Carbon tetrachloride	20
Chloral	0.05
Chlorine	0.3
<i>o</i> -Cresol	0.001
Dimethylacetamide	50
Dimethylformamide	100
Dimethylsulfide	0.001
Diphenyl sulfide	0.005
Ethanol	10
Ethyl acrylate	0.0005
Ethyl mercaptan	0.001
Formaldehyde	
Hydrochloric acid	10
Hydrogen sulfide gas	0.0005
Methanol	100
Methylene chloride	200
Methylene ethyl ketone	10
Methyl isobutyl ketone	0.5
Methyl mercaptan	0.002
Methyl methacrylate	0.2
Monochlorobenzene	0.2
Nitrobenzene	0.005
Perchloroethylene	5
Phenol	0.05
Phosgene	
Phosphine	0.02
Pyridine	0.02
Styrene	0.05
Sulfur dichloride	0.001
Sulfur dioxide	0.5
Toluene	5
Trichloroethylene	20
<i>p</i> -Xylene	0.5

The effects of odors may be as simple as a nuisance or as complex as a threat to life. In between these conditions, various odors may cause nausea or other forms of illness. In the eyes of the public, any odor represents the presence of air pollution.

Characterization

The human nose is extremely sensitive to the detection, intensity, and character of odors. Excluding persons with high sensitivity or insensitivity to odors, “odor panels” are often used to determine threshold concentrations. The panel findings of odor threshold may be reported as the effective dosage at which 100% (ED₁₀₀), 50% (ED₅₀), and 0% (ED₀) of the panelists recognize the odor. The threshold at ED₅₀ is commonly used.

An odor panel may also report odor units per cubic foot, which is defined as the amount of odorous gas necessary to contaminate a cubic foot of air to the detection threshold. This approach is analogous to the dilution to threshold (D/T) ratio. The D/T level is a ratio of the final air volume after dilution to the odor threshold of the initial volume of air. A volume of air that is at or below the odor threshold has a D/T value of one.

Several techniques are available for static or dynamic odor testing. The syringe dilution method is the primary static test. A very popular dynamic test is the force-choice-triangle olfactometer test, which comes from industrial sensory-quality control procedures. The American Society for Testing and Materials provides standardization of odor-testing procedures. Gas chromatography provides an alternate approach to panels.

The sensation of odor intensity varies logarithmically with the concentration of the odorous compound and is expressed by the Webber–Fechner equation

$$I = K \log C$$

where I = odor intensity

K = constant

C = odor concentration

This characteristic intensifies the requirement for highly effective control measures and is complicated when more than one odorous compound is present. The intensities of a mixture of odors may be independent, counteractive, additive, or synergistic.

Odor Control Methods

To better manage air flows and treat air streams that have similar characteristics, exhaust gases from similar process areas are often grouped into air handling zones. To estimate the quantities of air to be treated, the room volume and the number of air changes required to provide adequate odor control and provide protection from the accumulation of hazardous or explosive gases are determined. For example, fire safety codes require that a minimum of 12 air changes per hour be provided to wastewater treatment process areas where people may freely enter. Additional air changes may be required to ensure that negative pressure within a process building is maintained and odors do not escape. In a pump wet well, 24 air changes per hour are required. To minimize the buildup of moisture and corrosive gases, the headspace under covered channels and tanks usually has six to eight air changes per hour.

The various methods of odors control include those presented in Table 4.34. Selection of control alternatives is very source- and odor-type-specific. Also, the point(s) of effectiveness monitoring, such as stack emission, plant property line, or other locations, should be determined.

TABLE 4.34 Odor Control Methods

Method	Process
Source control	Replace or modify operating systems
Absorption	Capture by water or other fluids
Adsorption	Capture by activated carbon
Biological	Oxidation by microorganisms
Chemical	Oxidation of odor components
Condensation	Cooling of odorous steam
Containment	Hold or retain for treatment
Counteraction	Diminished in presence of another odor
Dilution	Dispersion below threshold levels
Incineration	Combustion of odor agent
Masking	Superimpose a fragrance

In addition to odor control, the dispersion of the treated exhaust must be done in a manner that minimizes potential downwind impacts. The stack height must be of sufficient height to avoid the aerodynamic influences of building cavities and wakes. As a guide, stack heights may equal 1.5 to 2.5 times the height of the largest nearby building, and exit velocities may range between 45 and 60 feet per second. Actual stack height designs are determined by dispersion modeling analysis.

Odor/dispersion modeling analyses are performed to validate that selected odor control technologies are sufficient to meet the plant odor criteria. A dispersion modeling analysis is a mathematical method which relates the odorous emission from the stack to ambient air concentrations.

The model inputs include the odor emission rates, stack parameters, building dimensions, site layout, surrounding topography, and meteorology. Proximity to property lines also is considered. Site plans, building configurations, and air flow estimates, developed during design, refine predictions of potential odor impacts and result in a more comprehensive evaluation of odor control systems. Finally, the height of the stacks for each air handling zone and/or odor control system is defined.

Source Control. The source of odor may be a raw product, process, and/or housekeeping. Each area should be examined as an area for odor control by replacement or modification of the operating scheme.

An example of raw product control includes the substitution for low-sulfur fuels to a boiler. In wastewater treatment, biological treatment processes may be expanded or high-rated to overcome odor-producing problems. Housekeeping solutions may be as simple as periodic cleaning of equipment and floor spaces.

Absorption. Water or other liquid solutions are used to scrub airstreams in many different applications. This absorption process requires that the odorous compound be soluble in the scrubbing solution. The scrubber systems (such as a packed bed) have the added benefit of removing other pollutants from the airstream. For example, absorption and chemical treatment may be designed to occur simultaneously in the same odor control scrubber unit; this is a popular approach when odor control is the only air quality control requirement.

Adsorption. Some solids will adsorb odorous compounds. Common treatment processes use activated carbon as the adsorbent. The efficiency is largely a function of temperature, pressure, and flow rate. For example, temperatures should not exceed 50°C (120°F). Once the adsorbent becomes loaded with the odorous compounds, it must be regenerated through chemical or physical means, or it must be disposed of and replaced with new adsorbent.

Other factors that affect the performance of an adsorption system include: the concentration of the odor, the characteristics of the adsorbent and its quality, the adsorber design, carbon particle size, bed depth, bed depth-to-diameter ratio, time to breakthrough, and regeneration frequency.

Biological Treatment. Some microorganisms will oxidize odorous compounds; however, performance data for direct biological odor control is lacking.

Chemical Treatment. Oxidizing agents and lime have application in source control to oxidize the odor-producing compound and increase its solubility, respectively. Under special controlled conditions, ozone is used by industry for gas-to-gas odor treatment. Other agents, such as chlorine, chlorine dioxide, and potassium permanganate, are successful odor treatments when added to a gas scrubber (absorption) solution.

Condensation. For odors associated with steam, contact condensers are effective control devices. The condensate should be discharged to a sanitary sewer or on-site treatment should be provided to prevent a water pollution problem.

Containment. Covers or other means of containment may be used to control odors. The emissions from the contained airstream generally provide a concentrated source of odors for treatment by other methods, such as scrubbers or incinerators.

Counteraction. When sufficient quantities of a counteractant are mixed with an odorous airstream such that no odor is distinguished, counteraction control is accomplished.

Dilution. Control of odors by simple dilution can occur with the addition of sufficient fresh air to reduce the odor concentration to below the threshold level. This method is not likely to be effective unless the quantity and intensity of odor compounds are very small. Tall stacks and heated gas streams have been successfully used by the utility industry for dilution of large volumes of gases by plume dispersion. The stack systems are useful for low concentrations of odor and can be a second-stage system following another control method.

Incineration. Combustion of odorous compounds is practiced using direct or catalytic incineration. The approach is especially applicable where there are small quantities of highly intensive odors and there are existing boilers or incinerators available.

Direct Incineration. Direct fume incineration is a traditional system that provides excellent removal of odors. The basis of this technology is to raise the temperature of the gas stream to 650 to 800°C (1200 to 1500°F) for a reaction time of less than one second.

The system includes a fuel feed system, open flame burners, combustion zone, and exhaust system. Natural gas and, to a lesser extent, propane, butane, or other fuels are used. It is important that complete combustion occur, since partially oxidized compounds may be created that are more odorous than the original source.

Catalytic Incineration. The catalytic incinerator operates in the 250 to 500°C (500 to 900°F) range with an associated smaller fuel demand. The system does not have good application when the gas stream is carrying metals, other solids, or liquids, or when the gas stream contains compounds, or their subsequent by-products, which will interact with the catalyst. Thus, gas and equipment characteristics must be examined closely before selection of a catalytic incineration system.

Masking. The masking of odors is accomplished by superimposing a less objectionable odor or fragrance, or by adding a compound to desensitize the nose. However, the objectionable odor may be more persistent and cause problems further from the source. Masking has its best application when the odor is of low intensity and of short or intermediate duration.

INDOOR AIR QUALITY

The control of indoor air quality is an area of increasing interest and concern. Early efforts were directed to the protection of health for industrial workers. The control measures are very source-specific, such as fume hoods on plating tanks and in chemical laboratories.

Indoor air contaminants individually may or may not pose a significant health risk, but many commercial and residential buildings have more than one source contributing to impact indoor air quality.^(123, 124) Sources that emit gases or particles into the air are the primary cause of indoor air quality problems in buildings. Inadequate ventilation can increase indoor pollutant levels by not bringing in enough outside air to dilute emissions from indoor sources and by not exhausting indoor air pollutants to the outside. High temperature and humidity levels also may increase concentrations of some pollutants. Table 4.35 provides an overview of the principal indoor air contaminants, their source, health effects, and typical control techniques.

The principal factors affecting the presence and concentration of indoor air contaminants are:

- Indoor sources (e.g., combustion sources and building materials)
- Outdoor sources
- Natural and mechanical indoor and outdoor air exchanges
- Dispersion within the building and its subdivisions
- Physical conditions indoors (e.g., humidity and temperature)
- Gas and/or particulate reactions dependently and independently in the indoor environment
- Building occupants (e.g., metabolism and smoking habits)

The three basic strategies for control of indoor air quality are source control, ventilation improvements and air cleaners. Generally, the cost of indoor air quality control is least for source control and most expensive for air cleaners.

Most often, the most effective way to improve indoor air quality is to eliminate individual pollution sources or to reduce their emissions. For example, sources that contain asbestos may be sealed or enclosed, and others sources, such as gas stoves, can be adjusted to decrease the emissions. Specific sources of indoor air pollution were listed in Table 4.35.

Increasing the amount of outdoor air coming indoors is another approach to lowering the concentrations of indoor air pollutants in buildings. For example, most home heating and cooling systems, including forced air heating systems, do not mechanically bring fresh air into the house. Control alternatives are to open windows and doors, operate window or attic fans, when the weather permits, or run a window air conditioner with the vent control open to increase the outdoor ventilation rate. Local bathroom or kitchen fans that exhaust outdoors remove contaminants directly from the room where the fan is located and thus, increase the outdoor air ventilation rate.

Source control is particularly important to control air pollutants during short-term activities that can generate high levels of pollutants, such as painting, paint stripping, heating with kerosene heaters, cooking, or engaging in maintenance and hobby activities, such as welding, soldering, or sanding. Weather permitting, these activities may be performed at more suitable facilities equipped with enhanced ventilation systems.

There are many types and sizes of air cleaners on the market, ranging from relatively inexpensive tabletop models to sophisticated and expensive whole-building systems. The effectiveness of an air cleaner is affected by the strength of the pollutant source. Some air cleaners are highly effective at particle removal, while others, including most tabletop models, are not. Air cleaners generally are designed to remove particulate pollutants.

The effectiveness of an air cleaner depends on how well it collects pollutants from indoor air and how much air it draws through the cleaning or filtering element. A very efficient collector with a low air-circulation rate will not be effective, nor will a cleaner with a high air-circulation rate but a less efficient collector. The long-term performance of any air cleaner requires routine and preventive maintenance.

TABLE 4.35 Indoor Air Pollutants Overview (123)

Sources	Health Effects	Control Techniques
<i>Asbestos</i>		
Deteriorating, damaged, or disturbed insulation, fireproofing, acoustical materials, and floor tiles.	No immediate symptoms, but long-term risk of chest and abdominal cancers and lung diseases. Smokers are at higher risk of developing asbestos-induced lung cancer.	Leave undamaged asbestos material alone if it is not likely to be disturbed. Use trained and qualified contractors for control measures that may disturb asbestos and for cleanup.
<i>Biologicals</i>		
Wet or moist walls, ceilings, carpets, and furniture; poorly maintained humidifiers, dehumidifiers, and air conditioners; bedding; household pets.	Eye, nose, and throat irritation; shortness of breath; dizziness; lethargy; fever; digestive problems. Can cause asthma, humidifier fever, influenza and other infectious diseases.	Use fans to vent kitchens and bathrooms outdoors. Vent clothes dryers to outdoors. Clean cool mist and ultrasonic humidifiers and refill with clean water daily. Empty water trays in air conditioners, dehumidifiers, and refrigerators frequently. Use basements as living areas only if they are leak proof and adequate ventilation. Use dehumidifiers, if necessary.
<i>Carbon Monoxide</i>		
Unvented kerosene and gas space heaters; leaking chimneys and furnaces; back-drafting from furnaces, gas water heaters, woodstoves, and fireplaces; gas stoves. Automobile exhaust from attached garages. Environmental tobacco smoke.	At low concentrations, fatigue in healthy people and chest pain in people with heart disease. At higher concentrations, impaired vision and coordination, headaches, dizziness; confusion, nausea. Can cause flu-like symptoms that clear up after leaving home. Fatal at very high concentrations.	Keep gas appliances properly adjusted. Consider a vented space heater to replace an unvented one. Use an exhaust fan over gas stoves, vented to outdoors. Open flues when fireplaces are in use. Have a trained professional inspect, clean, and tune up central heating system annually. Repair any leaks promptly. Do not idle the car inside garage.
<i>Environmental Tobacco Smoke (ETS)</i>		
Cigarette, pipe, and cigar smoking.	Eye, nose, and throat irritation; headaches; lung cancer; may contribute to heart disease. Specifically for children, increased risk of lower respiratory tract infections and ear infections; build-up of fluid in the middle ear; increased severity and frequency of asthma episodes; decreased lung function.	Do not smoke in the building or permit others to do so. Do not smoke if children are present, particularly infants and toddlers. If smoking indoors cannot be avoided, increase ventilation in the area where smoking takes place. Open windows or use exhaust fans.

(continues)

TABLE 4.35 Indoor Air Pollutants Overview (*continued*)

Sources	Health Effects	Control Techniques
<i>Formaldehyde</i>		
Pressed wood products and furniture made with these products. Urea-formaldehyde foam insulation (UFFI). Combustion sources and environmental tobacco smoke.	Eye, nose, and throat irritation; wheezing and coughing; fatigue; skin rash; severe allergic reactions. May cause cancer. May also cause other effects listed under "organic gases."	Use "exterior-grade" pressed wood products (lower-emitting because they contain phenol resins, not urea resins). Use air conditioning and dehumidifiers to maintain moderate temperature and reduce humidity levels. Increase ventilation, particularly after bringing new sources of formaldehyde into the home.
<i>Lead</i>		
Lead-based paint, contaminated soil, dust, and drinking water.	Lead affects practically all systems within the body. Lead at high levels (at or above 80 micrograms per deciliter (80 $\mu\text{g}/\text{dL}$) of blood) can cause convulsions, coma, and even death. Lower levels of lead can cause adverse health effects on the central nervous system, kidney, and blood cells. Blood lead levels as low as 10 $\mu\text{g}/\text{dL}$ can impair mental and physical development.	Keep areas where children play as dust-free and clean as possible. Leave lead-based paint undisturbed if it is in good condition; do not sand or burn off paint that may contain lead. Hire a qualified contractor to remove lead paint. Do not bring lead dust into the building. If your work or hobby involves lead, change clothes and doormats before entering your home.
<i>Nitrogen Dioxide</i>		
Kerosene heaters, unvented gas stoves and heaters. Environmental tobacco smoke.	Eye, nose, and throat irritation. May cause impaired lung function and increased respiratory infections in young children.	Keep gas appliances properly adjusted. Consider a vented space heater to replace an unvented one. Use an exhaust fan over gas stoves and vented to outdoors. Open flues when fireplaces are in use. Have a trained professional inspect, clean, and tune up central heating system annually. Repair any leaks promptly.
<i>Organic Gases</i>		
Household products including: paints, paint strippers, and other solvents; wood preservatives; aerosol sprays; cleansers and disinfectants; moth repellents; stored fuels and automotive products; hobby supplies; dry-cleaned clothing	Eye, nose, and throat irritation; headaches, loss of coordination, nausea; damage to liver, kidney, and central nervous system. Some organics can cause cancer in animals; some are suspected or known to cause cancer in humans.	Use household products according to manufacturer's directions. Provide plenty of fresh air when using these products. Throw away unused or little-used containers Keep out of reach of children and pets. Never mix household care products unless directed by the label.

TABLE 4.35 Indoor Air Pollutants Overview (*continued*)

Sources	Health Effects	Control Techniques
<i>Pesticides</i>		
Products used to kill household pests (insecticides, termiticides, and disinfectants).	Irritation to eye, nose, and throat; damage to central nervous system and kidney; increased risk of cancer.	Use strictly according to manufacturer's directions. Mix or dilute outdoors. Apply only in recommended quantities. Dispose of unwanted containers safely.
Products used on lawns and gardens that drift or are tracked inside the building.		Increase ventilation when using indoors. Use nonchemical methods of pest control where possible Do not store unneeded pesticides inside home. Store clothes with moth repellents in separately ventilated areas, if possible.
<i>Radon</i>		
Earth and rock beneath foundation; well water; building materials.	No immediate symptoms. Estimated to contribute to between 7,000 and 30,000 lung cancer deaths each year. Smokers are at higher risk of developing radon-induced lung cancer.	Test building for radon. Fix building if radon level is 4 picocuries per liter (pCi/L or higher). Radon levels less than 4 pCi/L still pose a risk, and in many cases may be reduced
<i>Respirable Particles</i>		
Fireplaces, woodstoves, and kerosene heaters.	Eye, nose, and throat irritation; respiratory infections and bronchitis; lung cancer.	Vent all furnaces to outdoors. Keep doors open when using unvented space heaters.
Environmental tobacco smoke.	Effects attributable to environmental tobacco smoke are listed above.	Have a trained professional inspect, clean, and tune up central heating system annually. Repair any leaks promptly. Change filters on central heating and cooling systems and cleaners according to manufacturer's directions.

The environmental engineer has an important role in the technical assessment and control of indoor air quality. Because of its widespread occurrence in residences and public interest, radon gas is discussed further.

Radon: Sources and Effects

Radon is a colorless, radioactive, inert gaseous element that occurs naturally from the radioactive decay of radium atoms found in soils and rocks throughout the world. (Radium and, thus, radon are decay products in the uranium series.) The radon decay process releases alpha radiation and produces the intermediate products polonium 218, lead 214, bismuth 214, and polonium 214. These products are referred to as radon decay products, radon progeny, or radon daughters.

Concerns about radon contamination come from studies and research linking the effects of radon exposure to lung cancer. An important health study by the National Academy of Science analyzed the effects of high exposures to radon and its daughters on underground miners (85). Numerous other studies have been, or are being, undertaken, especially with respect to residential exposure over time.

Soil gas is considered the most important contributor of radon, with decreasing levels of contribution from outdoor air, potable water, and building materials (86). Radon measurements may be taken using relatively simple and inexpensive devices that are commercially available and use charcoal canisters and alpha track-etch detector techniques. Public and private testing services are also available (87, 88). The information in Figure 4.54 is useful in the selection of measuring locations.

Levels of radon below 4 pCi/L are generally recognized as acceptable and require no action. As levels increase toward 200 pCi/L, additional measurements should be made and some mitigation is warranted. Higher levels indicate a significant enough problem that radon reduction techniques should be implemented.

Residential Radon Reduction Techniques

Radon levels in detached residential houses can be reduced by

1. Preventing the entry of radon gas
2. Ventilating the air containing radon gas

Key to Major Radon Entry Routes

Soil Gas

- A Cracks in concrete slab
- B Cracks between poured concrete (slab) and blocks
- C Pores and cracks in concrete blocks
- D Slab-footing joints
- E Exposed soil, as in sump
- F Weeping tile
- G Mortar joints
- H Loose-fitting pipes

Building Materials

- I Granite

Water

- J Water

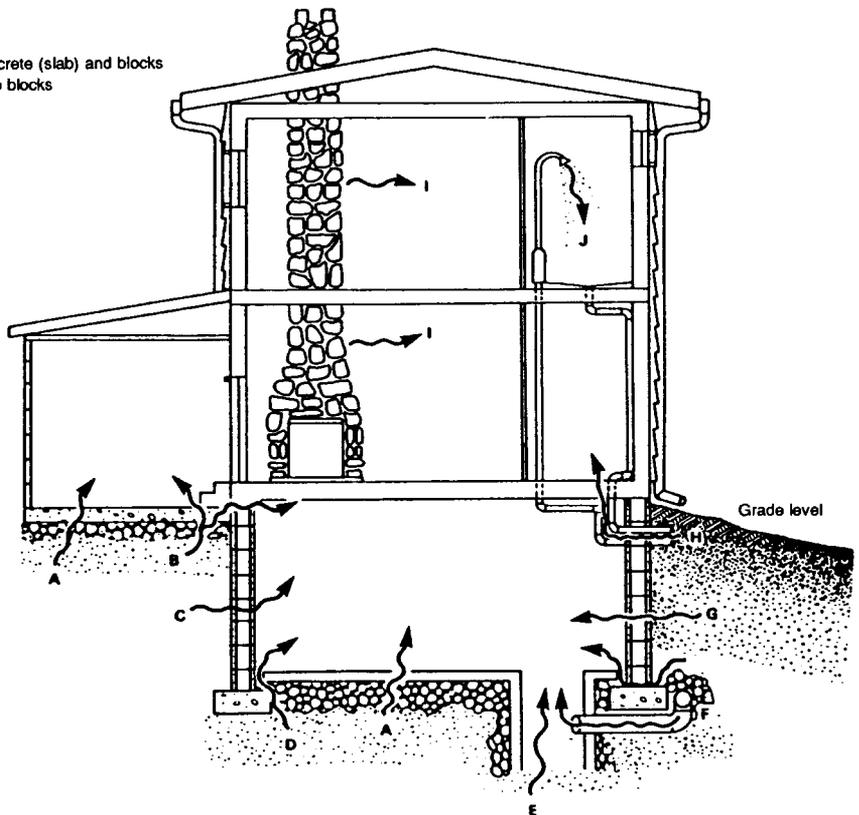


FIG. 4.54 Major radon entry routes into detached houses (86).

TABLE 4.36 Summary of Radon Reduction Techniques (86)

Method	Principle of operation	House types applicable	Estimated annual average concentration reduction, %	Confidence in effectiveness	Operating conditions and applicability	References
Natural ventilation	Air exchange causing replacement and dilution of indoor air with outdoor air by uniformly opening windows and vents	All ^a	90 ^b	Moderate	Open windows and air vents uniformly around house Air exchange rates up to 2 per hour may be attained. May require energy and comfort penalties and/or loss of living space use	89, 90
Forced-air ventilation	Air exchange causing replacement and dilution of indoor air with outdoor air by the use of fans located in windows or vent openings	All	90 ^b	Moderate	Continuous operation of a central fan with fresh air makeup, window fans, or local exhaust fans Forced-air ventilation can be used to increase air exchange rates up to 2 per hour May require energy and comfort penalties and/or loss of living space use	89–91
Forced-air ventilation with heat recovery	Air exchange causing replacement and dilution of indoor air with outdoor air by the use of a fan-powered ventilation system	All	96 ^c	Moderate to high	Continuous operation of units rated at 25–240 ft ³ /min Air exchange increased from 0.25 to 2 per hour In cold climates units can recover up to 70% of heat that would be lost through house ventilation without heat recovery	89–93
Active avoidance-of-house depressurization	Provide clean makeup air to household appliances that exhaust or consume indoor air	All	0–10 ^d	Moderate ^d	Provide outside makeup air to appliances such as furnaces, fireplaces, clothes dryers, and room exhaust fans	94
Sealing major radon sources	Use gas-proof barriers to close off and exhaust/ventilate sources of soil-gas-borne radon	All	Local exhaust of the source may produce significant housewide reductions	Extremely case specific	Areas of major soil-gas entry such as cold rooms, exposed earth, sumps, or basement drains may be sealed and ventilated by exhausting collected air to the outside	92, 94, 95
Sealing radon entry routes	Use gas-proof sealants to prevent soil-gas-borne radon entry	All	30–90	Extremely case specific	All noticeable interior cracks, cold joints, openings around services, and pores in basement walls and floors should be sealed with appropriate materials	92, 96

(continues)

TABLE 4.36 Summary of Radon Reduction Techniques (*continued*)

Method	Principle of operation	House types applicable	Estimated annual average concentration reduction, %	Confidence in effectiveness	Operating conditions and applicability	References
Drain tile soil ventilation	Continuously collect, dilute, and exhaust soil-gas-borne radon from the footing perimeter of houses	BB PCB S	Up to 98	Moderate ^f	Continuous collection of soil-gas-borne radon using a 160-ft ³ /min fan to exhaust a perimeter drain tile Applicable to houses with a complete perimeter footing level drain tile system and with no interior block walls resting on subslab footings	97
Active ventilation of hollow-block basement	Continually collect, dilute, and exhaust soil-gas-borne radon from hollow-block basement walls	BB	Up to 99 +	Moderate to high	Continuous collection of soil-gas-borne radon using one 250-ft ³ /min fan to exhaust all hollow-block perimeter basement walls Baseboard wall collection and exhaust system used in houses with French (channel) drains	97
Subslab soil ventilation	Continually collect and exhaust soil-gas-borne radon from the aggregate or soil under the concrete slab	BB PCB S	80–90, as high as 99 in some cases	Moderate to high	Continuous collection of soil-gas-borne radon using one fan (~100 ft ³ /min, ≥0.4 in; H ₂ O suction) to exhaust aggregate or soil under slab For individual suction point approach, roughly one suction point per 500 ft ² of slab area Piping network under slab is another approach, might permit adequate ventilation without power-driven fan	92, 97, 98

^aBB (block basement) houses with hollow-block (concrete block or cinder block) basement or partial basement, finished or unfinished; PCB (poured concrete basement) houses with full or partial, finished or unfinished poured-concrete walls; C (Crawl space) houses built on a crawl space; S (Slab, or slab-on-grade) houses built on concrete slabs.

^bField studies have validated the calculated effectiveness of fourfold to eightfold increases in air exchange rates to produce up to 90% reductions in indoor radon.

^cRecent radon mitigation studies of 10 inlet/outlet balanced mechanical ventilation systems have reported radon reduction up to 96% in basements. These studies indicate air exchange rates were increased from 0.25 to 1.3 per hour.

^dThis estimate assumes that depressurizing appliances (i.e., local exhaust fans, clothes dryers, furnaces, and fireplaces) are used no more than 20% of the time over a year. This suggests that during the heating season use of furnaces and fireplaces with provision of makeup air may reduce indoor radon levels by up to 50%.

^eStudies indicate that significant entry of soil-gas-borne radon is induced by pressure differences between the soil and indoor environment. Specific radon entry effects of specific pressurization and depressurization are also dependent on source strengths, soil conditions, the completeness of house sealing against radon, and baseline house ventilation rates.

^fOngoing studies indicate that where a house's drain tile collection system is complete (i.e., it goes around the whole house perimeter) and the house has no interior hollow-block walls resting on subslab footings, high radon entry reduction can be achieved.

3. Removing the source of radon
4. Removing radon and/or its decay products from indoor air

Practice focuses on use of the first two techniques; associated radon reduction methods are summarized in Table 4.36. Removing the source of radon creates a secondary problem in that the soil must be handled and treated as a low-level radioactive waste. Methods are not well developed for removal of radon by treatment of indoor air.

Preventing Radon Entry. The principal methods for preventing radon entry into a house are sealing all points of entry for soil gas (Figure 4.54); reversing airflow from soil to house by raising house air pressure relative to that of the soil; replacing or treating radon-contaminated potable water; and replacing or avoiding radon-producing building materials.

Ventilating for Radon Reduction. Once radon has entered the house, ventilation is used to replace indoor air with outside air (of low radon contamination) and to dilute radon levels in air not replaced. Figure 4.55 shows the effect of increasing ventilation rates from 0.2 to 2 air changes per hour. Principal characteristics of this method are that the effectiveness of ventilation decreases with increased ventilation rates and that the radon source strength and entry rate establishes a finite level where higher ventilation rates are not productive.

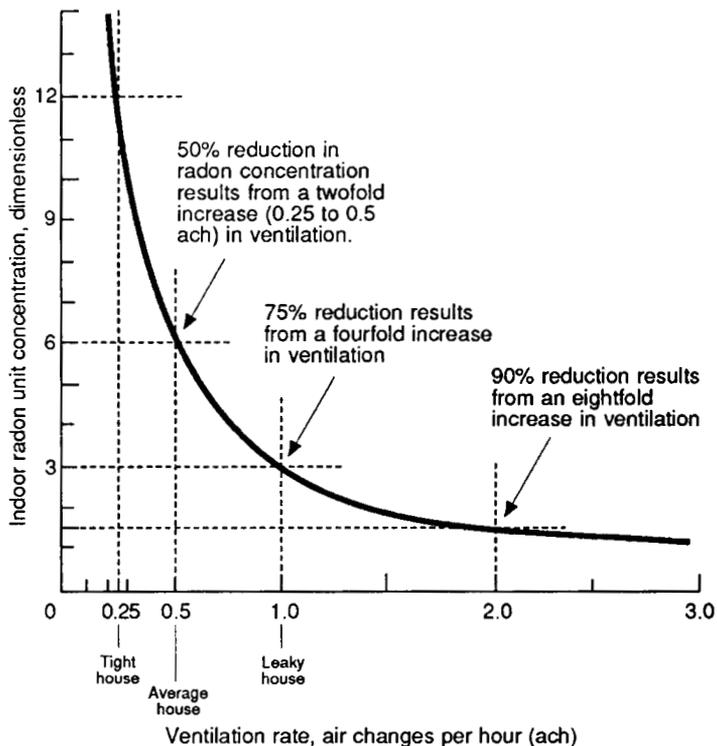


FIG. 4.55 Effect of ventilation on indoor radon concentrations (86).

NOISE POLLUTION

Noise is recognized as a form of pollution because it is a public health hazard causing hearing impairments and a nuisance causing psychological stress. These problems are controllable by the use of source and/or path controls and of protective equipment. Table 4.37 lists and defines some of the key terms and units used in noise control (99–104). This section focuses on noise from stationary sources.

Noise: Sources and Effects

Initially, the industrial workplace was the focus of noise investigative and control measures. Later, increased attention was given to community noises and to noises in the home.

The time distribution and variation of noise level are important in the evaluation of source-and-effect relationships. For example, random or sudden noise is more noticeable and unsettling than constant sound levels. Likewise, unusual sounds, cycles of periodic sounds, and background noise levels need to be related in the source-and-effect analysis.

Sources of Noise. Noise or sound comes from a universe of sources in nature as well as industry and community life. The major sources of community noise are transportation- (air, road, and rail) and construction-related. Table 4.38 compares average sound levels with common sources of noise.

Sound levels vary with the individual characteristics of the source and with the listener's distance from the source. Ranges for common sounds and industrial plant sounds are illustrated in Figures 4.56 and 4.57, respectively.

Urban versus suburban environments and factory versus office workplaces are simply two of the factors influencing the quantity and duration of noise exposure. Figure 4.58 depicts generalized noise exposure patterns for several lifestyles.

Effects of Noise. Historically, real and potential hearing damage were the most serious effects of noise. Efforts by the manufacturing and construction industries and by regulatory actions have lessened the likeli-

TABLE 4.37 Noise Control Terms and Units

Decibel, dB	A unit of sound pressure equal to 20 times the logarithm (base 10) of the magnitude of a sound pressure p to the reference sound pressure p_r . The reference pressures in air and water are 0.00002 and 0.1 N/m ² , respectively. The unit of dBA is the measured A-scale sound level. Since most sound-measuring instruments measure sound pressure, decibel usually refers to sound pressure level.
Frequency	The number of periods (cycles) repeated over a unit of time and is expressed as cycles per sec (cps) or Hertz (Hz).
Intensity	The average rate of sound energy transmitted through a unit of area in the direction specified and is expressed in watts per square meter, W/m ² .
Loudness	A subjective attribute and is expressed qualitatively, such as quiet or loud. Quantitative systems for loudness levels use phons or sones.
Noise	An undesired or unwanted audible sensation, sound, or an intermittent oscillation or vibration.
Phons	The average sound pressure level, dB, relative to a pure 1000 cps, or 1000 Hz, and is expressed on an arithmetic scale.
Sones	The loudness of a 1000-cps, or 1000-Hz, sound at 40-dB intensity above a listener's threshold; is expressed on a logarithmic scale.

TABLE 4.38 Noise Levels from Common Sources

Sound Level, dB	Source
20	Soft whisper, wind in tree
30	Concert hall, recording studios
40	Nearby conversation, residential neighborhood at night, hospitals
50	Business office, restaurant
60	Secretarial area, normal conversation
70	Along average street, transformers
80	Along busy street, pumps, steam valves
90	Cooling towers, hammermill, home shop tools
100	High-speed subway train, newspaper press room, air hammer
115 and above	Shaker on railroad car, pneumatic riveter, turbines

hood for hearing damage occurrences. Thus, the secondary impact of nuisance is the most common concern for noise control.

The nuisance effects of noise include interference with speech, physiologically unsettling environment at home and work, and more specific problems such as disruption of sleep. The extent of these effects varies, sometimes significantly, between individuals and as a factor of the noise source.

The noise characteristics that affect the listener's response include the overall loudness, sound pressure level, duration of exposure, time distribution of occurrence, and sound frequency. Other factors are the listener's total exposure (e.g., work life), age, and individual susceptibility.

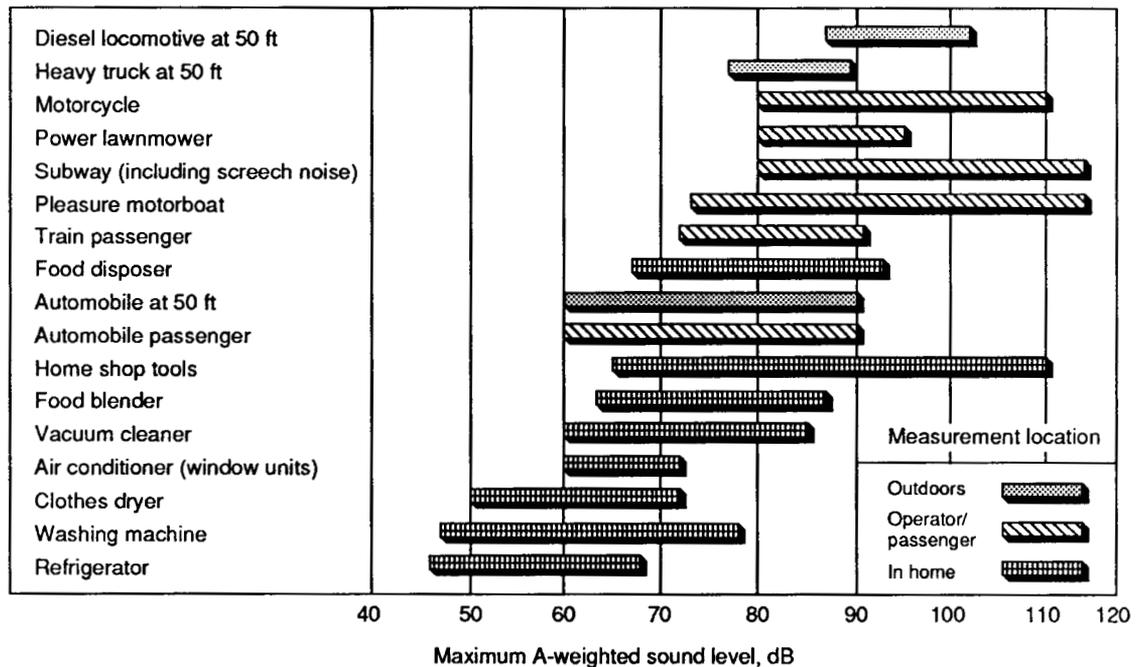


FIG. 4.56 Range of community noise levels (103).

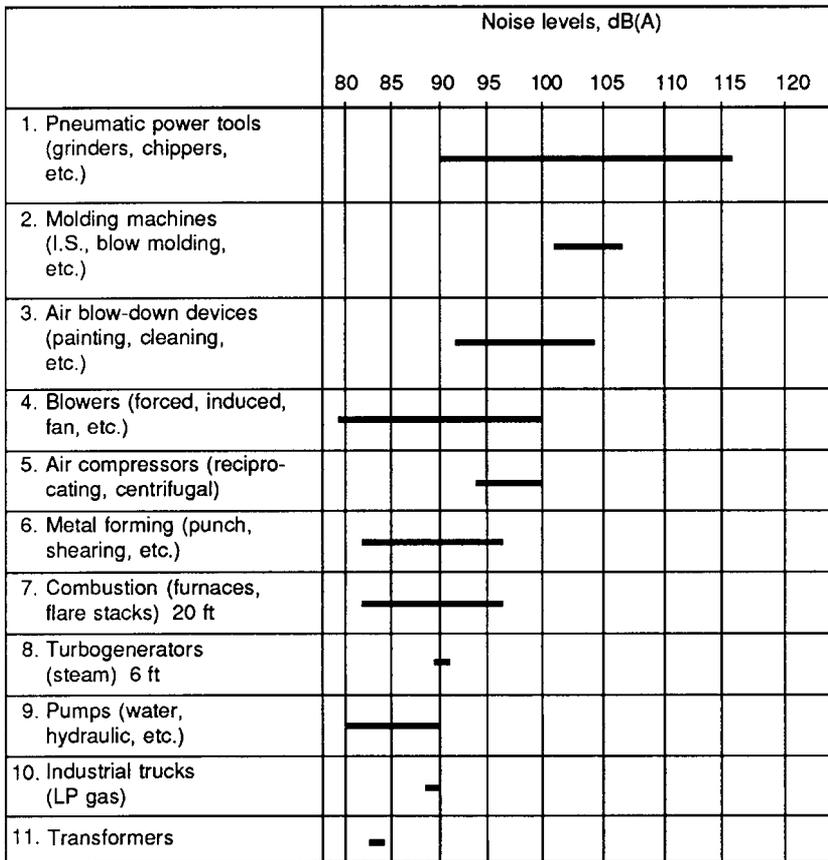


FIG. 4.57 Range of industrial plant noise levels at operator's position (103).

Interpreting available scientific information, the EPA identified a range of yearly day-night sound levels sufficient to protect public health and welfare from the effects of noise. These levels are presented in Table 4.39 and Figure 4.59. They do not represent standards but are viewed as levels below which there is no reason to suspect that the general population will be at risk from any of the identified effects.

Characterization

In order to identify strategies for noise control, the environmental engineer must characterize the source and impacts of the troublesome noise. The logical first step is a noise survey, which provides a data base for further definition of sound characteristics.

Noise Survey. Using a sound-level meter, a noise survey has one or more of the following objectives:

1. Determine if noise levels comply with applicable regulations.
2. Determine if noise levels are so high as to threaten hearing loss or to inhibit employee performance.
3. Develop a data base for a noise-reduction plan.

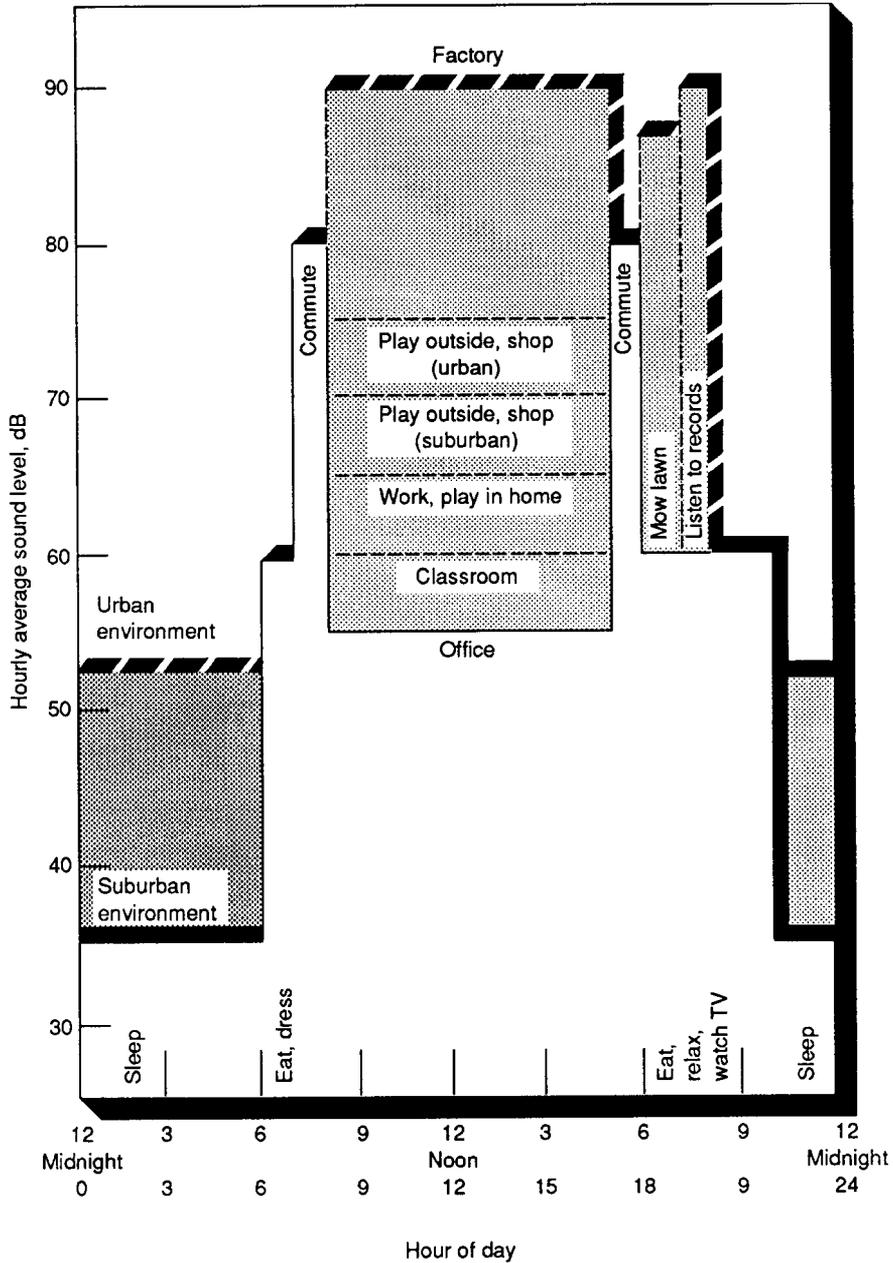


FIG. 4.58 Generalized individual noise exposure patterns (103).

TABLE 4.39 Yearly Average Equivalent Sound Levels to Protect Public Health and Welfare (99)

	Measure	Indoor			Outdoor		
		Activity interference	Hearing loss consideration	To protect against both effects ^b	Activity interference	Hearing loss consideration	To protect against both effects ^b
Residential with outside space and farm residences	L_{dn} $L_{eq(24)}$	45	70	45	55	70	55
Residential with no outside space	L_{dn} $L_{eq(24)}$	45	70	45			
Commercial	$L_{eq(24)}$	<i>a</i>	70	70 ^c	<i>a</i>	70	70 ^c
Inside transportation	$L_{eq(24)}$	<i>a</i>	70	<i>a</i>			
Industrial	$L_{eq(24)}$ ^d	<i>a</i>	70	70 ^c	<i>a</i>	70	70 ^c
Hospitals	L_{dn} $L_{eq(24)}$	45	70	45	55	70	55
Educational	$L_{eq(24)}$ $L_{eq(24)}$ ^d	45	70	45	55	70	55
Recreational areas	$L_{eq(24)}$		70	70 ^c	<i>a</i>	70	70 ^c
Farm land and general unpopulated land	$L_{eq(24)}$				<i>a</i>	70	70 ^c

^aSince different types of activities appear to be associated with different levels, identification of a maximum level for activity interference may be difficult except in those circumstances where speech communication is a critical activity. (See Figure 4.59 for noise levels as a function of distance which allow satisfactory communication.)

^bBased on lowest level.

^cBased only on hearing loss.

^dAn $L_{eq(s)}$ of 75 dB may be identified in these situations so long as the exposure over the remaining 16 h per day is low enough to result in a negligible contribution to the 24-h average, i.e., no greater than an L_{eq} of 60 dB.

Note: Explanation of identified level for hearing loss: The exposure period which results in hearing loss at the identified level is a period of 40 years.

The planning and conduct of the survey should consider the capability and calibration of the measuring equipment, interview data from users of the survey area, time variations as well as “normal” operations, and other techniques typical to a compliance and/or base data survey.

Sound Characteristics. In the assessment of noise, important characteristics include sound frequency and assorted sound levels. Each provides insight into the identification and development of abatement alternatives.

Frequency. The frequency of a sound is a function of the sound’s wavelength and the transmitting medium and is expressed as:

$$f = \frac{c}{x}$$

where f = frequency, cps or Hz
 c = velocity of sound, m/s
 x = wavelength, m

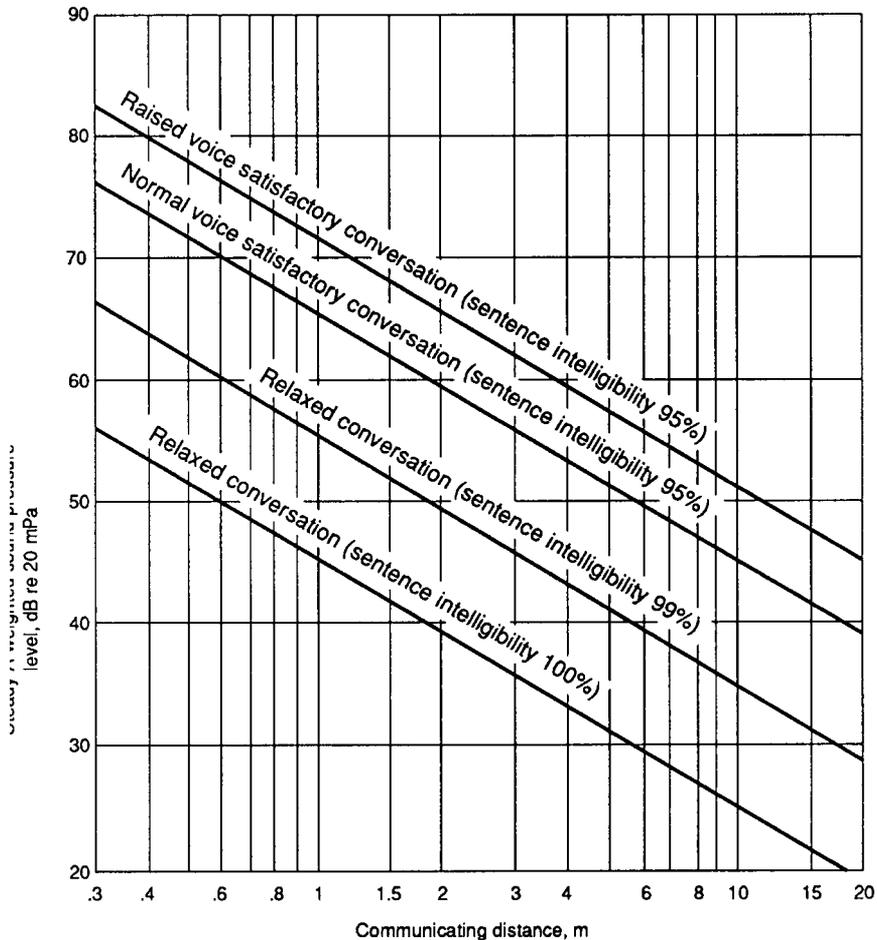


FIG. 4.59 Distances outdoors over which conversation is considered intelligible (99).

Furthermore, sound frequency has three zones:

1. Infrasonic: $f < 20$ cps
2. Audible: $20 \text{ cps} \leq f \leq 20,000$ cps
3. Ultrasonic: $f > 20,000$ cps

In addition, there are three categories of noise: wide-band, narrow-band, and impulse. A wide-band noise is one that spans a wide range of frequencies, such as those in a manufacturing operation. Power tools are examples of a noise over a narrow range of frequencies. Impulse noise is a temporary sound that may or may not be repetitive.

A frequency analysis is used to divide the noise into those three bands or into more narrow ranges and to determine relatively how much sound energy is in each range. Approaches to noise control design are directly related to frequency. A system for reducing high-frequency noise may have very little or no impact on low-frequency sound. The converse is true.

Sound Levels. For any noise or combination of noises, characterization includes the variation over time and the intensity, power, and/or pressure levels of the sound(s).

The time-related variations (distribution) in sound levels may be relatively constant, as with a waterfall; periodic (or cyclic), as with the cycling of an HVAC unit; or random, as with neighborhood activities. These variations are illustrated in Figure 4.60. Understanding variations in the level and occurrence of sound are important in assessing noise-reduction strategies (102).

In the assessment of noise, sound intensity, power, and pressure levels are expressed in decibels and are equal to a factor times the logarithm (base 10) of one quantity to a reference quantity. A summary of these background relationships follows.

Sound intensity I is expressed in watts per square meter by

$$I = \frac{W}{A}$$

where W = sound power, W

A = area of propagation, m²

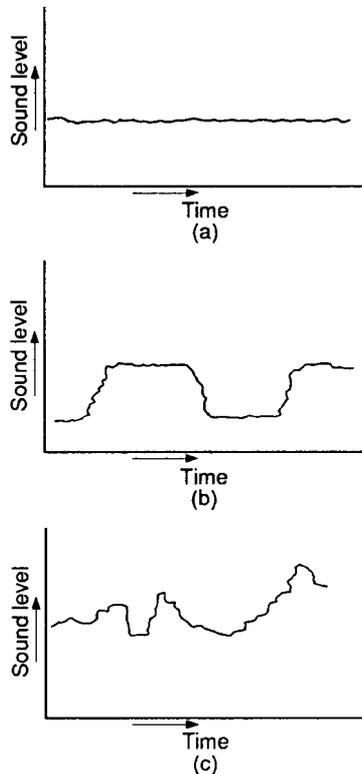


FIG. 4.60 Types of sound level variations. (a) Constant sound; (b) periodic sound; (c) random sound.

For a spherical wavefront, the surface area is $4\pi r^2$, and for a distance of r expressed in meters, the intensity equation becomes

$$I = \frac{W}{4\pi r^2}$$

Furthermore, sound intensity and sound pressure are related by

$$I = \frac{p^2}{\rho c}$$

where p = root mean square sound pressure, Pa

ρ = air density, kg/m³

c = sound propagation speed, m/s

The sound intensity level is defined by

$$L_I = 10 \log \frac{I}{I_0}$$

where L_I = sound intensity level, dB

I = sound intensity, W/m²

I_0 = reference sound intensity of 10^{-12} W/m²

The sound power level is defined by

TABLE 4.40 Applications of Noise Control Techniques

Noise control techniques	Noise source/environment		
	Community	Equipment	Transportation
<i>Isolation</i>			
Replacement	—	X	X
Air sources	X	X	X
Vibration	—	X	X
Damping	X	X	X
Enclosure	X	X	—
Valving	X	X	—
Location	X	X	X
<i>Suppression</i>			
Hearing protection	—	X	—
Conduits	X	X	—
Mufflers	—	X	X
Silencers	—	X	—
<i>Shielding</i>			
Absorption	X	X	—
Barriers	—	—	X

TABLE 4.41 Permissible Occupational Noise Exposures (106)

Sound level, dBA	Duration, h/day
90	8
92	6
95	4
97	3
100	2
102	1.5
105	1
110	0.5
115	≤0.25

$$L_w = 10 \log \frac{W}{W_0}$$

where L_w = sound power level, dB

W = sound power, W

W_0 = reference sound power of 10^{-12} W

The sound pressure level is defined by

$$L_p = 10 \log \frac{P^2}{P_0^2}$$

$$= 20 \log \frac{P}{P_0}$$

TABLE 4.42 Design Noise Level/Land Use Activities (107)

Activity category	Design noise level,* dBA		Description of land use activity category
	$L_{eg}(h)$	$L_{10}(h)$	
A	57†	60†	Land where serenity and quiet are of extraordinary significance, such as amphitheaters
B	67†	70†	Picnic areas, recreation areas, and the like that are not included in category A, and residences, hotels, public meeting rooms, churches, libraries, hospitals, and similar facilities
C	72†	75†	Developed lands with activities not included in categories A and B
D	—‡	—‡	Undeveloped lands
E	52§	55§	Residences, hotels, public meeting rooms, churches, libraries, hospitals, and similar facilities

*Either L_{10} or L_{eg} (but not both) may be used on a project

†Exterior

‡No Limit

§Interior

where L_p = sound pressure level, dB
 P = sound pressure, N/m²
 P_0 = reference sound pressure of 2×10^{-5} N/m²

Most sound-measuring instruments measure the quantity of sound pressure, i.e., sound pressure level.

Noise Control

The control of noise is achieved using one or more of a variety of measures. In particular, individual control measures are discussed under the categories of isolation, suppression, and shielding. Typical applications of these techniques are presented in Table 4.40 with respect to community (e.g., home and office), equipment (e.g., construction and manufacturing), and transportation (e.g., air, rail, and road) sources. The level of noise control required is generally dictated by local or national laws.

For example, under authority of the Occupational Safety and Health Act (OSHA), maximum occupational exposures to noise have been established. These sound levels are presented in Table 4.41, based on measurement on the A scale at a slow response using a standard sound meter. The act contains other provisions for intermittent noise and general protection of workers (106).

In another example, the U.S. Department of Transportation has regulations on design noise levels for several land use activities. The levels in Table 4.42 represent a balancing of what may be desirable and what may be achieved. The stated values are set as maximum values with the recognition that lower noise levels would result in even greater benefits to the community (107).

Isolation. Noise reduction at its source is the basis of isolation control measures.

Replacement. Often, significant noise reduction at the source is achieved by replacement of deteriorating or simply outdated equipment and engines with newer models that perform more quietly. In many cases, current models were redesigned to incorporate noise-reduction features, which may be changes in the physical components and/or in the operation mode.

In addition to customer requests, manufacturers have been under legislative and regulatory pressure to develop quieter equipment. One example is the role of the U.S. Federal Aviation Administration in reducing aircraft noise by establishing limits on aircraft engine noise.

Air Sources. The movement of large volumes of air and use of high-pressure air are two source control issues. The noise from the former may be lessened by using a lined bellows between the air source (outside) and the air-handling equipment (inside).

On the other hand, control of high-pressure air noises requires design modifications, such as redesigned nozzles or orifices, and/or operational changes, such as an increase in air volume with a corresponding reduction in velocity.

Vibration. Disturbing noises from production machines are the result of structural transmission noise and vibration. Control measures include isolation of the source by use of elastic discontinuities at joints and/or machine mounts. Isolation means and materials include springs, shock absorbers, cork, rubber, and plastics.

Damping. Some structural source noises and vibrations are a result of materials used in construction. In these cases, substitution of construction materials is a control. When noise levels are not highly bothersome, special damping tapes or spray-on coatings are successful control measures.

Enclosure. When vibration isolation and damping techniques are not sufficient controls, the source may be enclosed in a separately prepared enclosure. The simplest approach is to use a hinged cover especially prepared with sound-insulating material; an example is the common use of covers on high-speed printers in an office area.

Valving. Liquid flow, especially from cavitating conditions, and air- or steamflow, downstream of a significant pressure drop, are sources of noise that can be reduced by valving. Special valves and operating

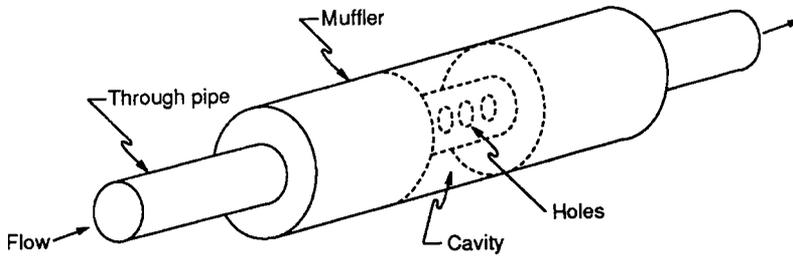


FIG. 4.61 Muffler component isometric.

techniques are used for liquid flow situations. An effective measure for compressible fluids (air) is to use two or more valves in the pressure-reduction step and, thus, reduce turbulence, which in turn reduces noise. In addition, localized insulation wrapping of downstream conduit provides added noise control.

Location. Space planning in the home, office, and manufacturing plant offers an opportunity to locate or relocate a noise source to an isolated site within the occupied structure. Normally, remote locating of noise sources is considered in design of new facilities. Thus, location controls tend to be most appropriate to additions of new processes and/or equipment as part of an industrial plant modification. Assessment of noise potential before installation is a major factor in the cost-effectiveness of this control measure.

In the transportation area, locating is better described as routing. Land travel controls almost always apply to new road and rail construction. However, in the air, aircraft patterns may be adjusted to meet special conditions, such as the routing of supersonic aircraft away from populated areas.

Suppression. For noise control near the source, suppression techniques are used. Most apply filtering to noise transmitted through the air.

Hearing Protection. Especially for construction and industrial workers with high on-the-job exposures to noise, earplugs and earmuffs are effective in noise suppression for hearing protection. The disadvantages of this approach, especially for extended periods, is the physical and/or psychological discomfort of the chosen device. Continued improvements for user comfort are being developed.

Earmuffs tend to be somewhat more effective than earplugs. The selection issue is mostly one of whether or not this approach to noise control is sufficient to reduce worker exposure to a time-weighted average limit established for the application.

Also, the frequency of the noise should be evaluated before selection of the hearing protector. Some earmuffs and earplugs provide best results at higher or lower frequencies.

Conduits. High-pressure transmission lines for liquids and gases as well as solid conveyor tunnels are conduits—transmitters of sources of noise. Control measures include internal liners or (multilayer) coverings. Sheet lead, plastics, glass, fiber, and rock wool are some of the materials used in these applications. In the case of HVAC ductwork, the effects of thermal and noise insulation are optimized by the covering technique.

Mufflers. A very effective approach for middle- and high-frequency noise control is the muffler. The unit is designed to attenuate sound waves with minimal back pressure. The results are achieved using baffles and reduced velocities.

For example, a typical muffler includes a cylinder-type unit wherein the outer portion from a through-pipe conduit is a number of side cavities where the noise suppression occurs. Figure 4.61 illustrates this concept. In some cases, a porous packing is used to increase efficiency. Airflow to the cavities is regulated by the size and number of holes from the center section.

Silencers. Depending on the required noise reduction, operating conditions, and noise frequency, silencers are used on equipment exhaust systems. Silencers, like mufflers, are commercially available in a variety of shapes and sizes and are typically of metal construction.

Two types of silencers are dissipative and reactive. Dissipative silencers are good for high-frequency sounds and rely on sound-absorbing liners. For low-frequency application, reactive silencers are used. They function by reflecting sound energy back toward the noise source.

Shielding. The third category of noise control is shielding and is the approach most distanced from the noise source.

Absorption. Indoors sound levels tend to accumulate; however, use of absorptive materials on ceilings and walls will reduce the area's overall noise level and lessen the additive effect of background noise on a machine operator. In a commercial and residential setting, floor coverings further reduce overall noise levels. Furthermore, in an office setting, fixed and movable partitions reduce noise levels, and the developments in modular furniture are very cognizant of needs to control noise.

Barriers. Sources freely radiate noise in the outdoors; this is the scenario for highway (and railway) noise conditions. Buildings, hills, and other terrain features function as noise barriers. Engineered barriers include walls and/or earthen berms. Noise control combines with visual screening to use barriers as a control measure for new and expanding major transportation routes through populated areas. The designer should consult the governing authority (and suppliers) for applicable design and construction requirements (108–110).

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CHAPTER 5

WATER SUPPLY

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Historically, the environmental engineer has found ways and means to provide ample quantities of quality drinking water for domestic use as well as quality water for commercial and industrial uses. Water supply issues include demand projections, quality requirements, surface water and groundwater source evaluations, groundwater production, surface water collection, surface water treatment, saline water treatment, nonconventional water production, and treated water distribution.

The typical surface water treatment plant uses chemicals to enhance removal of suspended solids and for disinfection. Physical treatment processes include simple settling and filtration. In sequence, the traditional unit processes are rapid mix, coagulation, flocculation, sedimentation, filtration, and disinfection. Auxiliary systems are needed for chemical feed facilities and for sludge handling. More specialized processes include carbon adsorption, ion exchange, and softening.

Treatment of other sources generally requires site-specific determination of raw water quality. Often, the quantity of demand will influence the cost-effective selection of treatment processes. Groundwater supplies require well development and treatment, such as aeration, softening, and/or disinfection. Brackish and saline waters also require site-specific determination of treatment processes. Typical processes include membrane technology, such as reverse osmosis and electrodialysis.

The water distribution system includes (small) service and distribution lines, (large) transmission mains, and storage facilities. Elevated storage tanks with gravity distribution and/or ground storage tanks with distribution pumping are designed to provide the quantity and pressure required to satisfy system demands. Generally, fire demands control design.

Since the early 1900's, the American Water Works Association (AWWA) has been developing standards for the drinking water industry. The standards also are designed to enhance water utility operations while addressing hundreds of products and procedures. A discussion and listing of AWWA standards is included at the end of this chapter.

WATER DEMAND

The nation's water resources have been extensively developed to satisfy a variety of beneficial uses. Water projects generally support a dominant single purpose, such as urban water supply, irrigation, flood control, or navigation.

Water withdrawn for offstream use consists of two variables: (1) the part returned to the surface water or groundwater source after being used, and (2) the part consumed but not returned to the source after use. A

typical example of consumption is vegetation transpiration. Much of the water used for irrigation, particularly sprinkler irrigation, either is transpired by the plants or is evaporated from the soil. Withdrawals may include saline water, but the major concern of this discussion is with freshwater withdrawals and consumption for each category of use.

Table 5.1 summarizes the actual use in 1975 and the projected uses for 2000 for each of the major consumption categories. As can be seen, the largest withdrawal use is for irrigation, and the next-largest use cooling water for steam electric generation (*I*).

In order to properly evaluate a water system, it is useful to be able to estimate demands for specific uses. Table 5.2 summarizes demand from a number of specific domestic, commercial, industrial, and agricultural activities (*2*). Each of these activities is discussed separately in the following sections.

Domestic Use

Average daily domestic (central) withdrawal in 1975 was 118 gal (0.43 m³) per capita per day; a typical family of four used about 87 gal (0.33 m³) per capita per day for drinking, culinary, washing, and sanitary needs (*I*). The 31 gal (0.12 m³) per capita per day difference is accounted for by delivery losses, fire protection, street washing, city and park maintenance, and other similar items. Most central systems have the capacity to deliver 150 gal (0.57 m³) per capita per day, but this amount includes some water for manufacturing, which is accounted for in manufacturing water use. In a typical rural household (noncentral) domestic use averages about 44 gal (0.25 m³) inside.

Recent studies have refined the estimates for indoor water use. One national study developed estimates of indoor residential water use where individual water using appliances were metered (*122*). In more recent studies, residential customer water meters were fitted with portable data loggers measured for analysis of indoor water use (*123–125*). Estimated percentages of indoor residential water use by fixture

TABLE 5.1 Water Withdrawals and Consumption, by Functional Use (*I*)

Functional use	Total withdrawals, 10 ⁶ gal/day			Total consumption, 10 ⁶ gal/day		
	1975	1985	2000	1975	1985	2000
Fresh water:						
Domestic:						
Central (municipal)	21,164	23,983	27,918	4,976	5,665	6,638
Noncentral (rural)	2,092	2,320	2,400	1,292	1,408	1,436
Commercial	5,530	6,048	6,732	1,109	1,216	1,369
Manufacturing	51,222	23,687	19,669	6,059	8,903	14,699
Agriculture:						
Irrigation	158,743	166,252	153,846	86,391	92,820	92,506
Livestock	1,912	2,233	2,551	1,912	2,233	2,551
Steam electric generation	88,916	94,858	79,492	1,419	4,062	10,541
Minerals industry	7,055	8,832	11,328	2,196	2,777	3,609
Public lands and others*	1,866	2,162	2,461	1,236	1,462	1,731
Total fresh water	338,500	330,375	306,397	106,590	120,545	135,080
Saline water, [†] total	59,737	91,236	118,815			
Total withdrawals	398,237	421,611	425,212			

*Includes water for fish hatcheries and miscellaneous uses.

[†]Saline water is used mainly in manufacturing and steam electric generation.

Note: To convert from 106 gallons per day to 90 cubic meters per day, multiply by 3.785.

TABLE 5.2 Typical Water Demand Levels (2)

Type of establishment	Gal/day (m ³ /day)
Airports (per passenger)	3.5 (0.013)
Apartments, multiple-family (per resident)	60 (0.23)
Bathhouses (per bather)	10 (0.04)
Camps:	
Construction, semipermanent (per worker)	50 (0.19)
Day with no meals served (per camper)	15 (0.06)
Luxury (per camper)	100–150 (0.38–0.57)
Resorts, day and night, with limited plumbing (per camper)	50 (0.19)
Tourist with central bath and toilet facilities (per person)	35 (0.13)
Cottages with seasonal occupancy (per resident)	50 (0.19)
Courts, tourist with individual bath units (per person)	50 (0.19)
Clubs:	
Country (per resident member)	50 (0.19)
Country (per nonresident member present)	25 (0.10)
Dwellings:	
Boarding houses (per boarder)	50 (0.19)
Additional kitchen requirements for nonresident boarders	50 (0.19)
Luxury (per person)	10 (0.038)
Multiple-family apartments (per resident)	50–75 (0.19–0.28)
Rooming houses (per resident)	60 (0.23)
Single family (per resident)	50–75 (0.19–0.28)
Estates	100–150 (0.38–0.57)
Factories (gallons per person per shift)	15–35 (0.06–0.13)
Highway rest area (per person)	5 (0.02)
Hotels with private baths (2 persons per room)	60 (0.23)
Hotels with private baths (per person)	50 (0.19)
Institutions other than hospitals (per person)	75–125 (0.28–0.47)
Hospitals (per bed)	250–400 (0.95–1.51)
Laundries, self-service (gallons per washing, i.e., per customer)	50 (0.19)
Livestock (per animal):	
Cattle (drinking)	12 (0.04)
Dairy (drinking and servicing)	35 (0.13)
Goat (drinking)	2 (0.01)
Hog (drinking)	4 (0.02)
Horse (drinking)	12 (0.04)
Mule (drinking)	12 (0.04)
Sheep (drinking)	2 (0.01)
Steer (drinking)	12 (0.04)
Motels with bath, toilet, and kitchen facilities (per bed space)	50 (0.19)
With bed and toilet (per bed space)	40 (0.15)
Parks:	
Overnight with flush toilets (per camper)	25 (0.10)
Trailers with individual bath units, no sewer connection (per trailer)	25 (0.10)
Trailers with individual baths, connected to sewer (per person)	50 (0.19)
Picnic:	
With bathhouses, showers, and flush toilets (per picnicker)	20 (0.08)
With toilet facilities only (gallons per picnicker)	10 (0.04)
Poultry:	
Chicken (per 100)	5–10 (0.02–0.048)
Turkeys (per 100)	10–18 (0.04–0.07)

(continues)

TABLE 5.2 Typical Water Demand Levels (*continued*)

Type of establishments	Gal/day (m ³ /day)
Restaurants with toilet facilities (per patron)	7–10 (0.03–0.04)
Without toilet facilities (per patron)	2 (0.01–0.0)
With bars and cocktail lounge (additional quantity per patron)	2 (0.01)
Schools:	
Boarding (per pupil)	75–100 (0.28–0.38)
Day with cafeteria, gymnasiums, and showers (per pupil)	25 (0.01)
Day with cafeteria but no gymnasiums or showers (per pupil)	20 (0.08)
Day without cafeteria, gymnasiums, or showers (per pupil)	15 (0.06)
Service stations (per vehicle)	10 (0.04)
Stores (per toilet room)	400 (1.51)
Swimming pools (per swimmer)	10 (0.04)
Theaters:	
Drive-in (per car space)	5 (0.02)
Movie (per auditorium seat)	5 (0.02)
Workers:	
Construction (per person per shift)	50 (0.19)
Day (school or offices per person per shift)	15 (0.06)

are shown in Table 5.3 (*126*). Based on a 1987 nationwide study, estimated indoor residential water use was 60 gal/capita/day (22.8 m³/capita/day), and a 1997 study reported an average of 60.45 gal/capita/day (23 m³/capita/day) (*125, 127*).

The following sections discuss variation in peak demand and special uses, such as lawn sprinkling and fire protection.

Peak Demands. The rate of water use for an individual water system will vary directly with domestic activity in the home or with an operational farm program. Usage rates are generally highest in the home near mealtimes, during midmorning laundry periods, and shortly before bedtime. During the intervening daytime hours and at night, water use may be virtually nil. Thus, the total amount of water used by a household may be distributed over only a few hours of the day, during which the actual use is much greater than the average rate determined from Table 5.2.

TABLE 5.3 Estimated Percent of Indoor Residential Water Use by Fixture

Source	1984 Study (<i>122</i>)	1997 Study A (<i>123</i>)	1997 Study B (<i>125</i>)
Fixture Type			
Bath	12	2	2
Clothes washer	22	24	24
Dishwasher	2	3	2
Faucet	14	15	18
Shower	14	17	21
Toilet	23	26	30
Leak	13	12	4
Total	100	100	100

Simultaneous operation of several plumbing fixtures will determine the maximum peak rate of water delivery for the home water system. For example, a shower, an automatic dishwasher, a lawn sprinkler, and a flush valve toilet all operated at the same time would probably produce a near-critical peak. It is true that not all of these facilities are usually operated together; but if they exist on the same system, there is always a possibility that a critical combination may result, and for design purposes, this method of calculation is sound. Table 5.4 summarizes the rate of flow that would be expected for certain household and farm fixtures (2).

Lawn Sprinkling. The amount of water required for lawn sprinkling depends upon the size of the lawn, type of sprinkling equipment, climate, soil, and water control. In dry or arid areas, the amount of water may equal or exceed the total used for domestic or farmstead needs. For estimating purposes, a rate of approximately $\frac{1}{2}$ in/h (1.27 cm/h) of surface area is reasonable. This amount of water can be applied by sprinkling 30 gal (0.11 m³) of water per hour over each 100 ft² (9.29 m²).

When possible, the water system should have a minimum capacity of 500 to 600 gal/h (1.89 to 2.27 m³/h). A water system of this size may be able to operate satisfactorily during a peak demand. Peak flows can be estimated by adding lawn sprinkling to peak domestic flows but not to fire flows.

Fire Protection. In areas of individual water supply systems, effective firefighting depends upon the facilities provided by the property owner. The National Fire Protection Association has prepared a report that outlines and describes ways to utilize available water supplies (2).

The use of gravity water supplies for firefighting presents certain basic problems. These include the construction of a dam, farm pond, or storage tank to hold the water until needed, and the determination of the size of pipeline installed from the supply. The size of pipe is dependent upon two factors:

1. The total fall or head from the point of supply to the point of use
2. The length of pipe required

TABLE 5.4 Rates of Flow for Certain Plumbing, Household, and Farm Fixtures (2)

Location	Flow pressure*		Flow rate	
	kgf/cm ²	lb/in ²	L/s	gal/min*
Ordinary basin faucet	0.56	8	0.13	2.0
Self-closing basin faucet	0.56	8	0.16	2.5
Sink faucet, $\frac{3}{8}$ in	0.56	8	0.28	4.5
Sink faucet, $\frac{1}{2}$ in	0.56	8	0.28	4.5
Bathtub faucet	0.56	8	0.38	6.0
Laundry tub faucet, $\frac{1}{2}$ in	0.56	8	0.32	5.0
Shower	0.56	8	0.32	5.0
Ballcock for closet	0.56	8	0.19	3.0
Flush valve for closet	1.05	15	0.95–2.52	15–40 [†]
Flushometer valve for urinal	1.05	15	0.95	15
Garden hose (50 ft, $\frac{3}{4}$ -in sill cock)	2.11	30	0.32	5.0
Garden hose (50 ft, $\frac{3}{8}$ -in outlet)	1.05	15	0.21	3.33
Drinking fountains	1.05	15	0.05	0.75
Fire hose, $\frac{1}{2}$ in, $\frac{1}{2}$ -in nozzle	2.11	30	2.52	40

*Flow pressure is the pressure in the supply near the faucet or water outlet while the faucet or water outlet is wide open and flowing.

[†]Wide range is due to variation in design and type of closet flush valves.

A properly constructed well tapping a good aquifer can be a dependable source for both domestic use and fire protection. If the well is to be relied upon for fire protection without supplemental storage, it should demonstrate, by pumping test, minimum capacity of 8 to 10 gal/min (0.03 to 0.04 m³/min) continuously for a period of 2 h during the driest time of the year.

A more dependable installation results when motor, controls, and powerlines are protected from fire. A high degree of protection is achieved when all electrical elements are located outside at the well, and there is a separate powerline bypassing other buildings.

The smallest individual pressure systems commercially available provide about 210 gal (0.80 m³) per house or 3.5 gal/min (0.01 m³/min). While this capacity will furnish a stream, through an ordinary garden hose, of some value in combating incipient fires or in wetting down adjacent buildings, it cannot be expected to be effective on a fire that has gained any headway. When such systems are already installed, connections and hose should be provided. When a new system is being planned or a replacement of equipment made, it is urged that a capacity of at least 500 gal/h (1.89 m³/h) an hour or 8.33 gal/min (0.032 m³/min) be specified and the supply increased to meet this demand. If necessary, storage should be added. The additional cost for the larger unit necessary for fire protection is partially offset by the increased quantities of water available for other uses and will reduce fire insurance premiums.

Manufacturing Use

In 1954, manufacturing industries withdrew about 32.4 billion gallons per day (bgd) (122.6 million m³/day) of fresh and saline water from surface and groundwater sources and recycled this water within the plant about 1.8 times before returning 93.2% to a water source. By 1975, the combined amount of water withdrawn increased to 60.9 bgd (230.4 million m³/day), which was recycled about 2.2 times before 90.5% was returned to surface water sources (*1*).

Agricultural Use

Water in agriculture is used for irrigation and livestock. Livestock watering is only 1% of the total water withdrawal for agricultural use. Of all functional water uses, irrigation is the largest agricultural use of water. There are about 45 million acres (18.2 ha) of irrigated farmland. This offers the greatest opportunity for conservation of water that could be diverted to additional agricultural production or other uses. In 1975, the irrigation water withdrawals were 158.7 bgd (600.7 million m³/day) and consumptive use was 86.4 bgd (327.0 million m³/day). Withdrawals in the future are projected to decline somewhat (*1*).

Energy Production

Steam-powered generation of electricity accounted for 95%, or 88.9 bgd (336.5 million m³/day), of the freshwater withdrawals for all energy production in 1975. As a result of advances in cooling technology, by the year 2000, freshwater withdrawals for steam-electric generation are projected to decrease by 11% to 79.5 bgd (300.9 million m³/day), but will constitute 94% of the freshwater withdrawals for energy production. Although steam-electric generation requires large amounts of water, its consumptive use of water was only 2% of the freshwater withdrawals for steam-electric generation in 1975. However, along with the projected decrease in withdrawals by the year 2000, due to improvements in cooling technology, consumption is projected to increase to 13% of the freshwater withdrawals for steam-electric generation. This significant increase is due largely to greater evaporation resulting from higher temperatures in the recycling process (*1*).

Minerals Industry

In the minerals industry, water is used for mining metals, nonmetals, and fuels. Metals include lead, zinc, manganese, gold, silver, copper, and molybdenum. Nonmetals are lime, clay, barite, phosphate, rock salt, pumic cement, sand and gravel, and stone. Fuels include coal, oil shale, petroleum, natural gas, and natural gas liquids. The minerals industry went through an average of 7.1 bgd (26.9 million m³/day) or 2.1% of the nation's freshwater withdrawals in 1975. Water withdrawals for the minerals industry are projected to increase 61% by the year 2000, to an average of 11.3 bgd (42.8 million m³/day) or 3.7% of the nation's fresh-water withdrawals.

Water Conservation

Properly constructed water conservation programs provide an alternative to increasing resource yields. In the following sections, conservation possibilities for urban household and commercial system use, industrial and agricultural use, and the effect of pricing possibilities in conservation are discussed (3).

Urban Water Conservation. Approximately 50% of urban water is for interior use. Table 5.5 lists some structural opportunities for water savings and their percentage reduction. The means for reducing residential waste of water are also appropriate for the commercial and governmental sectors. Other potential areas of saving are optimal lawn water programs and leak-detection programs (3).

Industrial Water Savings. In the industrial sector, savings are most possible through recycling of water. This already has received a great amount of attention by some industries in their effort to control waste discharges to avoid penalties for contributing to water pollution. Industry, particularly, has benefited from water-pricing systems that favor large water users. Revisions of these systems would encourage additional efforts to reduce freshwater intake (3).

Agricultural Water Conservation. Depending on the circumstances in each case, agricultural irrigation efficiency may be increased by changing to sprinkler or drip systems, improving operation of existing systems

TABLE 5.5 Potential Residential Interior Water Savings (3)

Feature	Water savings as a % of interior use
New Construction	
Low-flush toilets	18
Low-flow showerheads	12
Low-flow kitchen and lavatory faucets	2
Pressure-reducing valves	5
Insulated hot-water lines	4
Low-water-using clothes washers	5
Low-water-using dishwashers	4
Existing Housing	
Plastic bottles or water dams in toilet reservoir	18
Replace showerheads with low-flow variety or install flow restrictors	12
Place low-flow aerators on kitchen and lavatory faucets or replace entire unit	2
Pressure-reducing valves	5
Insulated hot-water lines	1

(including better irrigation scheduling), and improving other aspects of farm management. Irrigation water use may be reduced by selecting low-water-using crops and, in some cases, by actions to reduce plant consumptive use. Water districts can save water by lining ditches and canals and assisting farmers in becoming more efficient by following more effective water delivery schedules. The opportunity for water savings in an area depends on how much outflow is needed to maintain salt balance and the disposition of the excess applied water, i.e., whether it is reused or disposed of into bodies of saline surface or groundwater.

Pricing. The principal goal in setting rates for water is to secure sufficient revenue to offset all costs, and in case of commercial companies, to achieve a profit. Other goals should be that the pricing system is equitable and discourages waste. Although what is “equitable” and what is “waste” might be variously defined, examination of current pricing systems indicates that little attention has been given to either in most cases.

Seven common pricing systems are briefly described in Table 5.6. There are many others, including rates based on meter size and separate rates for different types of uses, but those described in Table 5.6 represent the basic types. A flat rate and a declining block rate are least equitable and do not promote the elimination of waste. Although the uniform rate can be considered equitable, it often has only minor effect on waste.

TABLE 5.6 Summary of Pricing Systems (3)

Type of system	Definition and comments	Degree of equity	Discouragement of waste
Metering	<ol style="list-style-type: none"> 1. Not generally thought of as a pricing method; is essential to effect most pricing programs. 2. Installation of meters in nonmetered areas usually results in decrease in consumption of at least 25%. 	Required for equity	Yes
Flat rate	<ol style="list-style-type: none"> 1. Usually found in unmetered areas; each customer is charged the same regardless of the amount of water used. 2. Sometimes the rate is varied according to the size of delivery line. 3. Easy for utilities to manage. 	Not equitable	No
Declining block rate	<ol style="list-style-type: none"> 1. Customer is charged a certain block rate amount for an initial quantity or “block” of water. The rate for succeeding blocks decreases with each block. 	Not equitable	No
Uniform rate	<ol style="list-style-type: none"> 1. Each unit of water costs the same. 	Equitable	Minor
Increasing block rate	<ol style="list-style-type: none"> 1. Customer is charged a certain amount for an initial quantity or “block” of water. The rate for succeeding blocks increases with each block. 	Equitable	Yes
Peak load, or seasonal, rate	<ol style="list-style-type: none"> 1. Customer is charged a uniform rate for a certain quantity of water. This quantity is usually based on the reduced lawn irrigation season use or on the average demands on the water distribution system. 2. Quantities used above the amounts determined in (1) are charged at a higher rate. 	Equitable	Yes
Lifeline rate	<ol style="list-style-type: none"> 1. Requires that the rate for a certain amount of energy service (lifeline amount) cannot be increased until rates for amounts above the lifeline amounts are raised by a fixed percentage. 	Equitable	Yes

The increasing block rate and the peak load, or seasonal, rate may offer the greatest opportunity for discouraging waste.

The exact relationship between water price and rate of use has not been clearly established. Documentation of case experiences of rate-of-use changes with increases in price is rare and usually incomplete. Although dissertations presenting economic theory abound, actual experiences sometimes appear to contradict the assertions in some of these that there will be significant and continuing reductions in water use with increase in price. However, it is only logical to assume that at some level, price will have a distinct impact on rate of use.

The key to an effective water pricing policy from the standpoint of conservation is to make it clear to the user that money can be saved by minimizing water use. Therefore, the increase in water costs for quantities above the reasonable minimum required must be great enough to attract attention and become a factor of special concern in the user's budget or operating expenses. Pricing systems, such as the increasing block rate and the peak load, or seasonal rate offer the opportunity to increase an awareness of the relationships between quantity used and cost.

Ad valorem taxes for water are not visible as part of the water bill. Such taxes are often imposed in recognition of the general benefit of capital improvements to a water system, such as providing water for firefighting. However, in the interest of water conservation, they should be discontinued wherever practical, and the revenue collected through the regular rate system.

In some areas, charges for sewage treatment are included in the water bill. This is justified on the basis that the size of sewage treatment facilities is a function of the quantity of water to be treated. It serves the purpose of encouraging conservation in the manner indicated above, i.e., by making the water bill significant to the user.

Metering. Metering serves two purposes related to water conservation. First, installation of meters has reduced water use by at least 25% in most cases (an example of the significance of creating an awareness on the part of the users of a relationship between quantity used and cost). Second, metering is necessary for any pricing system other than a flat rate.

WATER QUALITY

Few factors in drinking water are as important as the effects of raw water quality on finished water. In order to ensure potable water to the consumer, national drinking water standards have been established (see Chapter 3). These issues are discussed in this section.

Raw Water Supply

The nation's surface water, its lakes, wetlands, rivers, and streams, is a visible resource that is intensively used. Less visible, and generally of less public concern, is the water that lies underground. Nonetheless, ground water is a vital resource that supplies one-quarter of all the freshwater used by industry, in agriculture, and for drinking water in the United States (*I*).

Efforts made in the last decade to improve surface water quality are beginning to show results. Data suggest that the quality of surface waters is no longer deteriorating, despite continued increases in population and the gross national product. Factories, municipal treatment facilities, and other point sources of pollution are gradually coming under control, although street and farm runoff and other nonpoint sources are often as serious polluters of surface water as the point sources.

Similar efforts to monitor and control groundwater quality have not been made, primarily because groundwater has traditionally been considered very good. Recent data suggest that serious problems exist. There is increasing evidence of groundwater contamination by synthetic organic chemicals.

Groundwater has always been thought of as a pristine resource, but recent information reveals the fact that, in many locations, groundwater is contaminated. Data show that this contamination comes from many different sources, and includes a variety of materials, most notably toxic organic and inorganic chemicals. Because groundwater is widely used for drinking water, these toxic chemicals are being found in drinking water supplies, and they may be posing unacceptable human health risks.

The discovery of contamination of groundwater by synthetic organic chemicals is particularly disturbing. Their concentrations are often orders of magnitude higher than those found in raw or treated drinking water drawn from the most contaminated surface supplies, among them the Ohio and Mississippi rivers (4). There are no federal health standards for most of the organic compounds found in drinking water wells. Many of the chemicals are known or suspected carcinogens and mutagens, which can pose unacceptable human health risks with concentrations on the order of 10 ppb and below-levels typical of surface water contamination. Even at concentrations that can cause substantial risk, the compounds are often tasteless and odorless.

Groundwater found in unconsolidated formations (sand, clay, and gravel) and protected by similar materials from sources of pollution is more likely to be safe than water coming from consolidated formations (limestone, fractured rock, lava, etc.). Where limited filtration is provided by overlying earth materials, water of better sanitary quality can sometimes be obtained by drilling deeper. It should be recognized, however, that there are areas where it is not possible, because of the geology, to find water at greater depths.

Drinking Water Regulations

In the United States the first set of drinking water standards were issued in 1914 and revised repeatedly until 1962 (128). These early standards applied only to the source of supply for interstate carriers such as rail-roads, buses, and airlines. The Safe Drinking Water Act of 1974 (SDWA, PL 93-523) led to promulgation of the first set of nationwide primary and secondary drinking water standards that apply to all community water supplies in the United States (129, 130). The 1986 Amendments to the Safe Drinking Water Act (SDWAA) substantially increased the number of drinking water regulations that must be met by water utilities.

The SDWAA requires that the Environmental Protection Agency (EPA) establish maximum contaminant level goals (MCLGs) for each contaminant that may have an adverse effect on the health of drinking water consumers (131). Each MCLG is required to be set at a level at which no known or anticipated adverse effects to health occur, allowing for an adequate margin of safety. Maximum contaminant levels (MCLs) must be set as close to MCLGs as feasible.

The SDWAA authorized EPA to set treatment technique requirements in lieu of MCLs when it is not economically or technically feasible for water suppliers to determine the level of contaminants. In addition to the quantitative provisions of the MCLs, the SDWAA contains several far-reaching provisions on monitoring, filtration, disinfection, and use of lead products. EPA's regulatory effort under the SDWAA is divided into the following five phases:

- Phase I: Volatile organic compounds
- Phase II: Synthetic organic compounds, inorganic compounds, and unregulated contaminant monitoring
- Phase III: Radionuclide contaminants
- Phase IV: Disinfectants and oxidant by-products
- Phase V: Inorganic compounds and synthetic organic compounds

The EPA has entered into a negotiated rulemaking process with the drinking water industry, environmental interest groups and other affected parties to set the future agenda for regulatory activity in the United States. This agenda will include an information collection rule (ICR), two stage disinfection by-product rule, and an enhanced surface water treatment rule (ESWTR) (132).

In addition to the EPA regulations, the American Water Works Association (AWWA) has adopted a recommended set of “Water Quality Goals” These “goals” are part of an official AWWA policy and specify filtered water turbidity, color, and taste and odor (128). These “goals” include recommended levels for iron, aluminum, and manganese, and specify that water should be noncorrosive.

There also are many international standards and criteria that are intended to control drinking water quality. Most notable among these guidelines and standards are those established by the World Health Organization (WHO) and the European Community (EC).

Drinking Water Quality

Precipitation in the form of rain, snow, hail, or sleet contains very few impurities. It may entrain trace amounts of mineral matter, gases, and other substances as it forms and falls through the earth’s atmosphere. The precipitation, however, has virtually no bacterial content.

Once precipitation reaches the earth’s surface, many opportunities are presented for the introduction of mineral and organic substances, microorganisms, and other forms of pollution (contamination). When water runs over and through the ground surface, it may pick up particles of soil. This is noticeable in the water as cloudiness or turbidity. It also picks up particles of organic matter and bacteria. As surface water seeps down into the soil and through the underlying material to the water table, most of the suspended particles are filtered out. This natural filtration may be partially effective in removing bacteria and other particulate materials; however, the chemical characteristics of the water may change and vary widely when it comes in contact with mineral deposits. Chemical and bacteriological analyses can be performed by a state or local health department or by a commercial laboratory. A wide variety of analytical methods are used to characterize drinking water quality to meet regulatory requirements and to evaluate performance at each step of a treatment process (133–135).

The widespread use of synthetically produced chemical compounds, including pesticides and insecticides, has caused a renewed interest in the quality of water. Many of these materials are known to be toxic and others have certain undesirable characteristics that interfere with the use of the water even when present in relatively small concentrations. In recent years instances of water pollution have been traced to sewage or wastewater sources containing synthetic detergents.

Substances that alter the quality of water as it moves over or below the surface of the earth may be classified under four major headings (2).

1. *Physical*: Physical characteristics are related to the quality of water for domestic use and are usually associated with the appearance of water, its color or turbidity, temperature, taste, and odor.
2. *Chemical*: Chemical differences between waters are sometimes evidenced by their observed reactions, such as the comparative performance of hard and soft waters in laundering.
3. *Biological*: Biological agents are very important in their relation to public health and may also be significant in modifying the physical and chemical characteristics of water.
4. *Radiological*: Radiological factors must be considered in areas where there is a possibility that the water may have come in contact with radioactive substances.

Physical Characteristics. The water as used should be free from all impurities that are offensive to the sense of sight, taste, or smell. The physical characteristics of the water include turbidity, color, taste and odor, temperature, and foamability.

Turbidity. The presence of suspended material, such as clay, silt, finely divided organic material, plankton, and other inorganic material in water is known as turbidity. Turbidities in excess of 5 units are easily detectable in a glass of water and are usually objectionable for esthetic reasons. Clay or other inert suspended particles in drinking water may not adversely affect health, but water containing such particles may require treatment

to make it suitable for its intended use. Following a rainfall, variation in the groundwater turbidity may be considered an indication of surface or other introduced pollution.

Turbidity is a measure of the light scattered by the suspended particles and is standardized by using Formazin suspension as a calibration standard. All turbidimeters are designed to specified standardization criteria. (136–138) Optical particle counters are more sensitive to small changes in suspended matter concentrations, and are used to evaluate treatment plant performance (139). The Enhanced Surface Water Treatment Rule proposes quantification of removal of pathogens. Development of rapid analytical methods for detection and characterization of such pathogens using particle counters and cytometry are currently under investigation.

Color. Dissolved organic material from decaying vegetation and certain inorganic matter cause color in water. Occasionally, excessive blooms of algae or the growth of aquatic microorganisms may also impart color. While color itself is not usually objectionable from the standpoint of health, its presence is esthetically objectionable and suggests that the water needs appropriate treatment.

Taste and Odor. Taste and odor in water can be caused by foreign matter, such as organic compounds, inorganic salts, or dissolved gases. These materials may come from domestic, agricultural, or natural sources. Some substances found naturally in groundwater, while not necessarily harmful, may impart a disagreeable taste or undesirable property to the water. Magnesium sulfate (Epsom salt), sodium sulfate (Glauber's salt), and sodium chloride (common table salt) are but a few of these.

Acceptable waters should be free from any objectionable taste or odor at point of use. Knowledge concerning the chemical quality of a water supply source is important in order to determine what treatment, if any, is required to make the water acceptable for domestic use.

Temperature. The most desirable drinking waters are consistently cool and do not have temperature fluctuations of more than a few degrees. Groundwater and surface water from mountainous areas generally meet these criteria. Most individuals find that water having a temperature between 10 and 15°C (50 and 60°F) is most palatable. The temperature of groundwater remains nearly constant throughout the year. Water from very shallow sources (less than 50 ft deep) varies somewhat from one season to another, but water from deeper zones remains quite constant, its temperature being close to that for the average annual temperature at the surface. This is why water from a well may seem to be warm in winter or cold during the summer.

Contrary to popular opinion, colder water is not obtained by drilling deeper. Beyond about 100 ft of depth, the temperature of groundwater increases steadily at the rate of about 0.6°C (1°F) for each 75 to 150 ft (25 to 50 m) of depth. In volcanic regions, this rate of increase may be much greater.

Foamability. Since 1965, the detergent formulations have been changed to eliminate alkyl benzene sulfonate (ABS), which was very slowly degraded by nature. The more rapidly biodegradable linear alkylate sulfonate (LAS) has been substituted in most detergents. Even LAS is not degraded very rapidly in the absence of oxygen—a condition that exists in cesspools and some septic tank file fields.

Foam in water is usually caused by concentrations of detergents greater than 1 mg/L. 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 materials of sewage origin are also likely to be present.

Chemical Characteristics. The nature of the rocks that form the earth's crust affects not only the quantity of water that may be recovered but also its characteristics. As surface water seeps downward to the water table, it dissolves portions of the minerals contained by soils and rocks. Groundwater, therefore, usually contains more dissolved minerals than surface water.

The chemical characteristics of water in a particular locality can sometimes be predicted from analyses of adjacent water sources. These data are often available in published reports of the U.S. Geological Survey or from federal, state, and local health, geological, and water agencies. In the event that the information is not available, a chemical analysis of the water source should be made. Some state health and geological

departments, as well as state colleges and many commercial laboratories, have the facilities and may be able to provide this service.

Information that can be obtained from a chemical analysis is

- The possible presence of harmful or disagreeable substances
- The potential for the water to corrode parts of the water system
- The tendency for the water to stain fixtures and clothing

The following is a discussion of the chemical characteristics of water. Some parameters are based on the limits recommended by the 1962 Public Health Service Drinking Water Standards (5).

Toxic Substances. Water may contain toxic substances in solution. If analysis of the water supply shows that these substances exceed the following concentrations, the supply should not be used (Table 5.7).

Chlorides. Most waters contain some chloride in solution. The amount present can be caused by the leaching of marine sedimentary deposits, by pollution from seawater, brine, or industrial and domestic wastes. 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 groundwater or surface water may indicate possible pollution from sewage sources, particularly if the normal chloride content is known to be low.

Copper. Copper is found in some natural waters, particularly in areas where these ore deposits have been mined. Excessive amounts of copper 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 recommended limit for copper is 1.0 mg/L.

Fluorides. In some areas water sources contain natural fluorides. Where the concentrations approach optimum levels, beneficial health effects have been observed. In such areas, the incidence of dental caries has been found to be below the rate in areas without natural fluorides. The optimum fluoride level for a given area depends upon air temperature, since that is what primarily influences the amount of water people drink. Optimum concentrations from 0.7 to 1.2 mg/L are recommended. Excessive fluorides in drinking water supplies may produce fluorosis (mottling) of teeth, which increases as the optimum fluoride level is exceeded. The state or local health departments, therefore, should be consulted for their recommendations. (Maximum allowable concentrations are presented in Table 5.8.)

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 recommended limit for iron is 0.3 mg/L.

Lead. A brief or prolonged exposure of the body to lead can be seriously injurious to health. Prolonged exposure to relatively small quantities may result in serious illness or death. Lead taken into the body in

TABLE 5.7 Maximum Contaminant Levels (5)

Substance	Level, mg/L	Substance	Level, mg/L
Arsenic (As)	0.05	Fluoride (F)	2
Barium (Ba)	1.00	Lead (Pb)	0.05
Cadmium (Cd)	0.01	Selenium (Se)	0.01
Chromium (Cr ⁺⁶)	0.05	Silver (Ag)	0.05
Cyanides (CN)	0.2		

TABLE 5.8 Maximum Concentrations of Fluoride (5)

Annual average of maximum daily air temperature, °F	Maximum allowable fluoride concentrations, mg/L
50.0–53.7	2.4
53.8–58.3	2.2
58.4–63.8	2.0
63.9–70.6	1.8
70.7–79.2	1.6
79.3–90.5	1.4

Note: °C= $\frac{5}{9}$ (°F-32)

quantities in excess of certain relatively low “normal” limits is a cumulative poison. A maximum concentration of 0.05 mg/L of lead in water must not be exceeded. 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.

Manganese. There are two reasons for limiting the concentration of manganese in drinking water: (1) to prevent esthetic and economic damage, and (2) 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. The recommended limit for manganese is 0.05 mg/L.

Nitrates. Nitrate (NO_3^-) has caused methemoglobinemia (infant cyanosis or “blue baby disease”) in infants who have been given water or fed formulas prepared with water having high nitrates. A domestic water supply should not contain nitrate concentrations in excess of 45 mg/L (10 mg/L expressed as nitrogen). Nitrates in excess of normal concentrations, often in shallow wells, may be an indication of seepage from live-stock manure deposits. In some polluted wells, nitrate will also be present in concentrations greater than 1 mg/L and is even more hazardous to infants. When the presence of high nitrate 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.

Pesticides. Careless use of pesticides can 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. The use of pesticides near wells is not recommended.

Sodium. When it is necessary to know the precise amount of sodium present in a water supply, a laboratory analysis should be made. When home water softeners utilizing the ion-exchange method are used, the amount of sodium will 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 of salt from other sources 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.

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. Sulfate content should not exceed 250 mg/L.

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. For this reason, the recommended limit for zinc is 5.0 mg/L.

Serious surface and groundwater pollution problems have developed from existing and abandoned mining operations. Among the worst are those associated with coal mine operations, where heavy concentrations of iron, manganese, sulfates, and acids have resulted from the weathering and leaching of minerals.

Alkalinity. Alkalinity is imparted to water by bicarbonate, carbonate, or hydroxide compounds. The presence of these compounds is determined by standard methods involving titration with various indicator solutions. Knowledge of the alkalinity sources is useful in the treatment of water supplies.

Hardness. 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 scale (as in a kettle, heating coils, or cooking utensils) with a consequent waste of fuel.

The mineral content of groundwater reflects its movement through the minerals that make up the earth's crust. Generally, groundwater in arid regions is harder and more mineralized than water in regions of high annual rainfall. Also, deeper aquifers are more likely to contain higher concentrations of minerals in solution, because the water has more time (perhaps millions of years) to dissolve the mineral rocks.

Calcium and magnesium salts, which cause hardness in water supplies, are divided into two general classifications: carbonate, or temporary hardness, and 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. Non-carbonate hardness is due largely to the presence of the sulfates and chlorides of calcium and magnesium in the water.

pH. 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.

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

Some organisms that cause disease in humans originate with the fecal discharges of infected individuals. It is seldom practical to monitor and control the activities of human disease carriers. For this reason, it is necessary to exercise precautions against contamination of a normally safe water source or to institute treatment methods that will produce a safe water.

When water seeps downward through overlying material to the water table, particles in suspension, including microorganisms, may be removed (2). The extent of removal depends on the thickness and character of the overlying material. Clay or "hardpan" provides the most effective natural protection of groundwater. Silt and sand also provide good filtration if fine enough and in thick enough layers. The bacterial quality of the water also improves during storage in the aquifer, because storage conditions are usually unfavorable for bacterial survival. Clarity alone does not guarantee that groundwater is safe to drink; this can only be determined by laboratory testing.

Certain forms of aquatic vegetation and microscopic animal life in natural water may be either stimulated or retarded in their growth cycles by physical, chemical, or biological factors. For example, the growth of algae, minute green plants usually found floating in surface water, is stimulated by light, heat, nutrients such as nitrogen and phosphorus, and the presence of carbon dioxide as a product of organic decomposition. Their growth may, in turn, be retarded by changes in pH, the presence of inorganic impurities, excessive cloudiness or darkness, temperature, and the presence of certain bacterial species.

Continuous cycles of growth and decay of algal cell material may result in the production of noxious by-products that may adversely affect the quality of a water supply. The same general statements may be

made regarding the growth cycles of certain nonpathogenic bacteria and microcrustacea that inhabit natural waters.

Contamination of water by biological agents can be minimized by:

1. Selecting water sources that do not normally support much plant or animal life
2. Protecting the supply against subsequent contamination by biological agents
3. Minimizing entrance of fertilizing materials, such as organic and nutrient minerals
4. Providing treatment for the destruction of biological life or its by-products

Radiological Characteristics. 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 Public Health Service Drinking Water Standards are intended to limit the human intake of these substances so that the total radiation exposure of any individual will not exceed those defined in the Radiation Protection Guides recommended by the Federal Radiation Council. Humans have 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.

Radiological data indicating both background and other forms of radioactivity in an area may be available in publications of the U.S. Environmental Protection Agency, U.S. Public Health Service, U.S. Geological Survey, or from federal, state, or local agencies. For information or recommendations on specific problems, the appropriate agency should be contacted.

Potable Reuse. Potable reuse is the renovation of sewage effluent to a water product suitable for human consumption and the recycling of that water into a supply system (6). Wastewater can, of course, also be reused for nonpotable purposes, and this is already being done in industry, agriculture, and recreation.

As interest in renovated wastewater grows, more and more communities and organizations are investigating its potential. The uses to which reused water should be put will, of course, vary with local conditions; any community that is considering reuse will have to evaluate fully the problems and possibilities of its own situation.

Potable reuse can be accomplished in any of three ways:

1. Direct potable reuse is the reintroduction of highly treated sewage effluent from the treatment plant directly back into the existing water distribution system. This is the classic “pipe-to-pipe” definition of reuse.
2. Planned indirect reuse involves the purposeful discharge of highly treated wastewater upstream from a water supply intake.
3. Groundwater recharge involves either the injection of effluent into an aquifer that is the source of potable supply or the “spreading” of effluent on the ground to allow it to filter down to the aquifer.

In each of these methods, renovated wastewater eventually reaches the home water tap.

Nonpotable Water Quality

Commercial, industrial, and agricultural uses have different requirements for drinking water, and nonpotable sources may be used. In other cases, higher quality waters may be required. These criteria are discussed by topical use categories.

Industrial Process Requirements. The quality tolerances for process waters vary with the manufacturing process and with the quality of the goods to be produced. The point at which it becomes economical to improve the available supply is found through comparisons between (1) the cost of treatment and (2) the increase in value of the product together with the decrease in costs of operation, maintenance, and replacement of equipment. Rising costs, higher standards of production, closer quality control of manufactured articles, and deterioration of water sources have made for a growing awareness of the importance of water quality in industry. When the available water is treated, therefore, the maximum economic improvement in its quality is sought. Tolerances of impurities in the treated water are generally stricter than those for untreated waters that would be used if they were available.

Many industrial products or operations call for water that is clear, colorless, tasteless, relatively soft, free from iron, manganese, hydrogen sulfide, and organic matter, and of approved bacteriological quality. Examples are bottled beverages, fine chemicals, canned goods, processed milk, ice, packed meat, edible oils, laundering operations, and the printing and dyeing of textiles. Most soft municipal waters are of this nature. Process waters drawn from a private source, however, may have to be purified. The principles of treatment discussed for municipal water supplies apply equally well to the large body of process waters.

Wash waters should be soft when soap is the detergent of use and when deposition of residues by evaporating rinse water is to be avoided. Among industries that are sensitive to hardness are laundries, electroplating plants, milk plants, ice plants, and textile mills. They generally soften or demineralize the available water. Breweries, distilleries, and bakeries, on the other hand, require relatively hard water for some manufacturing operations and may have to add hardness. Pulp and paper mills, tanneries, oil refineries, and steel mills do not commonly need water of drinking quality, but they may have other requirements. Paper pulp and low-quality paper can be made with colored water that contains as much as 50 units of turbidity, but it should be low in iron and manganese, hardness, and carbon dioxide. Formation of organic slimes and mineral scales must also be kept under control. By contrast, high-quality paper can be manufactured only with high-quality water. The situation is much the same in tanning. In oil refineries and steel mills, water is used primarily for steam production and cooling. Quality requirements for the cooling water of these industries are much the same as those for cooling waters in general. In the rolling of steel, however, a chloride content above 150 mg/L in cooling water causes rapid deterioration of rolls. Quality tolerances for a variety of process waters are summarized in Table 5.9 (7).

As a case in point, it is impossible to give a single water quality specification even for such a common industrial water requirement as steam-boiler makeup. Most city water supplies can be used directly and without treatment for makeup to cast iron steam-heating boilers if these operate with virtually no loss of steam or condensate. With steel-tubed heating boilers, however, corrosion-control treatment is needed with most waters. Furthermore, scale-control treatment is essential with any boiler of either metal whenever appreciable amounts of makeup water are used, making it the equivalent of a process steam boiler. But water treatment satisfactory for use with high-pressure boilers is completely inadequate for use with high-pressure boilers generating steam for production of electricity in a turbine-driven generator. Certain of these require a boiler water that is as free of all dissolved solids and gases as possible.

In the operation of boilers, foaming, priming, scale formation, caustic embrittlement, and corrosion mount with operation pressures. Foaming and priming entrain moisture and solids in steam. The solids carried over may then be deposited in steam lines and in turbines and other equipment. The intercrystalline cracking of boiler metal, called *caustic embrittlement*, is associated (1) with localized stresses that have strained the metal beyond its elastic limit, and (2) with a high concentration of caustic soda in the absence of an adequate concentration of sulfate in the feed water. Failure generally occurs at riveted seams and similar places of confined extent that have been subjected to stresses of high intensity. Table 5.10 suggests feedwater tolerances for different pressures. The values listed take into account (1) reductions in turbidity, color, organic matter (as measured by oxygen consumed); (2) reductions in hardness, silica, and alumina alleviate scale formation; (3) maintenance of a high ratio of sulfate to carbonates controls caustic embrittlement; and (4) elimination of oxygen, reduction of bicarbonate ions, and increase in pH suppress corrosion. Municipal

TABLE 5.9 Quality Tolerances for Industrial Process Waters (All values are expressed in mg/L) (7)

Industry or use	Turbidity	Color	Hardness as CaCO ₃	Iron as Fe	Manganese as Mn	Total solids	Alkalinity as CaCO ₃	Odor, taste	Hydrogen sulfide	Other requirements
Air conditioning	0.5*	0.5	Low	1	No corrosiveness or slime formation
Baking	10	10	...	0.2*	0.2	Low	0.2	Potable water
Brewing light beer	10	0.1*	0.1	500	75	Low	0.2	Potable, NaCl less than 275 ppm, pH 6.5-7.0
dark beer	10	0.1*	0.1	1,000	150	Low	0.2	Potable, NaCl less than 275 ppm, pH 7.0 or more
Canning legumes general	10	...	25-75	0.2*	0.2	Low	1	Potable
Carbonated beverages	2	10	250	0.2(0.3)*	0.2	850	50-150	Low	0.2	Potable
Confectionery	0.2*	0.2	100	...	Low	0.2	Potable, organic color plus oxygen consumed <10 mg/L
Food, general	10	0.2*	0.2	Low	...	Potable
Ice	5	5	...	0.2*	0.2	Low	...	Potable, SiO ₂ <10 mg/L
Laundry	50	0.2*	0.2	Low	...	No grit or corrosiveness
Plastics, clear, uncolored	2	2	...	0.02*	0.02	200	
Paper and pulp ground wood	50	20	180	1.0*	0.5	300	
Kraft pulp	25	15	100	0.2*	0.1	200	
Soda and sulfite	15	10	100	0.1*	0.05	200	No slime formation
High-grade light papers	5	5	50	0.1*	0.05	200	Al ₂ O ₃ <8 mg/L, SiO ₂ <25 mg/L, Cu <5m g/L, pH 7.8 to 8.3
Rayon (Viscose) pulp production	5	5	8	0.05*	0.03	100	Total 50, Hydroxide 8	
manufacture	0.3	...	55	0.0	0.0	...	Total 135, Hydroxide 8	
Tanning	20	10-100	50-135	0.2*	0.2	Constant composition, residual alumina <0.5 mg/L
Textiles general	5	20	...	0.25	0.25	
dyeing	5	5-20	...	0.25*	0.25	200	...	Low	...	
Wool scouring	...	70	...	1.0*	1.0	Low	...	
Cotton bandage	5	5	...	0.2*	0.2	Low	...	

*Limit given applies to both iron alone and the sum of iron and manganese.

TABLE 5.10 Boiler Feedwater Quality Tolerances (7)

Measure of quality	Units	Pressure range, lb/in ² (kgf/cm ²)			
		0–150 (0–10.54)	150–250 (10.55–17.58)	250–400 (17.59–28.12)	over 400 (over 28.12)
Turbidity	NTU	20	10	5	1
Color		80	40	5	2
Oxygen consumed	mg/L	15	10	4	3
Dissolved oxygen (O ₂)*	mg/L	1.5	0.1	0	0
Hydrogen sulfide (H ₂ S) [†]	mg/L	5	3	0	0
Total hardness (CaCO ₃)	mg/L	80	40	10	2
Sulfate–carbonate ratio (Na ₂ SO ₄ :Na ₂ CO ₃)		1:1	2:1	3:1	
Aluminum oxide (Al ₂ O ₃)	mg/L	5	0.5	0.05	0.01
Silica (SiO ₂)	mg/L	40	20	5	0
Bicarbonate (HCO ₃ ⁻)	mg/L	50	30	5	0
Carbonate (CO ₃ ²⁻)	mg/L	200	100	40	20
Hydroxide (OH ⁻)	mg/L	50	40	30	15
Total solids [‡]	mg/L	3000 to 500	2500 to 500	1500 to 100	50
Minimum pH value		8.0	8.4	9.0	9.6

*Feedwater entering boiler.

[†]Except when odor in live steam is objectionable.

[‡]Depends on design of boiler.

*All values but the pH value are expressed in parts per million.

water supplies, as well as private water supplies, must generally be suitably prepared for use in high-pressure boilers (7).

Accordingly, it should be kept in mind that the information given below merely illustrates in general terms the broad scope of water quality requirements for cooling, steam generation, and heating. Remember, too, that the requirements for makeup water, that is, the water entering a system—and those for the water in the system—circulating water or boiler water, as the case may be, must also be carefully differentiated.

Cooling Waters. Once-through cooling waters should be reasonably free of suspended matter that may clog or settle out in the system and of excessively heavy contamination with living organisms. It is difficult to place quantitative labels upon the quality requirements for these waters. At best, it can only be pointed out that any defects of the available water supply must be correctable by very inexpensive treatment methods because of the large volume of water used. A few specific limitations that have been suggested, and which are equally applicable to open recirculating systems, are that the turbidity be below 50 mg/L, iron or manganese below 0.5 mg/L, and sulfides (as H₂S) below 5 mg/L. The latter are objectionable because of their corrosive effects.

For open recirculating systems, the calcium hardness and alkalinity of the makeup water should be low enough so that a cooling tower can operate with the circulating water equal to at least several concentrations of makeup before scale forms at a significant rate if the water-saving purpose of a cooling tower is to be achieved. Other quality requirements for cooling-tower makeup water are relatively unimportant because the changes in water composition that take place while the water is circulating through the cooling tower are much more significant in the development of operating problems than is the makeup water composition itself (8).

The circulating water must be treated to keep it reasonably noncorrosive and non-scale-forming, as well

as free from slime and algae growths. It is difficult to be specific about concentration limits for particular constituents in a circulating cooling water, because these will depend in great part upon construction and operating details for a particular installation and upon the type of treatment selected. For example, consider two cases in which corrosion is the major problem. The minimum corrosion rate can be obtained by addition of sodium chromate maintained at 200 to 500 mg/L with a minimum pH of 7 for most effective results. On the other hand, lower chemical costs are possible if a slightly higher corrosion rate is acceptable. This can be done by using much lower chromate concentrations (20 to 50 mg/L) along with a similar concentration of a corrosion inhibitor, such as a polyphosphate, but in this case, the optimum corrosion inhibition can only be obtained within a narrow, lower pH range, generally about 6.2 to 6.8 (9).

For scale control, a positive Langelier index of 0.5 to 1.0 is frequently called for, but even limits of circulating water composition for scale control based upon the Langelier index or the Ryznar stability index will vary according to the maximum temperature in the system, which may range from 40 to 43°C (105 to 10°F) for the usual air-conditioning system to 93°C (200°F) or higher for cooling waters used in steam condensers.

It is generally considered good practice to limit the total dissolved solids to not over 2000 mg/L in circulating waters of large cooling towers so as to minimize galvanic corrosion in the equipment. When sulfuric acid is used for calcium carbonate scale control, the alkalinity of the makeup water is usually held at about 15 to 25 mg/L, with that of the circulating water in the 50 to 100 mg/L range. Other common, but not invariable, limits for circulating cooling-tower waters are a pH of 6 to 8 and maximum suspended solids about 100 to 150 mg/L.

In all-wood cooling towers, the maximum chlorine concentration added for slime or algae control is held to 1 mg/L and the maximum pH to 7.0 in order to minimize delignification of wood structural members. It is not always economically possible to maintain such restrictions in smaller equipment.

Closed circulating cooling-water systems rarely have sufficient makeup to create serious scale problems, but corrosion is fairly common. For this reason it is desirable to maintain a minimum of 200 mg/L sodium chromate in such waters with a pH in the 7.0 to 8.5 range. Diesel jacket and similar closed cooling systems which have higher operating temperatures or must combat cavitation effects, often carry a minimum of 2000 mg/L sodium chromate. Suspended matter, particularly abrasive suspended matter, can damage strainers inserted in the water lines.

Steam Generation. Specific standards of concentration for the various constituents in boiler waters can be more readily established, although these will still vary considerably with operating pressure, construction, and type of operation of the boiler. Although the makeup water may be subjected to a considerable amount of treatment before entering the boiler, water quality requirements are usually expressed in terms of water within the boiler itself. Some quality requirements for the makeup water are almost self-apparent. It should certainly be free of any suspended matter. For those cases requiring low dissolved oxygen in the boiler, as much of this as possible should be removed before the water enters the boiler.

Heating boilers in which almost 100 percent of the steam returns as condensate generally require protection only from corrosion, but regular checking is necessary to ensure against development of excessive accumulation of dissolved solids, which might lead to scale formation. With chromate-treated boilers, water treatment standards range from 500 mg/L sodium chromate, which provides ample protection under conditions of good control, to 2000 mg/L sodium chromate as recommended for general use by the Steel Boiler Institute. With borate-buffered sodium nitrite as the corrosion inhibitor, a minimum of 2000 mg/L sodium nitrite is generally preferred.

Table 5.11 summarizes some typical recommendations for boiler-water composition at various operating steam pressures for process-steam boilers, those for which some regular makeup water requirement is anticipated (10). Quite probably, it would be impossible to obtain unanimous agreement of these water quality requirements from any substantial group of industrial water chemists, but they can serve as a general guide.

The divisions into various pressure classifications are arbitrary. Some of the quality specifications pertain particularly to certain types of construction for systems with certain auxiliary equipment, such as turbines.

TABLE 5.11 Boiler-Water Quality (All values except pH are given in mg/L) (10)

Constituent	Intermediate					High					Very high					Supercritical, over 3000 lb/in ²
	Low, to 250 lb/in ²	250– 300 lb/in ²	300– 450 lb/in ²	450– 500 lb/in ²	500– 600 lb/in ²	600– 750 lb/in ²	750– 900 lb/in ²	900– 1000 lb/in ²	1000– 1250 lb/in ²	1250– 1500 lb/in ²	1500– 2000 lb/in ²	2000– 2500 lb/in ²	2500– 3000 lb/in ²	3000– 5000 lb/in ²		
Total hardness, as CaCO ₃	0	0	0	0	0	0	0	0	0	0	0	0	0	0		
Total alkalinity, as CaCO ₃	...	700†	...	500†	500†	400†	300†	250	200†	150†	100†	100†	100†	100†		
Hydroxide, as CaCO ₃	150–300	...	130	100	100	100	100	50–75	50–75	50–75	50–75	50–75	50–75	50–75		
Silica [‡] as SiO ₂	100–133	50–67	50–67	50–67	19.25	19.25	7.5–10	7.5–10	3.8–5	2.1–2.9	0.8–1.1	0.3–0.5	0.09–0.13	0.02		
Dissolved oxygen, as O	0.015	0.007	0.005		
Solids, dissolved	3000– 5000	35001	3000	25001	2500	2000	1500	1250†	1000†	1000†	750†	500†	500†	0.05– 0.5†		
Solids, suspended	300–600	3001	...	150†	150†	100†	60†	401	20†	20†	10.3– 10.8	10.3– 10.8	10.3– 10.8	9.0– 9.6		
pH*	9.5	9.7	10.8	10.8	10.8		
Oil	1.0	0.1		
Iron, as Fe	0.01†		
Copper, as Cu	0.01†		
Sodium Sulfite,* as Na ₂ SO ₃	30–60		
Phosphate,* as PO ₄	40–80	30–50	30–50	20–40	20–40	20–40	20–40	20–40	20–40	20–40	20–40	20–40	20–40	20–40		
Organic dispersant*	50–75		

*These values minima. All others maxima.

†American Boiler Manufacturers Association recommendation

‡Values are for feedwater, not boiler water, for silica, this applies to pressures over 500 lb/in².

Note: lb/in² × 6.8948 = fN/m²

For each range of concentration given in Table 5.11, it is generally preferable to approach the lower limit as the boiler operating pressure approaches the upper limit of the pressure range.

Again, keep in mind that a table is no substitute for a competent boiler-water specialist. Many situations call for modification of the figures given in Table 5.11. For example, in the very high pressure range, in addition to the usual thermal circulation boilers, both forced-circulation boilers and once-through boilers are used. For both of these, boiler-water requirements differ from those given in Table 5.11 for the same operating pressures. Thus, in a once-through boiler that evaporates the water almost completely in a single passage, total solids in the feedwater should be no higher than 0.5 mg/L, and the pH should be in the range 8.8 to 9.2.

Another example of the flexibility of the figures in this table may be seen from a consideration of the maximum total-dissolved-solids limit. Generally speaking, this limit is for the purpose of minimizing foaming and pitting. Few boilers were more susceptible to carrying over slugs of water as a result of foaming or priming than railroad steam-locomotive boilers. Yet experiments showed that when such boilers are fed with completely softened water containing a minimum of suspended solids, the dissolved solids content could safely be carried above 10,000 mg/L with no significant foaming or priming. Therefore, it is well to keep in mind that the various boiler-water quality recommendations are designed for reasonably safe operation but are not rigid limits.

Heating Waters. Quality requirements for water in closed recirculating heating systems are fairly simple. In all cases, it is desirable to avoid suspended solids in order to minimize damage to pumps and other mechanical equipment. If the systems were truly tight, this is all that would be required. However, experience and chemical tracer studies have shown that completely tight systems are in the minority. One tracer study of 84 systems showed that the average water loss per month equaled the system volume (10). A monthly loss of more than 10 system volumes has been observed.

In the usual low-pressure hot-water heating system, as in steam-heating boilers, corrosion, the major operating problem, can be minimized by maintaining a minimum of 500 mg/L sodium chromate as recommended for general application by the Steam Boiler Institute. Borate-buffered sodium nitrite at 2500 mg/L minimum is sometimes used as a colorless alternative.

Although corrosion control by the addition of sodium sulfite is not reliable for low-pressure steam-heating boilers because of the amount of oxygen that may be picked up by the condensate returned to the boiler, these conditions do not apply in hot-water heating systems, so that corrosion can also be minimized by maintaining a minimum of 50 to 100 mg/L of sodium sulfite in the circulating water. With this type of treatment, periodic analyses are required.

As the operating pressure of closed heating systems increases to the range of high-temperature hot-water systems, 250 lb/m² (1724 kN/m²), a completely softened and deaerated water becomes necessary for makeup. The circulating-water pH is generally adjusted to a minimum of 9.5, and a sodium sulfite residual of 50 to 100 mg/L may be maintained, together with a phosphate residual of 30 to 60 mg/L.

For closed water loops used in nuclear installations, the development of radioactive impurities from either dissolved or suspended materials must be minimized. In these systems, the total dissolved solids and total suspended solids are kept to the lowest possible value in the circulating water by using deionized water as makeup and by passing the circulating water through a mixed-bed deionizer, which reduces both suspended and dissolved solids.

Recycle and Reuse. It is important to distinguish between water which is to be used for drinking and water intended for all other uses (11). As a matter of fact, less than 1% of the water used in the United States today needs to be of drinking water quality. However, the municipal systems, which produce most of the country's water, are designed and operated to deliver water of sufficient quality to meet the standards required by this minor use.

This will not necessarily continue to be the case. And even if it is, the majority of wastewater reuse

applications in the next 50 years will not include direct human ingestion of the renovated water. The principal purposes for which renovated wastewater will be used in the foreseeable future include:

- Agricultural irrigation and land treatment
- Industrial processes not involving food or drugs
- Creation of recreational lakes
- Groundwater recharge
- Nonpotable public utility uses, such as firefighting and greenbelt irrigation
- Limited domestic use

The potential for eventual or inadvertent human contact and/or ingestion is present in all of these uses. Thus, all have health considerations associated with their implementation.

It is important to differentiate between user quality requirements and user quality concerns. The former do not change as a function of the water source, while the latter may vary greatly as a function of the source. Thus, the user requirements for reused water for any purpose will be identical to those for water from more traditional sources; but the user concerns about reused water are likely to be increased significantly.

Health concerns to humans from reused water can be reduced to two classes—microorganisms and chemicals. Some research has been directed toward health effects associated with noningestion exposures from both of these. However, most evaluations must be made by transferring data from research done for other purposes.

Some of the possible routes of human exposure to microorganisms and chemicals associated with various nonpotable modes of reuse previously listed include aerosols, food from both plants and animals, skin and eye contact from recreation, industrial processes, land treatment, and contact for hygienic purposes.

Biological warfare studies have given us some understanding of the infectivity of aerosolized microorganisms. Little is known, however, about viruses in aerosols, since the methodology to detect and identify viruses was not well advanced during the days of biological warfare research. Food studies directed toward both microorganisms and chemicals have been conducted, but knowledge of the effects of organic chemicals in foods is sketchy.

Most data on skin and eye problems resulting from exposure to microorganisms must be transferred from studies conducted on swimmers. Information on skin and eye problems resulting from exposure to chemicals is based chiefly on occupational health studies. Use of renovated waters to fill recreational lakes will continue to have limited applications because of the costs involved, but should not be delayed by the lack of accepted health standards.

Reuse within industry, except for food processing and pharmaceutical applications, now can be fully exploited, with a low probability of increasing health hazards. Industrial reuse presents relatively few health concerns, and controlling the treatment of the influent wastewater allows an additional opportunity for reducing these concerns. If the addition of organic chemicals is controlled by the nature of the water usage, the reuse treatment train can be simplified. The major factor delaying the expansion of wastewater reuse in industry is not health concerns but rather the difficulty of producing and delivering water to the user at a competitive price and meeting quality specifications required for the industrial operations.

Land treatment offers a near-term reuse application of potentially significant magnitude. In the United States, about one-half of the states already have regulations or guidelines for the design and operation of wastewater land treatment systems. Many of these regulations are intended to prevent hazards to human health and to the environment from the incomplete renovation of the wastewater before application and from the distribution of contaminants during the application process, such as the generation of aerosols from spray irrigation machinery. The requirements set by different states vary widely, and there is no uniform federal policy toward land treatment with renovated wastewater. This confusion stems primarily from gaps in our knowledge of the dose-response relationships for many organisms and chemicals.

The research needs concerning the health effects of land application of wastewater are achievable within a couple of years. Crucial to the public and professional understanding and acceptance of these reuse

techniques is the completion of several key research efforts dealing with both microbial and chemical hazards. These efforts include identifying appropriate indicators of wastewater microbial levels; determining the human dose-response relationship for wastewater microbial aerosols; defining the potential for translocation of microbial or chemical contaminants to surface and groundwaters; and evaluating the usability of crops and domestic animals raised at land treatment sites.

Many of the requirements for research on potential chemical hazards should be transferable from ongoing or planned efforts on sludge land treatment. On the other hand, studies of potential hazards due to pathogenic microorganisms must be specific to wastewater because of the great differences in the microbial quality of wastewaters and sludges.

As mentioned earlier, studies on organic contaminants are generally lacking.

WATER SUPPLY SOURCE

At some time in its history, all water resided in the oceans. By evaporation, moisture is transferred from the ocean surface to the atmosphere, where the winds carry the moisture-laden air over land masses (*1*). Under certain conditions, this water vapor condenses to form clouds, which release their moisture as precipitation in the form of rain, hail, sleet, or snow

When rain falls toward the earth, a part may evaporate and return immediately to the atmosphere. Precipitation in excess of the amount that wets a surface or supplies evaporation requirements is available as a potential source of water supply.

Source Water Protection

Concerns over the quality of source water and its impact on public health is as old as recorded history. Hippocrates (460–354 B.C.) the father of medicine, wrote in *Air, Water and Places*—the first treatise on public hygiene—“One should...consider the water which the inhabitants use, whether they be marshy and soft, or hard and running from elevated and rocky situations, and then if salty and unfit for cooking...for water contributes much to health.” Hippocrates’ discussion of the qualities of water centers on the selection of the most health-giving sources of supply rather than on rectifying the waters that were bad (*140*).

More recent history illustrates why Hippocrates’ concerns are justified, as illustrated by the city of Cincinnati, Ohio typhoid statistics between 1905 and 1915. Prior to the installation of water treatment in 1907, both the typhoid death and incidence rates were increasing steadily. In 1906 the typhoid death and incidence rates per 100,000 were 68 and 556, respectively. Both incidence (morbidity) or case rate and death rate (mortality) dropped sharply after filtration began and then dropped again after chlorination was started. The typhoid death rate per 100,000 people dropped from an average of 53 (1900 to 1907) before filtration began in late 1907 to 10 immediately after 1907. This pattern was repeated in virtually every industrialized country in the world, leading to the universal adoption of conventional water treatment systems in most developed countries. Ground water has also proven to be vulnerable to various types of contamination, especially to industrial solvents (*141*).

Passage of the 1996 Amendments to the Safe Drinking Water Act (SDWAA) has focused the attention of water utility managers and public health and regulatory officials on source water protection and its role in protecting public water supplies. There is growing awareness that water treatment and/or disinfection may not always be enough to ensure the provision of potable and safe water to the consumer. The 1993 cryptosporidiosis outbreak in Milwaukee, Wisconsin has raised the possibility that even water suppliers that meet all of the Surface Water Treatment Rule (SWTR) requirements are vulnerable. In the Milwaukee case, more than 400,000 illnesses and as many as 100 deaths were associated with the outbreak. The cause of the outbreak was attributed to a combination of poor treatment operations and a sudden increase in

source water turbidity resulting from a wet weather flow event (142). This and other similar but less dramatic waterborne disease outbreaks have spurred an interest in assessing the occurrence of protozoan cysts in source and treated water.

Most utilities invest a great deal of time, energy, and capital in developing mechanisms for protecting against the impact of sudden changes in influent water quality. Some of these mechanisms include investment in excess capacity and development of emergency procedures. For example, the city of Cincinnati has invested over 50 million dollars in the largest granular-activated carbon treatment facility in the world, with on-site reactivation (143). In addition, Cincinnati has long retention times as part of its pretreatment facility. However, very little research has been conducted on attempting to characterize the impacts that sudden changes in raw water quality can have on finished water quality.

Another event that has focused attention on source water protection is the application for a filtration variance by New York City. This request was based on the Surface Water Treatment Rule (SWTR), which was promulgated under the Safe Drinking Water Act and its Amendments of 1986. The SWTR requires that all surface water sources must be filtered but allows a water purveyor to gain “avoidance of filtration” if the source is of high quality and is adequately protected, if specified turbidity and fecal coliform levels are not exceeded, if adequate disinfection is provided, and if no outbreaks of waterborne illness have occurred (142).

Concern over source water protection is not limited to surface water supplies. Many groundwater supplies have proven to be vulnerable as well, resulting in the various states implementing wellhead protection programs. Based on the 1996 Amendments, the states will have to implement programs to decide if a system’s source of supply is threatened as well as determine the means to prevent pollution. Communities will be allowed to ask for state assistance, and a certain percentage of the State Revolving Loan Fund has been earmarked to assist with source water protection (144).

Water supplies vary greatly in the nature of the source water they use and in the circumstances under which they provide water to their customers. Nevertheless, there are some common elements that are relevant to source water protection in general. For example, land use planning can provide information that is relevant to source water protection. Information on population densities, the ratio of pervious to impervious land cover and the location of point and nonpoint sources of pollution can be relevant to assessing problems associated with both ground and surface source water protection. As part of the Clean Water Act, Comprehensive River Basin Planning was initiated under section 208. A major effort was undertaken to bring to bear the existing art and science of comprehensive planning in river basins with regard to minimizing the impact of point and nonpoint pollution on water quality in streams, lakes, and groundwater. Many of the approaches suggested in those studies are very relevant to the issue of source water protection today.

Water Budget

On the average, about 40,000 bgd ($150 \times 10^9 \text{ m}^3/\text{day}$) of water passes over the United States in the form of water vapor (*l*). Of this, approximately 10% is precipitated as rainfall, snow, sleet, or hail. The remainder continues in atmospheric suspension. Of the 4200 bgd ($16 \times 10^9 \text{ m}^3/\text{day}$) (equivalent to the average rainfall of 30 in (76 cm) falling on the United States), about two-thirds is evaporated immediately from wet surfaces or transpired by vegetation. The remaining 1450 bgd ($5.5 \times 10^9 \text{ m}^3/\text{day}$) accumulates in ground or surface storage; flows to the oceans, the Gulf of Mexico, or across the nation’s boundaries; is consumptively used; or is evaporated from reservoirs. Figure 5.1 illustrates this cycle.

Only part of the potential 1450 bgd ($5.5 \times 10^9 \text{ m}^3/\text{day}$) can be developed for intensive beneficial use. To date, with existing surface storage and because of the extremes of annual precipitation that cause floods and droughts, only 675 bgd ($2.5 \times 10^9 \text{ m}^3/\text{day}$) is considered available in 95 out of 100 years.

Surface water can be available for use as freshwater from the time it strikes the land surface as precipitation until it is discharged into the ocean or mixes with saline water in estuaries, landlocked seas, or saline lakes.

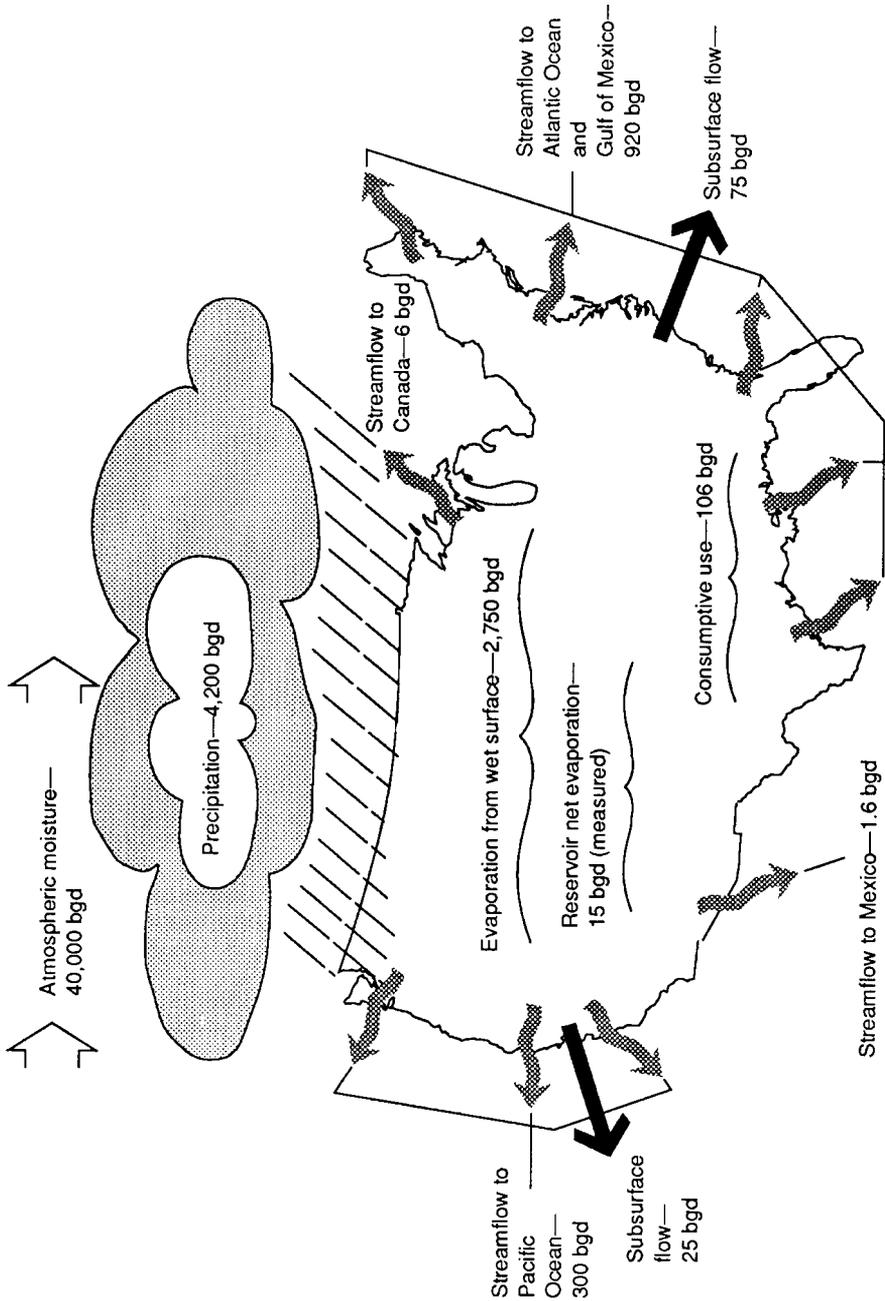


FIGURE 5.1 Water budget of conterminous United States (1).

It is usable only so long as it is not excessively polluted with natural or man-made pollutants. The average annual runoff is sometimes assumed to be the theoretical upper limit of surface supplies. However, the increased storage required to make this quantity of water available for use would be so large that the resulting evaporation from the required reservoirs would substantially decrease the available water. Thus, a mix of increased storage and other alternatives seems to offer the most effective solution. To sustain the desired withdrawals 95% of the time, considerable additional storage is needed, but in many areas neither the reservoir sites nor the uncommitted flows are available. Consequently, if projected water requirements are to be satisfied, there is a strong need for water conservation to save water, weather modification to increase precipitation, and improved methods to reduce evaporation.

Surface water occurs in rivers, streams, lakes, swamps, marshes, and reservoirs. Stream flow varies from region to region, from season to season, and from year to year. For example, within a normal year for a given region, the ratio of maximum streamflow to minimum streamflow can be more than 500 to 1. Annual variations in flows also are substantial. Even in areas of high precipitation and streamflow, a series of dry years sometimes occurs, resulting in serious droughts, such as those in the Northeast in 1961 and 1966, and in the West and other parts of the United States in 1976, 1977, and 1988. Effects of droughts are particularly serious in areas that use a high proportion of the available average runoff or where storage and distribution facilities are inadequate to provide sufficient carryover during prolonged periods of low streamflow.

Variations in annual streamflow are expressed in terms of flow that is expected to be exceeded in a specific percentage of years. For example, a flow of 5% exceedence represents an extremely high flow that would be exceeded only 5 out of 100 years on the average. Many flood plains will be inundated when flow is at or above this level. The 50% exceedence flow is the median flow; streamflow will exceed the median flow half the time. The 80% exceedence flow, on the other hand, represents a low flow, one that will be exceeded 80 years out of 100, on the average. During those years when flows are at 80% exceedence, water shortages varying in intensity can occur. The 95% exceedence represents extreme low flow.

As Figure 5.2 shows, a few of the 5% exceedence (high) flows are twice the mean flow or more, but most are about 1.5 times the mean. At the 95% exceedence (low), flows generally range from 0.4 to 0.7 of the mean flow.

Groundwater

Groundwater storage is considerably in excess of all artificial and natural surface storage. This enormous groundwater reserve sustains the continuing outflow of streams and lakes during prolonged periods that often follow the relatively few runoff-producing rains each year. However, the relation between groundwater and surface storage is one of mutual interdependence. For example, groundwater intercepted by wells as it moves toward a stream represents a real diversion, just as if the water has actually been taken from the stream.

The usable groundwater storage is distributed in quantities determined primarily by precipitation, evapotranspiration, and geologic structure. There are two components to this supply; one, a part of the hydrologic cycle (see Figure 5.3), the other, water trapped underground in past ages, and no longer naturally circulated in the cycle.

In an undeveloped groundwater basin, movement of water to lower basins, seepage from and to surface-water sources, and transpiration are dependent upon the water in storage and the rate of recharge. During periods following abundant rainfall, recharge may exceed discharge. When recharge exceeds discharge, the excess rainfall increases the amount of water available in storage in the groundwater basin. As the water table or artesian pressure rises, the gradients to points of discharge become steeper and outflows increase. When recharge ceases, storage decrease from outflow causes water-table levels and artesian pressures to decline. In most undeveloped basins, the major fluctuations in storage are seasonal, with the mean annual elevation of water levels showing little variation. Thus, the average annual inflow to storage equals the average annual outflow, a quantity of water referred to as the basin yield.

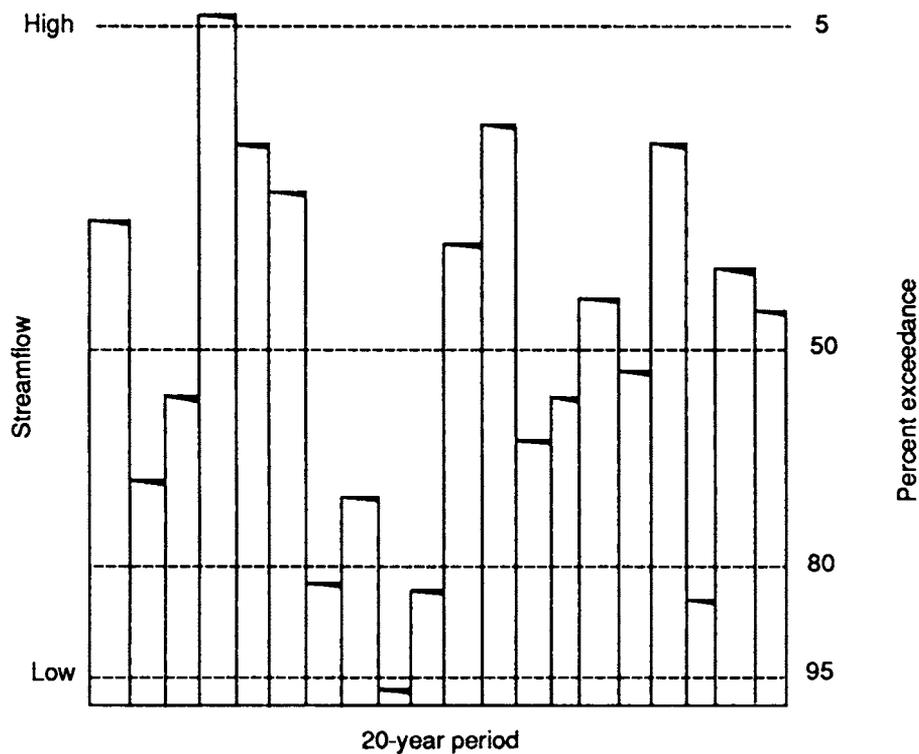


FIGURE 5.2 Hypothetical example of streamflow frequency (1).

Groundwater distribution may be generally categorized into zones of aeration and saturation. The saturated zone is one in which all the voids are filled with water under hydrostatic pressure. The aeration zone, in which the interstices are filled partly with air and partly with water, may be subdivided into several subzones. These are classified as follows (12):

1. *Soil-water zone*: This begins at the ground surface and extends downward through the major root zone. Its total depth is variable and dependent upon soil type and vegetation. The zone is unsaturated except during periods of heavy infiltration. Three categories of water classification may be encountered in this region: hygroscopic water, which is adsorbed from the air; capillary water, which is held by surface tension; and gravitational water, which is excess soil water draining through the soil.
2. *Intermediate zone*: This zone extends from the bottom of the soil-water zone to the top of the capillary fringe and may vary from nonexistence to several hundred feet in thickness. The zone is essentially a connecting link between the near-ground-surface region and the near-water-table region, through which infiltrating waters must pass.
3. *Capillary zone*: This zone extends from the water table to a height determined by the capillary rise that can be generated in the soil. The capillary-zone thickness is a function of soil texture and may vary not only from region to region but also within a local area.

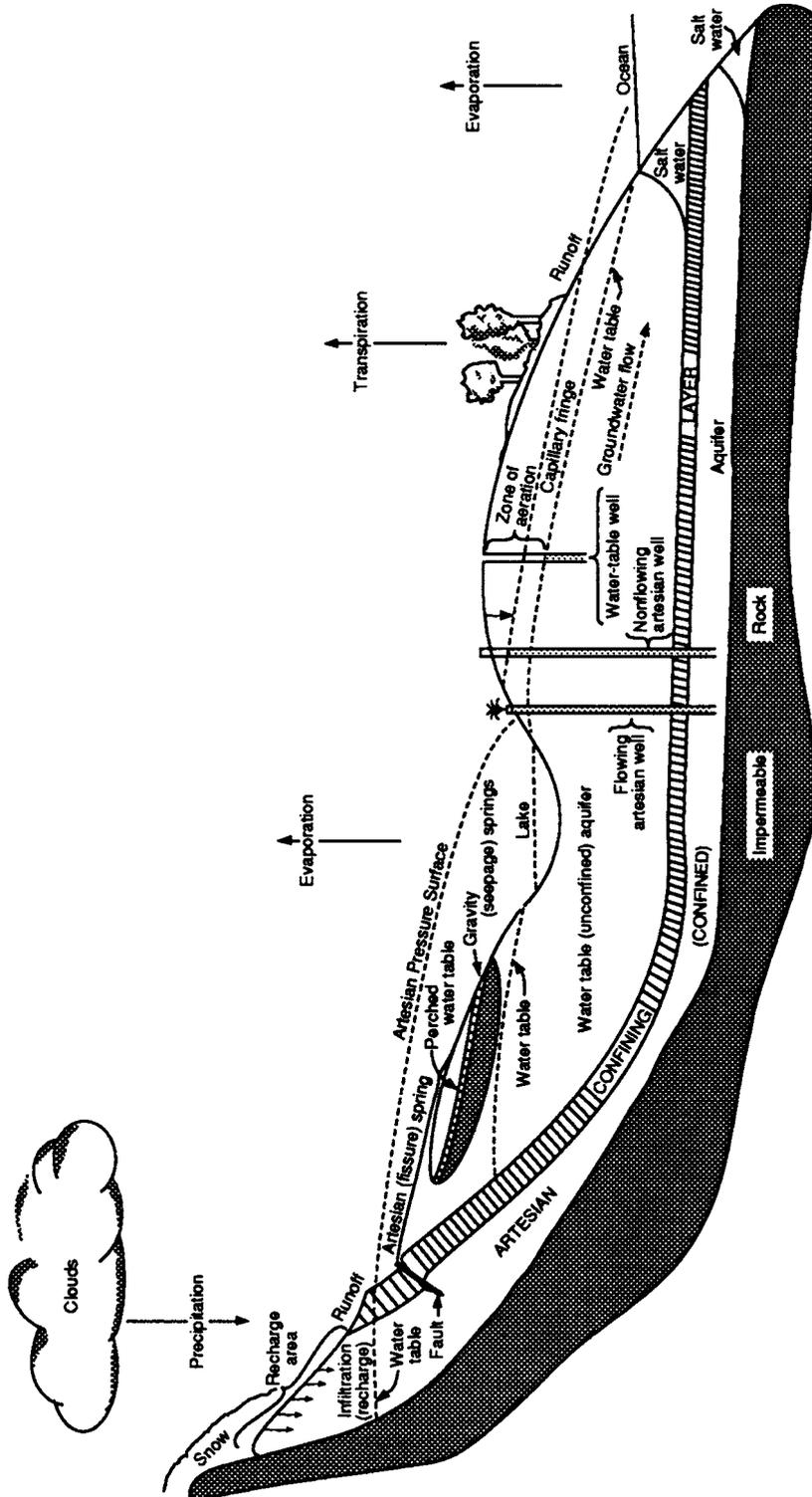


Figure 5.3 The hydrologic cycle (2)

Water that drains downward below the root zone finally reaches a level at which all of the openings or voids in the earth's materials are filled with water. This zone is known as the zone of saturation. Water in the *zone of saturation* is referred to as *groundwater*. The upper surface of the zone of saturation, if not confined by impermeable material, is called the *water table*. When an overlying impermeable formation confines the water in the zone of saturation under a pressure greater than atmospheric pressure, the groundwater is under artesian pressure. The name "artesian" comes from the ancient province of Artesium in France, where, in the day of the Romans, water flowed to the surface of the ground from a well. Not all water from wells that penetrate artesian formations flows above the surface of the land. For a well to be artesian, the water in the well must stand above the top of the aquifer. An aquifer, or water-bearing formation, is an underground layer of permeable rock or soil that permits the passage of water.

Because of the irregularities in underground deposits and in surface topography, the water table occasionally intersects the surface of the ground or the bed of a stream, lake, or ocean. As a result, groundwater moves to these locations and out of the aquifer or groundwater reservoir. Thus, groundwater is continually moving within the aquifer even though the movement may be 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. At any point the slope is a reflection of the rate of flow and resistance to movement of water through the saturated formation. Seasonal variations in the supply of water to the underground reservoir cause considerable changes in the elevation and slope of the water table and artesian pressure level.

Wells. A well can be used to extract water from the groundwater reservoir. Pumping will cause a lowering of the water table near the well. If pumping continues at a rate that exceeds the rate at which the water may be replaced by the water-bearing formations, the sustained yield of the well is exceeded. 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 "safe yield" of the aquifer is exceeded. Under these conditions, saltwater encroachment may occur where wells are located near the seashore or other surface or underground saline waters.

Springs. An opening in the ground surface from which groundwater flows is a spring. Water may flow by force of gravity (from water-table aquifers), or be forced out by artesian pressure. The flow from a spring may vary considerably. When the water-table or artesian pressure fluctuates, so does the flow of springs.

Groundwater may become surface water at springs or at intersections of a water body and a water table. During extended dry periods, streamflows consist largely of water from the groundwater reservoir. As the groundwater reservoir is drained by the surface stream, the flow will reach a minimum or may cease altogether. It is important in evaluating stream and spring supplies to consider seasonal fluctuations in flow.

Surface Water

Precipitation that does not enter the ground through infiltration or is not returned to the atmosphere by evaporation flows over the ground surface and is classified as direct runoff. Direct runoff is water that moves over saturated or impermeable surfaces, and in stream channels or other natural or artificial storage sites. The dry weather (base) flow of streams is derived from groundwater or snow melt.

In some areas, a source of water for individual development is the rainfall intercepted by roof surfaces on homes, barns, or other buildings. Water from such impermeable surfaces can be collected and stored in tanks called cisterns. In some instances, natural ground surfaces can be conditioned to make them impermeable. This conditioning will increase runoff to cisterns or large artificial storage reservoirs, thereby reducing loss by infiltration into the ground.

Runoff from ground surfaces may be collected in either natural or artificial reservoirs. A portion of the

water stored in surface reservoirs is lost by evaporation and from infiltration to the groundwater table from the pond bottom. Transpiration from vegetation in and adjacent to ponds constitutes another means of water loss.

Snow

Much of the snow falling on a watershed is kept in storage on the ground surface until temperatures rise above freezing. In the mountainous areas of the western United States, snow storage is an important source of water supply through much of the normal irrigation season. Measures taken to increase the snowpack and reduce the melt rate are usually beneficial to individual water supply systems in these areas.

Saline Waters

Saline waters include (1) undiluted seawater containing about 35,000 mg/L total dissolved solids (TDS) and (2) brackish water containing from 1000 mg/L to about 10,000 to 15,000 mg/L TDS, and are largely represented by highly mineralized groundwaters and diluted seawater.

Seawater. Seawater is available at sea level and at the seacoast. It is important as a major water resource because of the size and importance of coastal communities that can be served by converted seawater (diluted or undiluted) and because of the inexhaustible quantities available.

Brackish Water. Sources of brackish water are less obvious and less well known than those of seawater. Brackish waters are widely distributed over the land areas of the North American and other continents. Because of their lower mineral content, brackish or highly mineralized waters are, in most cases, less expensive to treat than seawater. Brackish water is found underground as well as in estuaries, rivers, lakes, and certain wastewaters (12). These sources of brackish water are discussed in more detail below.

Groundwater. The most widely available and, in many ways, the most desirable source of brackish water is brackish groundwater. Brackish groundwater reserves are found in many parts of the world, including the United States, Canada, Mexico, southern and western Europe, North Africa, the Middle East, Australia, western Africa, and South America. Well over half the land area of the continental United States is underlaid by saline waters having total dissolved solids content in the 1000 to 3000 mg/L range.

Estuaries. Estuaries of some major rivers extend inland for many miles and represent major sources of brackish water (13). Practical salt-removal processes give water engineers a new degree of freedom in dealing with estuarial water problems. In the past, salt had to be avoided at all costs. When, in an emergency, salt appeared, one either had to “live with it,” shut down, or move upstream. In designing new facilities, one had to go far enough upstream to avoid salt under the worst rainfall or streamflow conditions one could envision with the records and methods then available. Now salt can be considered as a pollutant that can be removed.

Rivers and Lakes. A few major rivers and large lakes are saline or brackish in nature. Some examples in the United States are the Arkansas, Red, Pecos, and Colorado rivers, as well as Lake Texoma on the Red River. Attempts are being made to reduce the salinity of some of these streams by control of natural salt pollution sources (14). These rivers have large flows of water and could benefit major areas surrounding them. The Arkansas River, for example, runs through or near such cities as Wichita, Kansas; Tulsa, Oklahoma; and Little Rock, Arkansas. It is now possible to compare the cost of the proposed natural pollution control measures with the cost of removing the salt at points of local withdrawals for municipal or other beneficial uses.

Wastewaters. Three of the major beneficial uses of water result in substantial increases in its mineral content—namely irrigation, industrial cooling by evaporation, and municipal use (15). In many parts of the

southwestern United states, the drainage water from irrigation projects will have on the order of double to triple the mineral content of the raw water initially applied. For example, the New and Alamo rivers in California, which are formed by drainage from irrigation projects in the Imperial Valley using Colorado River water, have TDS content in the neighborhood of 2000 to 3000 mg/L, whereas the raw water from the Colorado River has 600 to 900 mg/L TDS.

The wastewater from municipal sources will usually show an increase of 300 to 400 mg/L in TDS content above that of the public water supply of the community. Thus, the TDS in the sewage effluent from such cities as Los Angeles and San Diego reflects both the relatively high TDS content (300 to 900 mg/L) of their “fresh” water sources and the 300 to 400 mg/L increment from municipal use.

GROUNDWATER PRODUCTION

Providing groundwater as drinking water simply starts with an understanding of aquifers and the capability to meet expected demands. The development of a groundwater source is discussed, with emphasis on well systems.

Aquifers

An aquifer is a water-bearing stratum or formation capable of transmitting water in quantities sufficient to permit development. Aquifers may be considered as falling into two categories, confined and unconfined, depending upon whether or not a water table or free surface exists under atmospheric pressure. The storage volume within an aquifer is changed whenever water is recharged to or discharged from an aquifer. In the case of an unconfined aquifer, this may be easily determined as

$$S = (S_y)(\Delta V) \quad (5.1)$$

where S = change in storage volume

S_y = average specific yield of the aquifer

ΔV = volume of the aquifer lying between the original water table and the water table at a later specified time

For saturated, confined aquifers, pressure changes produce only slight changes in storage volume. In this case, the weight of the overburden is supported partly by hydrostatic pressure and partly by the solid material in the aquifer. When the hydrostatic pressure in a confined aquifer is reduced by pumping or other means, the load on the aquifer increases, causing its compression, with the result that some water is forced from it. Decreasing the hydrostatic pressure also causes a small expansion, which in turn produces an additional release of water. For confined aquifers, the water yield is expressed in terms of a storage coefficient S_c . This storage coefficient can be defined as the volume of water that an aquifer takes in or releases per unit surface area of aquifer per unit change in load normal to the surface. Figure 5.4 illustrates the classifications of aquifers (16).

In addition to water-bearing strata exhibiting satisfactory rates of yield, there are also non-water-bearing and impermeable strata. An aquiclude is an impermeable stratum that may contain large quantities of water but whose transmission rates are not high enough to permit effective development. An aquifuge is a formation that is impermeable and devoid of water.

Level Fluctuations. Any circumstance that alters the pressure imposed on underground water will also cause a variation in the groundwater level. Seasonal factors, change in stream and river stages,

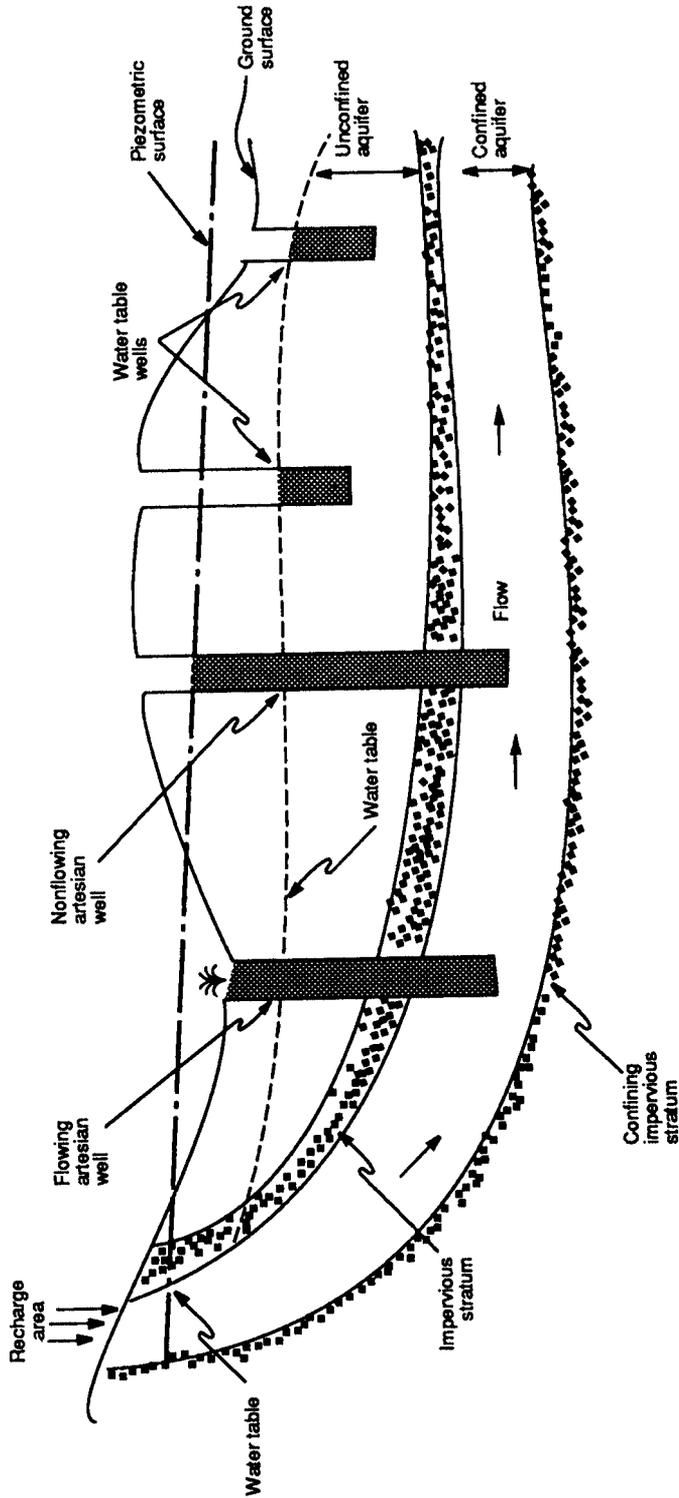


Figure 5.4 Confined and unconfined aquifers.

evapotranspiration, atmospheric pressure changes, winds, tides, external loads, various forms of withdrawal and recharge, and earthquakes all may produce fluctuations in the level of the water table or the piezometric surface, depending upon whether the aquifer is free or confined (17). It is important that the engineer concerned with the development and utilization of groundwater supplies be aware of these factors. He should also be able to evaluate their importance relative to the operation of a specific groundwater basin.

Hydraulic Characteristics. The hydraulic characteristics of an aquifer (which are described by the storage coefficient and the aquifer permeability) may be determined by laboratory or field tests. The three most commonly used field methods are the application of tracers, use of field permeameters, and aquifer performance tests (17). A discussion of aquifer performance tests will be given here along with the development of flow equations for wells (18–20).

Aquifer performance tests may be classified as either equilibrium or nonequilibrium tests. In the former, the cone of depression must be stabilized for the flow equation to be derived. In the latter, the derivation includes the condition that steady-state conditions have not been reached. Thiem published the first performance tests based on equilibrium conditions in 1906 (21).

The equilibrium equation for an unconfined aquifer can be derived using the notation of Figure 5.5. In this case, the flow is assumed as radial, the original water table to be horizontal, the well to fully penetrate the aquifer of infinite areal extent, and steady-state conditions must prevail. Then the flow toward the well at any location x from the well must equal the product of the cylindrical element of area at that section and the flow velocity. Using Darcy's law, this becomes

$$Q = 2\pi xy K_y dy/dx \tag{5.2}$$

where $2\pi xy$ = area at any section, ft²

$K_y dy/dx$ = flow velocity, ft/s

Q = discharge, ft³/s

Integrating over the limits specified below yields

$$\int_{r_1}^{r_2} Q \left(\frac{dx}{x} \right) = 2\pi K_f \int_{h_1}^{h_2} y dy \tag{5.3}$$

$$Q \ln \left(\frac{r_2}{r_1} \right) = \frac{2\pi K_f (h_2^2 - h_1^2)}{2} \tag{5.4}$$

and

$$Q = \frac{\pi K_f (h_2^2 - h_1^2)}{\ln(r_2/r_1)} \tag{5.5}$$

This equation can then be solved for K_f , yielding

$$K_f = \left[\frac{1005 Q \ln(r_2/r_1)}{h_2^2 - h_1^2} \right] \tag{5.6}$$

where \ln has been converted to \log_{10} , K_f is in gallons per day per square foot, Q is in gallons per minute, and r and h are measured in feet. If the drawdown is small compared with the total aquifer thickness, an approximate formula for the discharge of the pumped well can be inserting h_w for h_1 and the height of the aquifer for h_2 in Eq. (5.5).

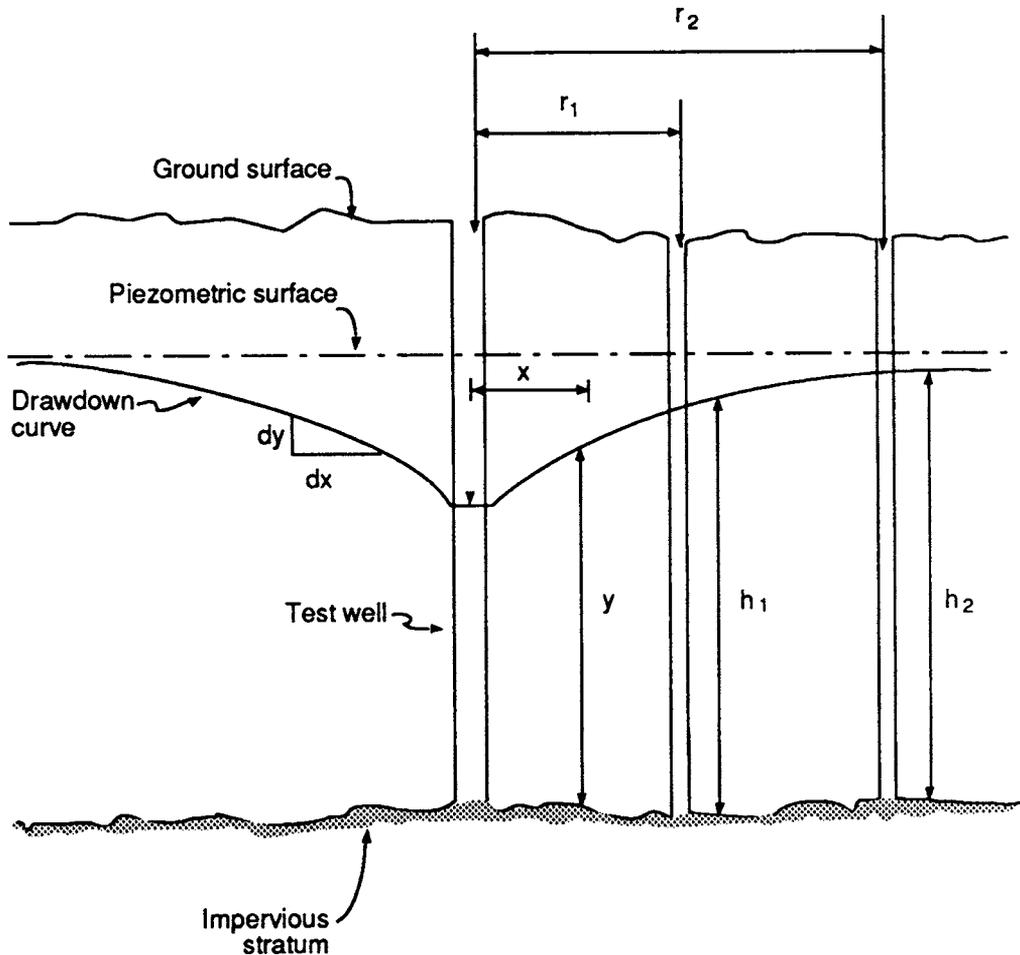


FIGURE 5.5 Radial flow to a well in an unconfined aquifer.

The basic equilibrium equation for a confined aquifer can be obtained in a similar manner, using the notation of Figure 5.6. The same assumptions apply. Mathematically the flow in cubic feet per second may be determined as follows:

$$Q = 2\pi xmK_f \frac{dy}{dx} \quad (5.7)$$

Integrating, we obtain

$$Q = 2\pi K_f \frac{h_2 - h_1}{\ln(r_2/r_1)} \quad (5.8)$$

where m = aquifer thickness.

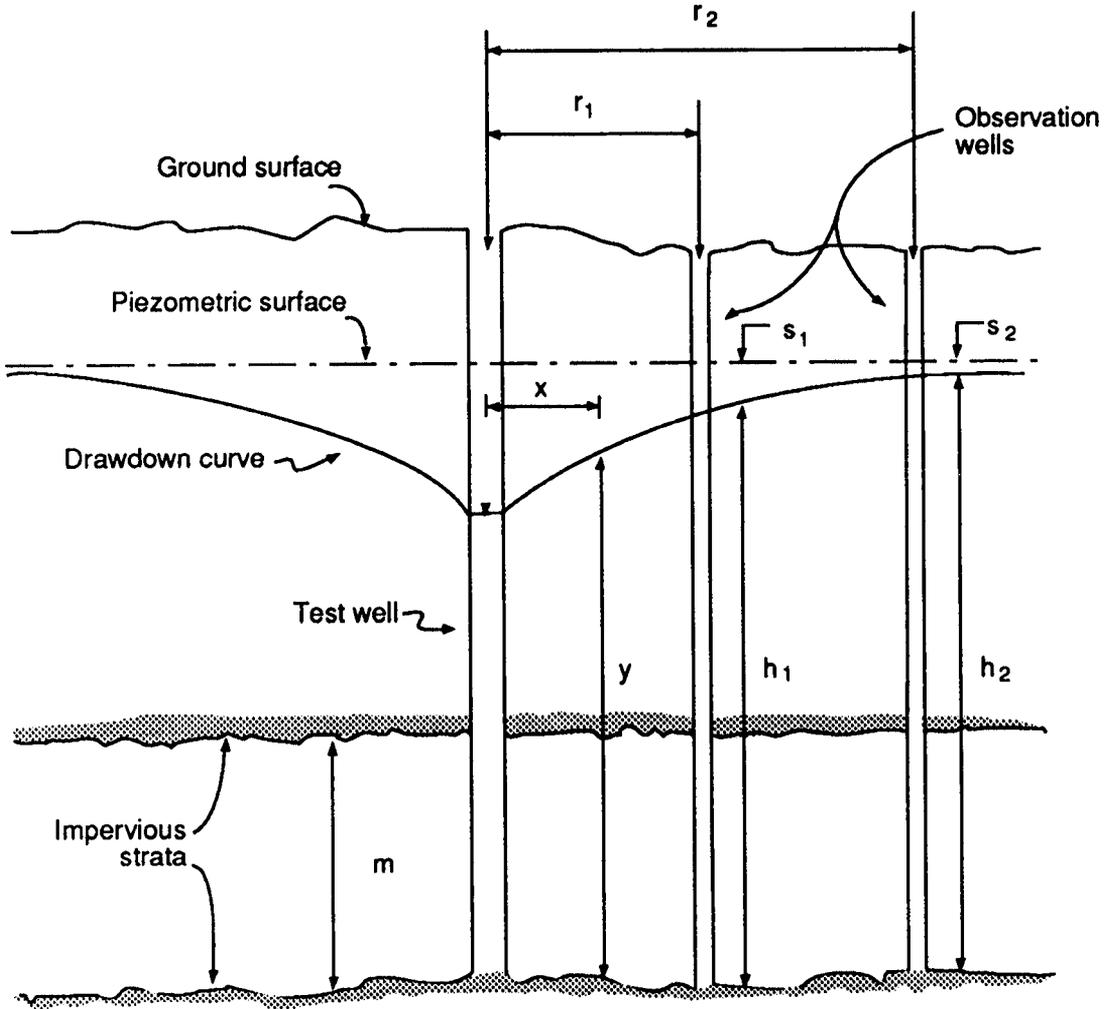


FIGURE 5.6 Radial flow to a well in an confined aquifer.

The coefficient of permeability may be determined by rearranging Eq. (5.8) to the form

$$Q = 2\pi xmK_f \frac{dy}{dx} \tag{5.9}$$

where Q is in gallons per minute, K_f is the permeability in gallons per day per square foot, and r and h are measured in feet.

Safe Yield. The development engineer must be able to determine the quantity of water that can be produced from a groundwater basin in a given period of time. He or she must also be able to evaluate the consequences

that will result from the imposition of various drafts on the underground supply. A knowledge of the safe yield of an aquifer is therefore exceedingly important. Following are pertinent definitions:

1. *Safe yield*: The quantity of water that can be withdrawn annually without the ultimate depletion of the aquifer.
2. *Maximum sustained yield*: Maximum rate at which water can be withdrawn on a continuing basis from a given source.
3. *Permissive sustained yield*: Maximum rate at which withdrawals can be made legally and economically on a continuing basis for beneficial use without the development of undesired results.
4. *Maximum mining yield*: Total storage volume in a given source that can be withdrawn and used.
5. *Permissive mining yield*: Maximum volume of water that can be withdrawn legally and economically, to be used for beneficial purposes, without causing an undesired result.

Methods for determining the safe yield of an aquifer have been proposed by Hill, Harding, Simpson, and others (17). The Hill method, based on groundwater studies in Southern California and Arizona, will be presented here. In this method, the annual change in groundwater table elevation or piezometric surface elevation is plotted against the annual draft. The points can be fitted by a straight line, provided that the water supply to the basin is fairly uniform. That draft that corresponds to zero change in elevation is considered to be the safe yield. The period of record should be such that the supply during this period approximates the long-time average supply. Even though the draft during the period of record may be an overdraft, the safe yield can be determined by extending the line of best fit to an intersection with the zero change in elevation line. An example of this procedure is given in Figure 5.7.

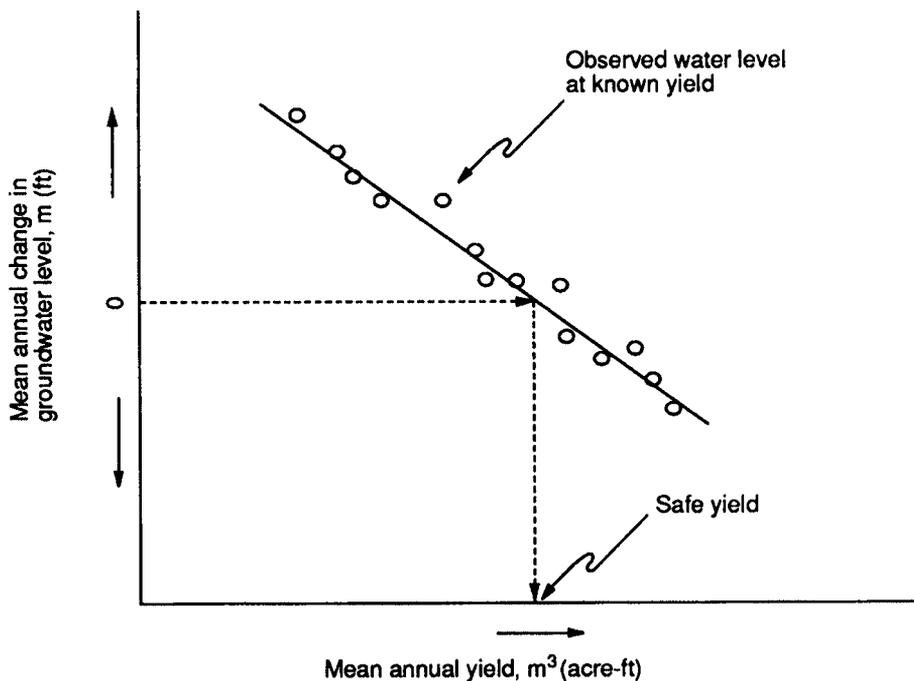


FIGURE 5.7 Graphical determination of safe yield by the Hill method.

It should be noted that the safe yield of a groundwater basin may be variable with time. This is because the groundwater basin conditions under which the safe yield was determined are subject to change, and these changes will be reflected in the modified value of the safe yield.

Source Contamination. All groundwater sources should be located a safe distance from sources of contamination. In cases where sources are severely limited, however, a groundwater aquifer that might become contaminated may be considered for a water supply if treatment is provided. After a decision has been made to locate a water source in an area, it is necessary to determine the distance the source should be placed from the origin of contamination and the direction of water movement. Table 5.12 is offered as a guide in determining these distances.

Groundwater Development

The type of groundwater development to be undertaken is dependent upon the geological formations and hydrological characteristics of the water-bearing formation. The development of groundwater falls into two main categories:

1. Development by wells
 - a. Nonartesian or water table
 - b. Artesian
2. Development from springs
 - a. Gravity
 - b. Artesian

Wells. Nonartesian wells are those that penetrate formations in which groundwater is found under water-table conditions. Pumping from the well lowers the water table in the vicinity of the well and water moves toward the well under the pressure differences thus artificially created.

Artesian wells are those that penetrate aquifers in which the groundwater is found under hydrostatic pressure. Such a condition occurs in an aquifer that is confined beneath an impermeable layer of material at an elevation lower than that of the intake area of the aquifer. The intake areas or recharge areas of confined aquifers are commonly at high-level surface outcrops of the formations. Groundwater flow occurs from high-level outcrop areas to low-level outcrop areas, which are areas of natural discharge. It also flows toward

TABLE 5.12 Distance to Sources of Contamination (2)

Formations	Minimum acceptable distance from well to source of contamination
Favorable (unconsolidated)	50 ft; lesser distances only on health department approval following comprehensive sanitary survey of proposed site and immediate surroundings.
Unknown	50 ft only after comprehensive geological survey of the site and its surroundings has established, to the satisfaction of the health agency, that favorable formations do exist.
Poor (consolidated)	Safe distances can be established only following both the comprehensive geological and comprehensive sanitary surveys. These surveys also permit determining the direction in which a well may be located with respect to sources of contamination. In no case should the acceptable distance be less than 50 ft.

points where water levels are lowered artificially by pumping from wells. When the water level in the well stands above the top of the aquifer, the well is described as artesian. A well that yields water by artesian pressure at the ground surface is a flowing artesian well.

Springs. Gravity springs occur where water percolating laterally through permeable material overlying an impermeable stratum comes to the surface. They also occur where the land surface intersects the water table. This type of spring is particularly sensitive to seasonal fluctuations in groundwater storage and frequently dwindles to a seep or disappears during dry periods. Gravity springs are characteristically low-discharge sources, but when properly developed, they make satisfactory individual water supply systems.

Artesian springs discharge from artesian aquifers. They may occur where the confining formation over the artesian aquifer is ruptured by a fault or where the aquifer discharges to a lower topographic area. The flow from these springs depends on the difference in recharge and discharge elevations of the aquifer and on the size of the openings transmitting the water. Artesian springs are usually more dependable than gravity springs, but they are particularly sensitive to the pumping of wells developed in the same aquifer. As a consequence, artesian springs may be dried by pumping.

Springs may be further classified by the nature of the passages through which water issues from the source.

Seepage springs are those in which the water seeps out of sand, gravel, or other material that contains many small interstices. The term as used here includes many large springs as well as small ones. Some of the large springs have extensive seepage areas and are usually marked by the presence of abundant vegetation. The water of small seepage springs may be colored or carry an oily scum because of decomposition of organic matter or the presence of iron. Seepage springs may emerge along the top of an impermeable bed, but they occur more commonly where valleys are cut into the zone of saturation of water-bearing deposits. These springs are generally free from harmful bacteria, but they are susceptible to contamination by surface runoff which collects in valleys or depressions.

Tubular springs issue from relatively large channels, such as the solution channels and caverns of limestone, and soluble rocks and smaller channels that occur in glacial drift. They are sometimes referred to as “bold” springs because the water issues freely from one or more large openings. When the water reaches the channels by percolation through sand or other fine-grained material, it is usually free from contamination. When the channels receive surface water directly or receive the indirect effluent of cesspools, privies, or septic tanks, the water must be regarded as unsafe.

Fissure springs issue along bedding, joint, cleavage, or fault planes. Their distinguishing feature is a break in the rocks along which the water passes. Some of these springs discharge uncontaminated water of deep-source origin. A large number of thermal springs are of this type. Fissure springs, however, may discharge water which is contaminated by surface drainage from strata close to the surface (22).

Flow. The rate of movement of water through the ground is of an entirely different magnitude than that through natural or artificial channels or conduits (16). Typical values range from 5 ft/day (1.5 in/day) to a few feet per year. Methods for determining these transmission rates are primarily based on the principles of fluid flow represented by Darcy’s law. Mathematically, this law can be stated as

$$V = kS \quad (5.10)$$

where V = velocity flow

k = coefficient having the same units as velocity

S = slope of the hydraulic gradient

Darcy’s law is limited in its applicability to flows in the laminar region. The controlling criterion is the Reynolds number.

$$N_r = \frac{Vd}{\nu} \quad (5.11)$$

where N_r = Reynolds number
 d = mean grain diameter
 ν = kinematic viscosity
 V = flow velocity

For Reynolds numbers of less than 1 unit, groundwater flow may be considered laminar. Departure from laminar conditions develops normally in the range of Reynolds numbers from 1 to 10, depending on grain size and shape. Under most conditions, with the exception of regions in close proximity to collecting devices, the flow of groundwater is laminar and Darcy's law applies.

Safeguards. Some of the safeguards for groundwater development are: (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; and (4) prevention of backflow into intakes.

Well Development

The collection of groundwater is accomplished primarily through the construction of wells or infiltration galleries. Numerous factors are involved in the quantitative estimation of the performance of these collection works. Some cases are amenable to solution through the utilization of relatively simple mathematical expressions. Other cases can be solved only through graphical analysis or the use of various kinds of models (17). A more thorough treatment of groundwater and seepage problems may be found in numerous references (17, 23, 24).

A well system may be considered to be composed of three elements: the well structure, the pump, and the discharge piping. The well itself contains an open section through which flow enters and a casing through which the flow is transported to the ground surface. The open section is usually a perforated casing or a slotted metal screen which permits the flow to enter and at the same time prevents collapse of the hole. Occasionally gravel is placed at the bottom of the well casing around the screen.

When a well is pumped, water is removed from the aquifer immediately adjacent to the screen. Flow then becomes established at locations some distance from the well in order to replenish this withdrawal. Owing to the resistance to flow offered by the soil, a head loss is encountered and the piezometric surface adjacent to the well is depressed. This is known as the cone of depression (Figure 5.5). The cone of depression spreads until a condition of equilibrium is reached and steady-state conditions are established.

When a well is pumped, the level of the water table in the vicinity of the well will be lowered (2). This lowering or "drawdown" causes the water table or artesian pressure surface, depending on the type of aquifer, to take the shape of an inverted cone called a cone of depression. This cone, with the well at the apex, is measured in terms of the difference between the static water level and the pumping level. At increasing distances from the well, the drawdown decreases until the slope of the cone merges with the static water table. The distance from the well at which this occurs is called the radius of influence. The radius of influence is not constant but tends to continuously expand with continued pumping. At a given pumping rate, the shape of the cone of depression depends on the characteristics of the water-bearing formation. Shallow and wide cones will form in highly permeable aquifers composed of coarse sands or gravel. Steeper and narrower cones will form in less-permeable aquifers. As the pumping rate increases, the drawdown increases and consequently the slope of the cone steepens.

The characteristics of the aquifer, whether artesian or water table, and the physical characteristics of

the formation which will affect the shape of the cone include thickness, lateral extent, and the size and grading of sands or gravels. In a material of low permeability, such as fine sand or sandy clay, the drawdown will be greater and the radius of influence less than for the same pumpage from very coarse gravel (see Figure 5.8).

For example, when other conditions are equal for two wells, it may be expected that pumping costs for the same pumping rate will be higher for the well surrounded by material of lower permeability because of the greater drawdown.

When the cones of depression overlap, the local water table will be lowered (see Figure 5.8). This requires additional pumping lifts to obtain water from the interior portion of the group of wells. In addition, a wider distribution of the wells over the groundwater basin will reduce the cost of pumping and will allow the development of a larger quantity of water.

Yield. The amount of water that can be pumped from any well depends on the character of the aquifer and the construction of the well.

Doubling the diameter of a well increases its yield only about 10%. The casing diameter should be chosen to provide enough room for proper installation of the pump. Individual wells seldom require casings larger than 15 cm (6 in); 4-in (10-cm) wells are common in many areas.

A more effective way of increasing well capacity is by drilling deeper into the aquifer—assuming, of course, that the aquifer has the necessary thickness. The inlet portion of the well structure (screen, perforations, slots) is also important in determining the yield of a well in a sand or gravel formation. The amount of “open area” in the screened or perforated portion exposed to the aquifer is critical. Wells completed in consolidated formations are usually of open-hole construction, i.e., there is no casing in the aquifer itself.

It is not always possible to predict accurately the yield of a given well before its completion. Knowledge can be gained, however, from studying the geology of the area and interpreting the results obtained from other wells constructed in the vicinity. This information will be helpful in selecting the location and type of well most likely to be successful. The information can also provide an indication of the quantity or yield to expect.

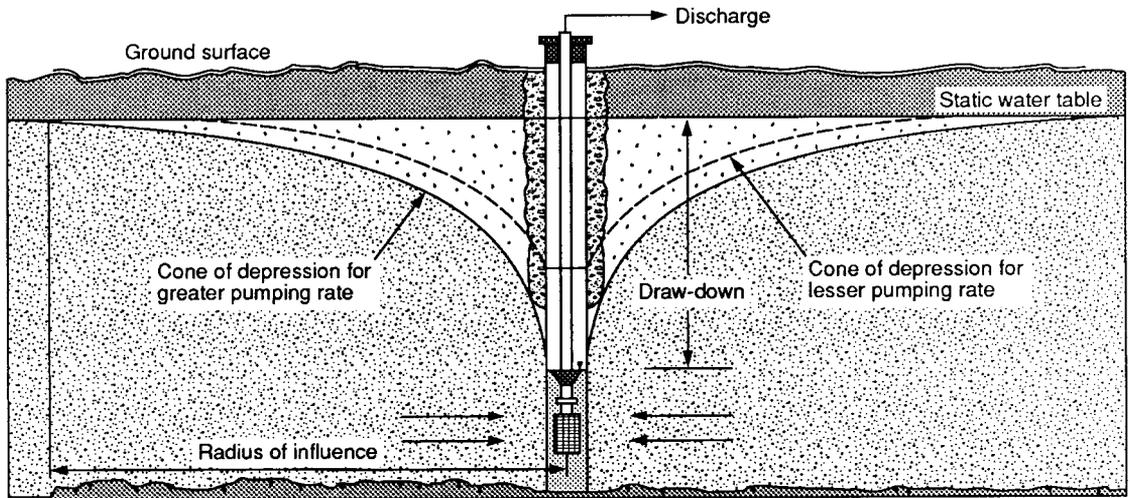
A common way to describe the yield of a well is to express its discharge capacity in relation to its drawdown. This relationship is called the specific capacity of the well and is expressed in gallons per minute (gpm) per foot (cubic meters per minute per meter) of drawdown. The specific capacity may range from less than 1 gpm/ft (0.004 m³/min-m) of drawdown for a poorly developed well or one in a tight aquifer to more than 100 gpm/ft (0.38 m³/min-m) of drawdown for a properly developed well in a highly permeable aquifer.

Table 5.13 gives general information on the practicality of penetrating various types of geologic formations by the methods indicated.

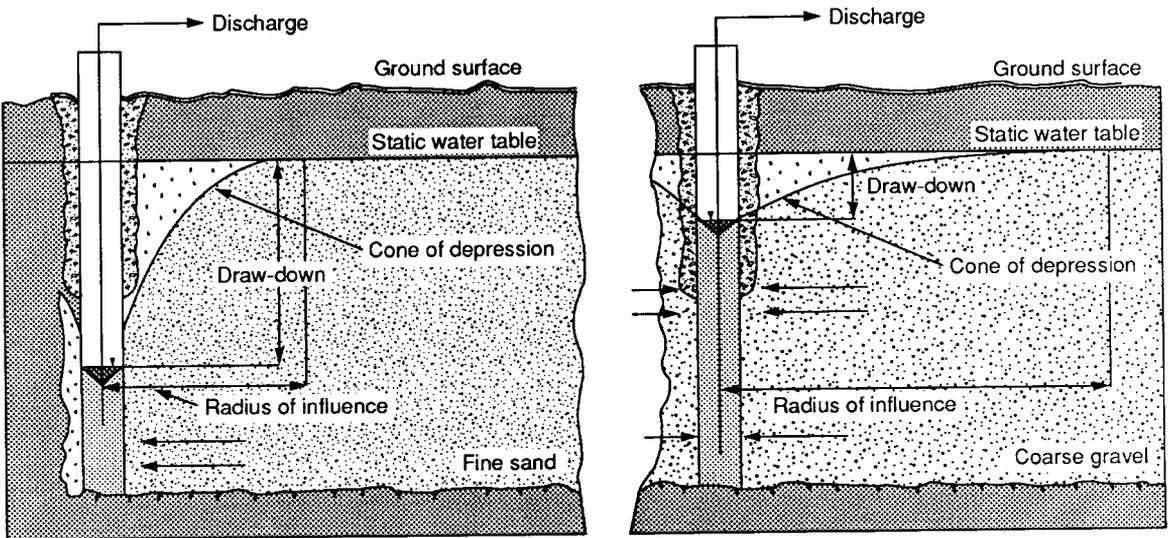
Site Preparation. A properly constructed well should exclude surface water from a groundwater source to the same degree as does the undisturbed overlying geologic formation. The top of the well must be constructed so that no foreign matter or surface water can enter. The well site should be properly drained and adequately protected against erosion, flooding, and damage or contamination from animals. Surface drainage should be diverted away from the well.

The pumping equipment for either power-driven or manual systems should be so constructed and installed as to prevent the entrance of contamination or objectionable material either into the well or into the water that is being pumped. The following factors should be considered.

1. Designing the pump head or enclosure so as to prevent pollution of the water by lubricants or other maintenance materials used during operation of the equipment. Pollution from hand contact, dust, rain, birds, flies, rodents, or animals, and similar sources should be prevented from reaching the water chamber of the pump or the source of supply.
2. Designing the pump base or enclosure so as to facilitate the installation of a sanitary well seal within the well cover or casing.

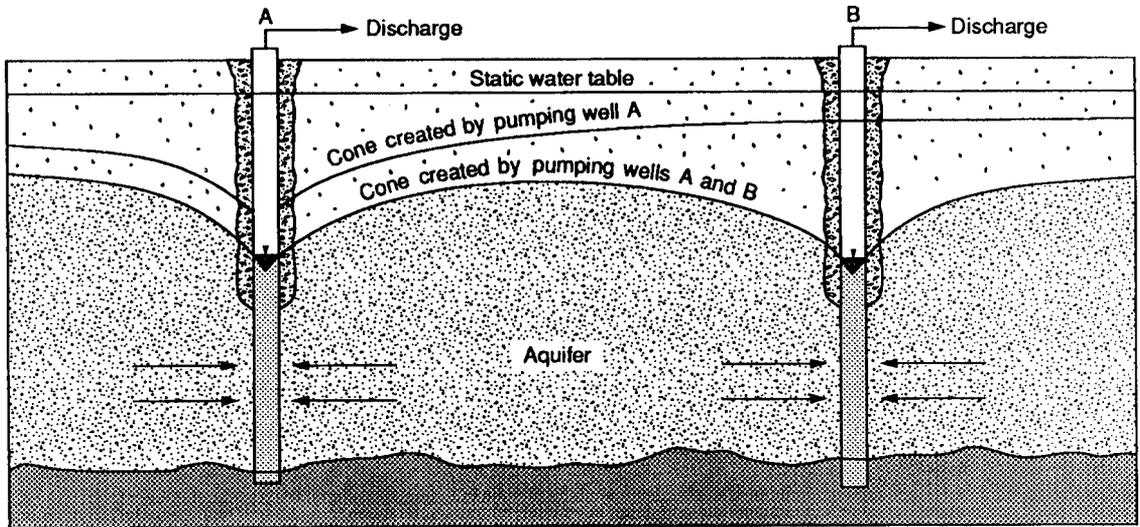


(a)



(b)

FIGURE 5.8 Pumping on aquifers (a) Effect of pumping on cone of depression (2). (b) Effect of aquifer material on cone of depression (2).



(c)

FIGURE 5.8 (continued) (c) Effect of overlapping field on influence pumped wells (2).

3. Installation of the pumping portion of the assembly near or below the static water level in the well so that priming will not be necessary.
4. Designing for frost protection, including pump drainage within the well when necessary.
5. Overall design consideration so as to best facilitate necessary maintenance and repair, including overhead clearance for removing the drop pipe and other accessories.

Well Construction. The following section describes the various types of well construction.

Dug Wells. The dug well, constructed by hand, is usually shallow. It is more difficult to protect from contamination, although if finished properly it may provide a satisfactory supply. Because of advantages offered by other types of wells, consideration should first be given to one of those described in this section.

Dug wells are usually excavated with pick and shovel. The excavated material can be lifted to the surface by a bucket attached to a windlass or hoist. A power-operated clamshell or orange-peel bucket may be used in holes greater than 3 ft (0.91 m) in diameter where the material is principally gravel or sand. In dense clays or cemented materials, pneumatic hand tools are effective means of excavation.

To prevent the native material from caving, one must place a crib or lining in the excavation and move it downward as the pit is deepened. The space between the lining and the undisturbed embankment should be backfilled with clean material. In the region of water-bearing formations, the backfilled material should be sand or gravel. Cement grout should be placed to a depth of 10 ft (3 m) below the ground surface to prevent entrance of surface water along the well lining (see Figure 5.9).

Dug wells may be lined with brick, stone, or concrete, depending on the availability of materials and the cost of labor. Precast concrete pipe, available in a wide range of sizes, provides an excellent lining. This lining can be used as a crib as the pit is deepened. When the lining is to be used as a crib, concrete pipe with tongue-and-groove joints and smooth exterior surface is preferred (see Figure 5.9).

Bored Wells. In general, bored wells have the same characteristics as dug wells, but they may be extended deeper into the water-bearing formation. Bored wells are commonly constructed with earth augers turned

TABLE 5.13 Suitability of Well Construction Methods to Different Condition* (2)

Characteristics	Drilled								
	Dug	Bored	Driven	Percussion	Rotary				Jetted
					Hydraulic	Air	Air	Air	
Range of practical depths (general order of magnitude), ft (in)	0-50 (0-15.24)	0-100 (0-30.48)	0-50 (0-15.24)	0-1000 (0-304.80)	0-1000 (0-304.80)	0-750 (0-228.60)	0-750 (0-228.60)	0-750 (0-228.60)	0-100 (0-30)
Diameter, cm (in)	3-20 (7.62-50.80)	2-30 (5.08-76.20)	1¼-2 (3.18-5.08)	4-18 (10.16-45.72)	4-24 (10.16-60.96)	4-10 (10.16-25.40)	4-10 (10.16-25.40)	4-10 (10.16-25.40)	2-12 (5-30)
Type of geologic formation									
Clay	Yes	Yes	Yes	Yes	Yes	No	Yes	No	Yes
Silt	Yes	Yes	Yes	Yes	Yes	No	Yes	No	Yes
Sand	Yes	Yes	Yes	Yes	Yes	No	Yes	No	Yes
Gravel	Yes	Yes	Yes	Yes	Yes	No	Yes	No	¼ in (0.64 cm)
Cemented gravel	Yes	No	No	Yes	Yes	No	Yes	No	No
Boulders	Yes	Yes, if less than well diameter	No	Yes, when in firm bedding	(Difficult)	No	No	No	No
Sandstone	Yes, if soft and/or fractured	Yes, if soft and/or fractured.	Thin layers only	Yes	Yes	Yes	Yes	Yes	No
Limestone	No	No	No	Yes					
Dense igneous rock	No	No	No	Yes	Yes	Yes	Yes	No	No

*The ranges of values in this table are based upon general condition. They may be exceeded for specific areas or conditions.

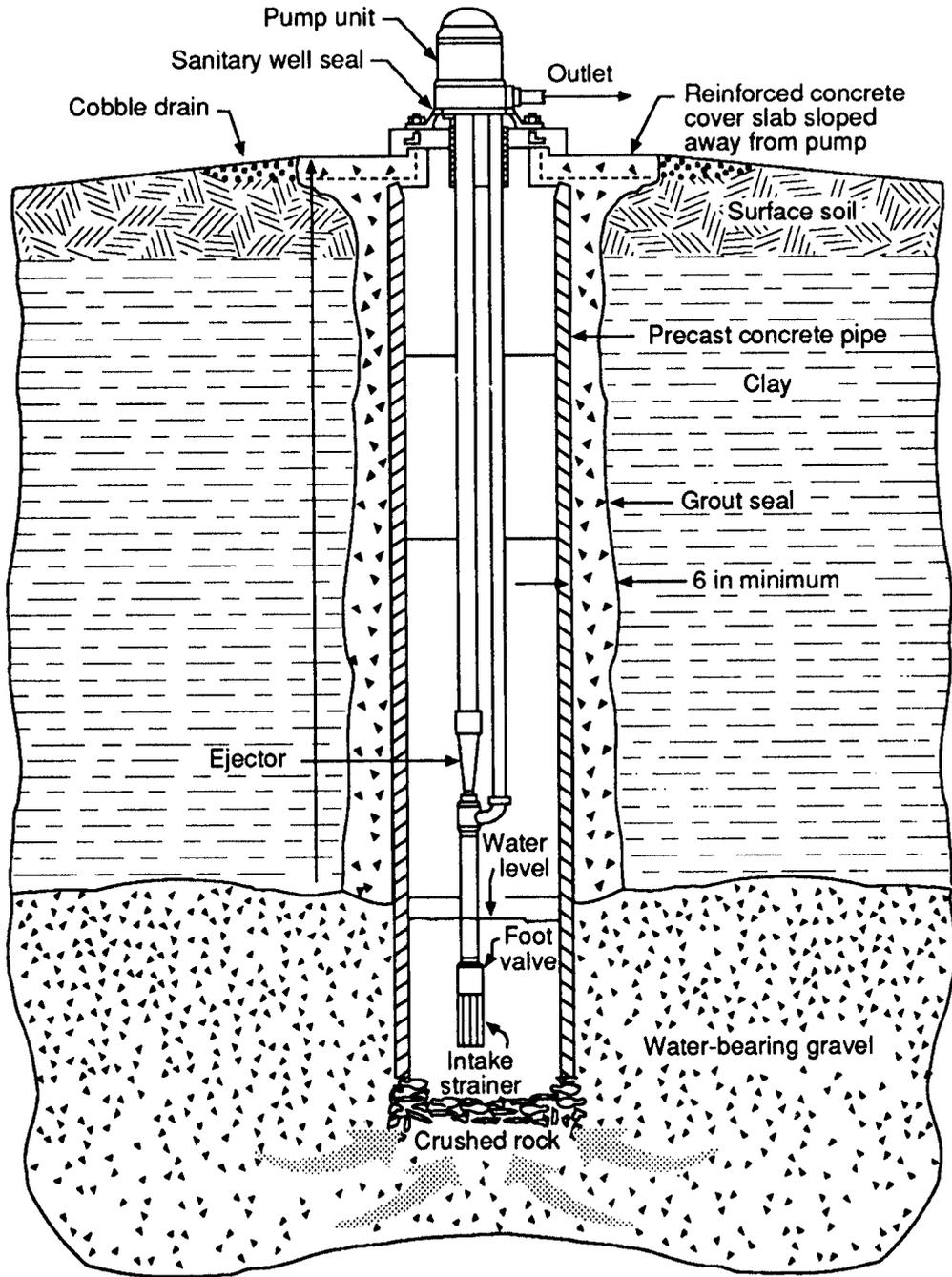


FIGURE 5.9 Dug well with two-pipe jet pump installation. *Note:* Pump screen to be placed below point of maximum draw-down (2).

either by hand or by power equipment. Such wells are usually regarded as practical at depths of less than about 100 ft (30 m) when the water requirement is low and the material overlying the water-bearing formation has noncaving properties and contains few large boulders. In suitable material, holes from 2 to 30 in (5 to 75 cm) in diameter can be bored to about 100 ft (30 m) without caving.

Bored wells may be cased with vitrified tile, concrete pipe, standard wrought iron, steel casing, or other suitable material capable of sustaining imposed loads. The well may be completed by installing well screens or perforated casing in the water-bearing sand and gravel. Proper protection from surface drainage should be provided by sealing the casing with cement grout to the depth necessary to protect the well from contamination.

Driven Wells. The simplest and least expensive of all well types is the driven well. It is constructed by driving into the ground a drive-well point which is fitted to the end of a series of pipe sections (Figures 5.10 and 5.11.) The drive point is of forged or cast steel. Drive points are usually 1.25 or 2 in (3 or 5 cm) in diameter. The well is driven with the aid of a maul or a special drive weight (see Figure 5.11.) For deeper wells, the well points are sometimes driven into water-bearing strata from the bottom of a bored or dug well (Figure 5.12).

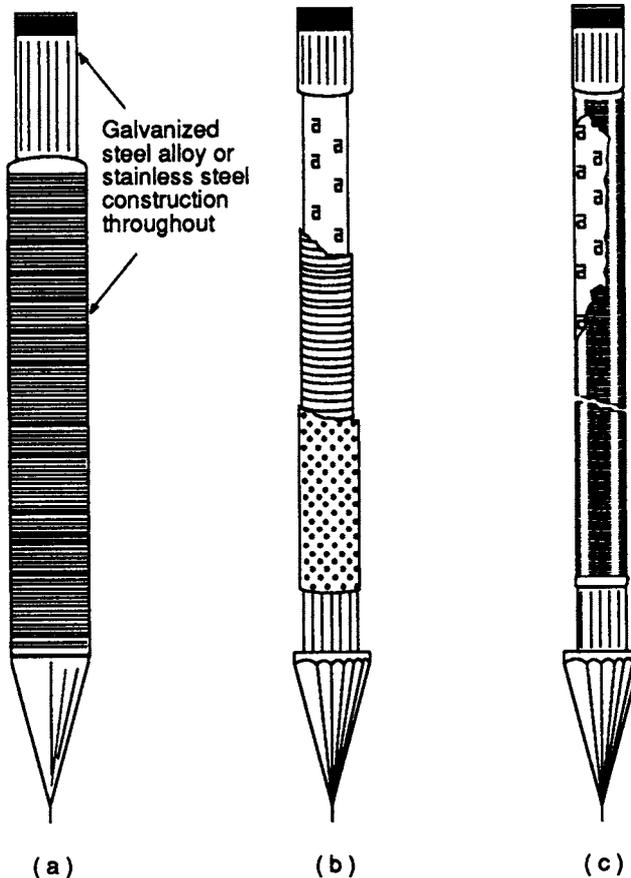


FIGURE 5.10 Different kinds of drive-well points, (a) continuous-slot type; (b) brass jacket type; (c) brass tube type (2).

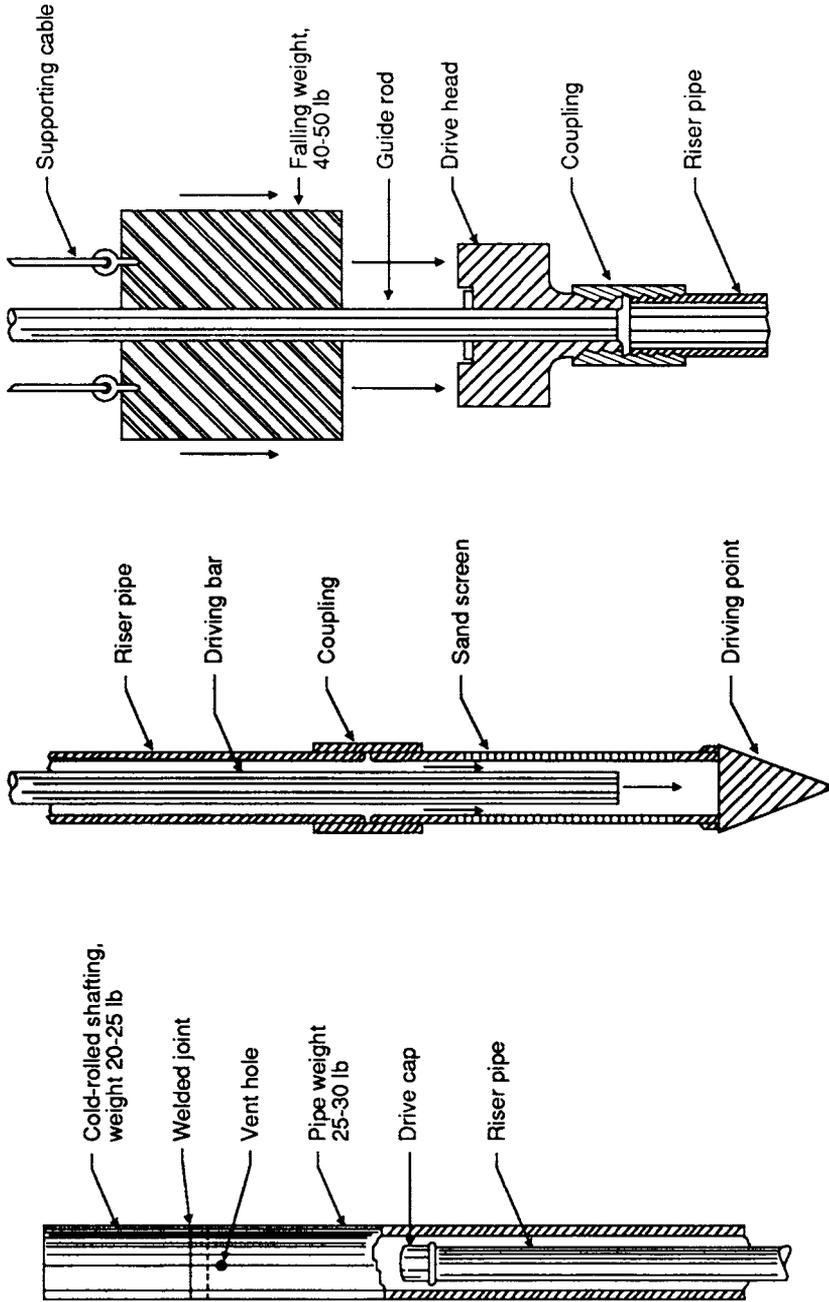


FIGURE 5.11 Well point driving methods (2)

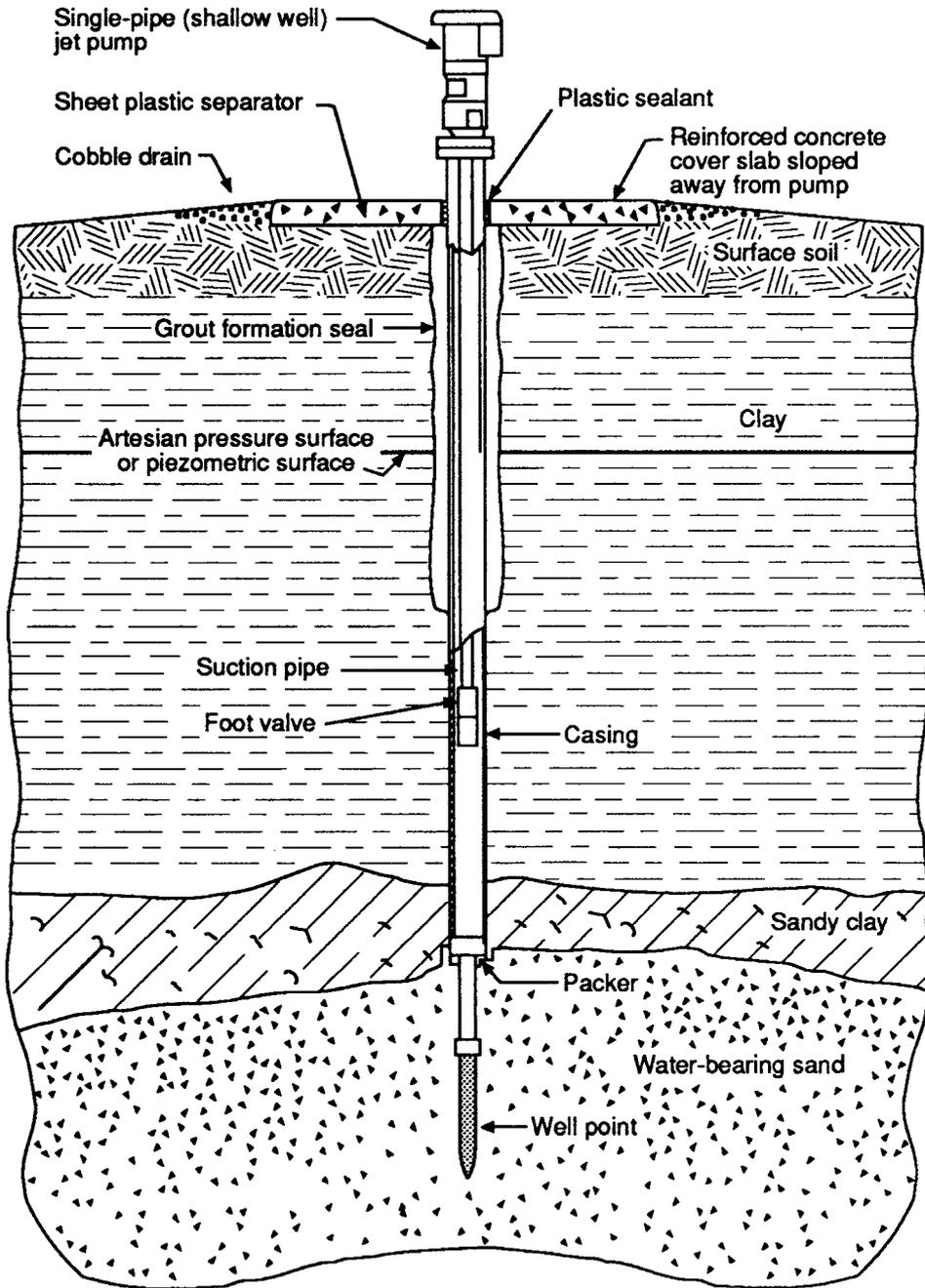


FIGURE 5.12 Hand-bored well with driven well point and “shallow well’ jet pump (2).

The yield of driven wells is generally small to moderate. Where they can be driven an appreciable depth below the water table, they are no more likely than bored wells to be seriously affected by water-table fluctuations. The most suitable locations for driven wells are areas containing alluvial deposits of high permeability. The presence of coarse gravels, cobbles, or boulders interferes with sinking the well point and may damage the wire mesh jacket.

Jetted Wells. A rapid and efficient method of sinking well points is that of jetting or washing-in. This method requires a source of water and a pressure pump. Water forced under pressure down the riser pipe issues from a special washing point. The well point and pipe are then lowered as material is loosened by the jetting.

The riser pipe of a jetted well is often used as the suction pipe for the pump. In such instances, surface water may be drawn into the well if the pipe develops holes by corrosion. An outside protective casing may be installed to the depth necessary to provide protection against the possible entry of contaminated surface water. The annular space between the casings should then be filled with cement grout. The protective casing is best installed in an auger hole and the drive point then driven inside it.

Drilled Wells. Construction of a drilled well (see Figure 5.13) is ordinarily accomplished by percussion or rotary hydraulic drilling. The selection of the method depends primarily on the geology of the site and the availability of equipment.

1. *Percussion (cable-tool) method:* Drilling by the cable-tool or percussion method is accomplished by raising and dropping a heavy drill bit and stem. The impact of the bit crushes and dislodges pieces of the formation. The reciprocating motion of the drill tools mixes the drill cuttings with water into a slurry at the bottom of the hole. This is periodically brought to the surface with a bailer, a 10- to 20-ft (3- to 6-m) long pipe with a valve at the lower end.
2. *Hydraulic rotary drilling method:* The hydraulic rotary drilling method may be used in most formations. The essential parts of the drilling assembly include a derrick and hoist, a revolving table through which the drill pipe passes, a series of drill-pipe sections, a cutting bit at the lower end of the drill pipe, a pump for circulation of drilling fluid, and a power source to drive the drill.
3. *Air rotary drilling method:* The air rotary method is similar to the rotary hydraulic method in that the same type of drilling machine and tools may be used. The principal difference is that air is the fluid used rather than mud or water. In place of the conventional mud pump to circulate the fluids, air compressors are used. Many drillers equip the rig with a mud pump to increase the versatility of the equipment.
4. *Down-hole air hammer:* The down-hole pneumatic hammer combines the percussion effect of cable-tool drilling and the rotary movement of rotary drilling. The tool bit is equipped with tungsten carbide inserts at the cutting surfaces. Tungsten carbide is very resistant to abrasion.

Well Pumps. Three types of pumps are commonly used in individual water distribution systems (2). They are the positive displacement, the centrifugal, and the jet. These pumps can be used in a water system utilizing either a ground or surface source. They are desirable in areas where electricity or other power (gasoline, diesel oil, or windmill) make use of a power-operated pump possible. When a power supply is not available, a hand pump or some other manual method of supplying water must be used. Special types of pumps with limited application for individual water-supply systems include air-lift pumps and hydraulic rams.

Positive-Displacement Pumps. The positive-displacement pump forces or displaces the water through a pumping mechanism. These pumps are of several types.

One type of positive-displacement pump is the reciprocating pump. This pump consists of a mechanical device that moves a plunger back and forth in a closely fitted cylinder. The plunger is driven by the power source, and the power motion is converted from a rotating action to a reciprocating motion by the combined work of a speed reducer, crank, and a connecting rod. The cylinder, composed of a cylinder wall, plunger, and check valve, should be located near or below the static water level to eliminate the need for priming. The pumping action begins when the water enters the cylinder through a check valve. When the piston moves,

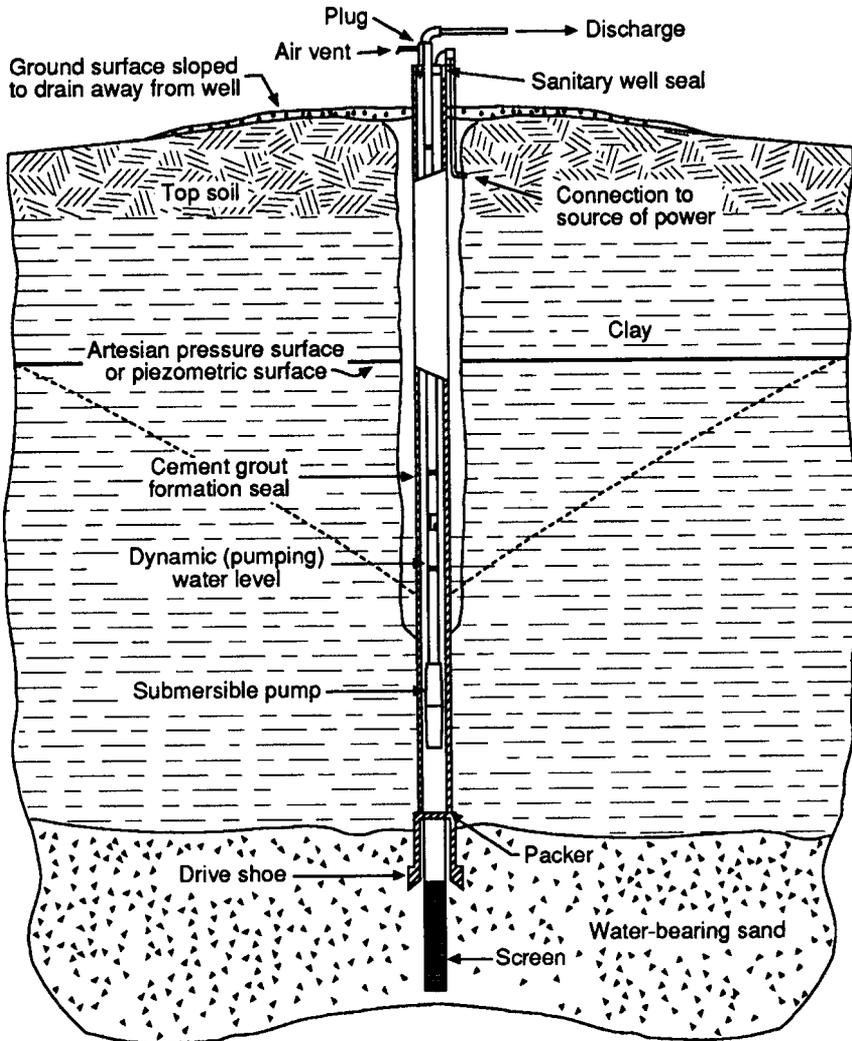


FIGURE 5.13 Drilled well with submersible pump (2).

the check valve closes, and in so doing forces the water through a check valve in the plunger. With each subsequent stroke, the water is forced toward the surface through the discharge pipe.

Another type of positive-displacement pump is the helical or spiral rotor. The helical rotor consists of a shaft with a helical (spiral) surface which rotates in a rubber sleeve. As the shaft turns, it pockets or traps the water between the shaft and the sleeve and forces it to the upper end of the sleeve.

Other types of positive-displacement pumps include the regenerative turbine type. It incorporates a rotating wheel or impeller which has a series of blades or fines (sometimes called buckets) on its outer edge and a stationary enclosure called a raceway or casting. Pressures several times that of pumps relying solely on centrifugal force can be developed.

Centrifugal Pumps. Centrifugal pumps are pumps containing a rotating impeller mounted on a shaft

turned by the power source. The rotating impeller increases the velocity of the water and discharges it into a surrounding casing shaped to slow down the flow of water and convert the velocity to pressure. This decrease of the flow further increases the pressure.

Each impeller and matching casing is called a stage. The number of stages necessary for a particular installation will be determined by the pressure needed for the operation of the water system, and the height the water must be raised from the surface of the water source.

When the pressure is more than can be practicably or economically furnished by a single stage, additional stages are used. A pump with more than one stage is called a multistage pump. In a multistage pump, water passes through each stage in succession, with an increase in pressure at each stage. Multistage pumps commonly used in individual water systems are of the turbine and submersible types.

The vertical-drive turbine pump consists of one or more stages with the pumping unit located below the drawdown level of the water source. A vertical shaft connects the pumping assembly to a drive mechanism located above the pumping assembly. The discharge casing, pump housing, and inlet screen are suspended from the pump base at the ground surface. The weight of the rotating portion of the pump is usually suspended by a thrust bearing located in the pump head. The intermediate pump bearings may be lubricated by either oil or water. From a sanitary point of view, lubrication of pump bearings by water is always preferable, since lubricating oil may leak and contaminate the water.

When a centrifugal pump is driven by a closely coupled electric motor constructed for submerged operation as a single unit, it is called a submersible pump (Figure 5.14). The electrical wiring to the submersible motor must be waterproof. The electrical control should be properly grounded to minimize the possibility of shorting and thus damaging the entire unit. The pump and motor assembly are supported by the discharge pipe; therefore, the pipe should be of such size that there is no possibility of breakage.

The turbine or submersible pump forces water directly into the distribution system; therefore, the pump assembly must be located below the maximum drawdown level. This type of pump can deliver water across a wide range of pressures with the only limiting factor being the size of the unit and the horsepower applied. When sand is present or anticipated in the water source, special precautions should be taken before this type of pump is used since the abrasion action of the sand during pumping will shorten the life of the pump.

Jet (Ejector) Pumps. Jet pumps are actually combined centrifugal and ejector pumps. A portion of the discharged water from the centrifugal pump is diverted through a nozzle and venturi tube. A pressure zone lower than that of the surrounding area exists in the venturi tube; therefore, water from the source (well) flows into this area of reduced pressure. The velocity of the water from the nozzle pushes it through the pipe toward the surface where the centrifugal pump can lift it by suction. The centrifugal pump then forces it into the distribution system (see Figure 5.15).

Pump Selection. The type of pump selected for a particular installation should be determined on the basis of the following fundamental considerations.

1. Yield of the well or water source
2. Daily needs and instantaneous demand of the users
3. The "usable water" in the pressure or storage tank
4. Size and alignment of the well casing
5. Total operating head pressure of the pump at normal delivery rates, including lift and all friction losses
6. Difference in elevation between ground level and water level in the well during pumping
7. Availability of power
8. Ease of maintenance and availability of replacement parts
9. First cost and economy of operation
10. Reliability of pumping equipment

Table 5.14 provides useful information on pump selection.

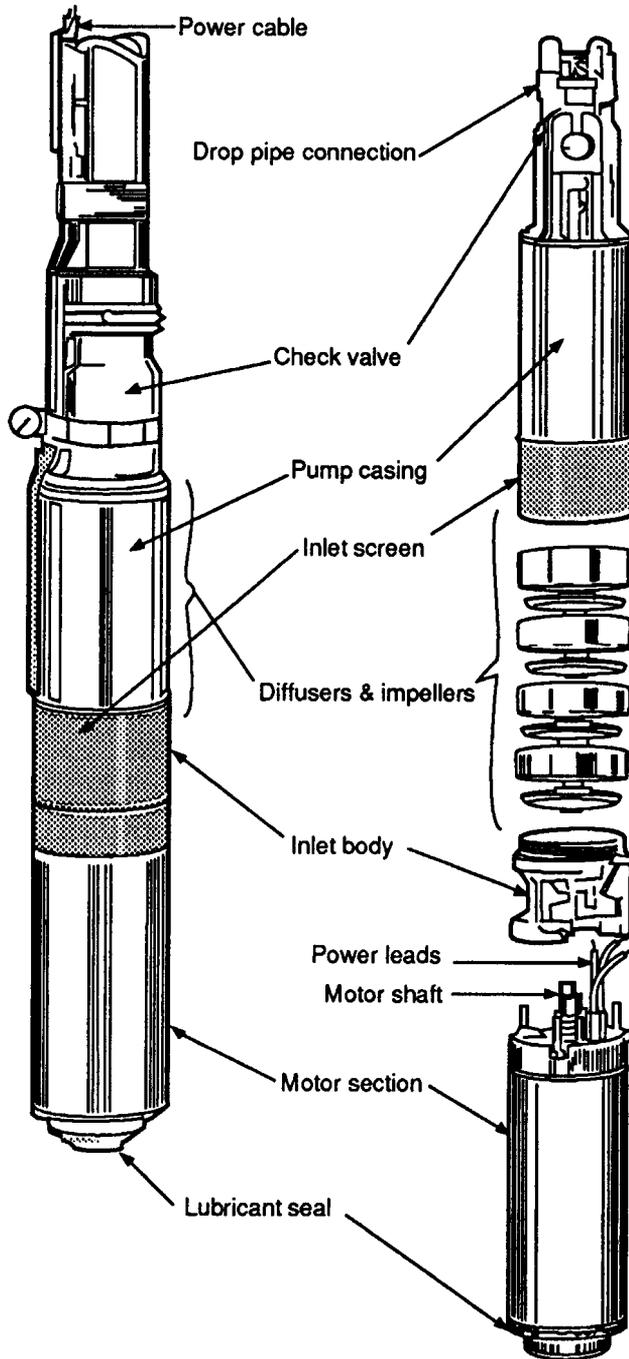


FIGURE 5.14 Exploded view of submersible pump (2).

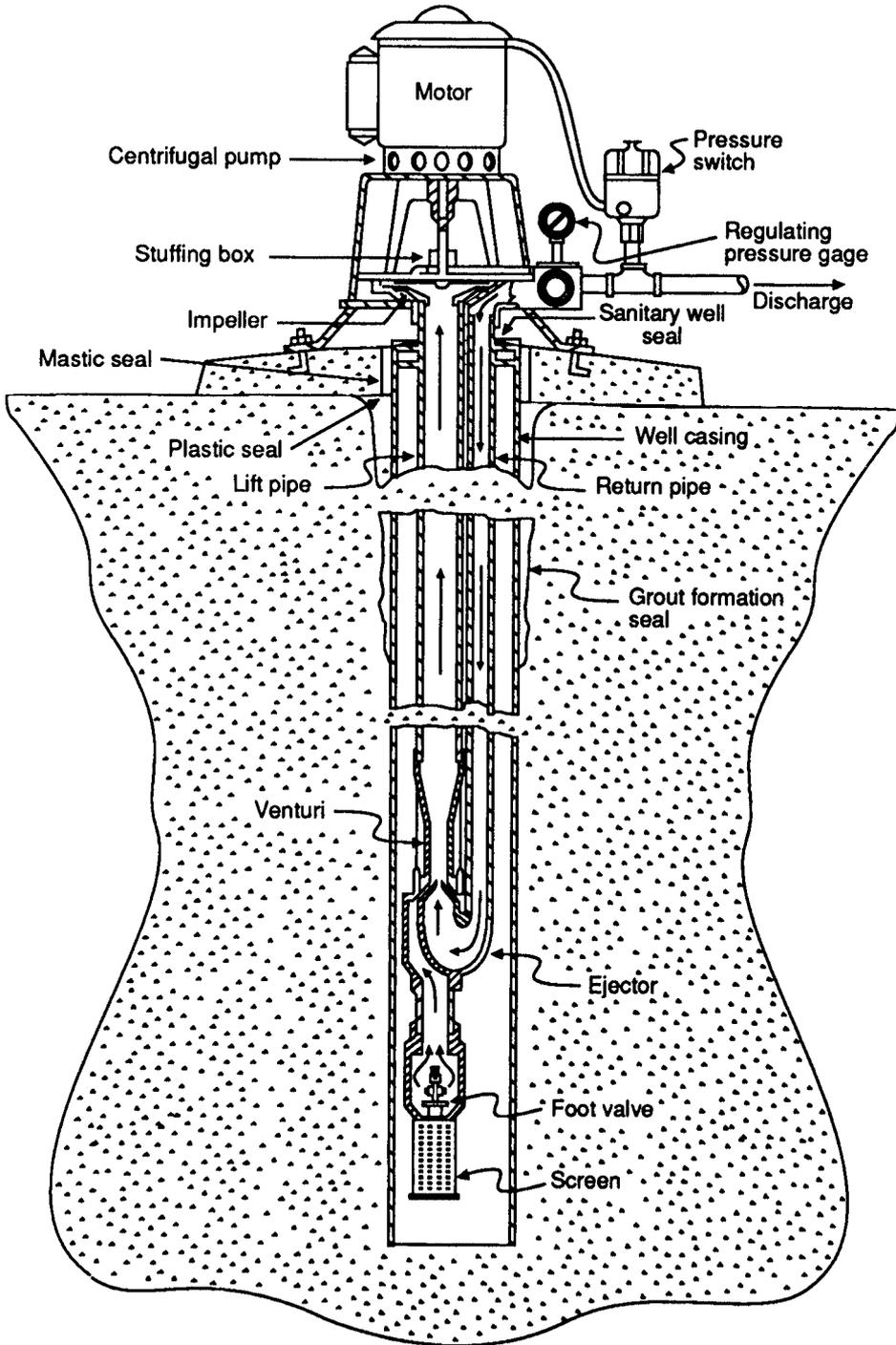


FIGURE 5.15 “Over-the-well” jet pump installment (2).

TABLE 5.14 Information on Pump Selection (2)

Type of pump	Practical suction lift,* ft (m)	Usual well-pumping depth, ft (m)	Usual pressure heads, ft (m)	Advantages	Disadvantages	Remarks
Positive displacement:						
1. Shallow well (reciprocating)				1. Positive action 2. Discharge against variable heads	1. Pulsating discharge 2. Subject to vibration and noise	1. Best suited for capacities of 5–25 gpm
2. Deep well (reciprocating)	22–25 (6.71–7.62)	22–25 (6.71–7.62)	100–2200 (30.48–670.56)	3. Pumps water containing sand and silt 4. Especially adapted to low capacity and high lifts	3. Maintenance cost may be high 4. May cause destructive pressure if operated against closed valve	2. Against moderate to high heads 3. Adaptable to hand operation 4. Can be installed in very small diameter wells (2-in casing) 5. Pump must be set directly over well (deep well only)
Centrifugal:						
1. Shallow well				1. Smooth, even flow 2. Pumps water containing sand and silt 3. Pressure on system is even and free from shock 4. Low starting torque 5. Usually reliable and good service life	1. Loses prime easily 2. Efficiency depends on operating under design heads and speed	
a. Straight centrifugal (single stage)	20 ft. (6.10) max.	10–20 (3.05–6.10)	100–150 (30.48–45.72)			
b. Regenerative vane turbine type (single impeller)	28 (8.53) max.	28(8.53)	100–200 (30.48–45.72)	1. Same as straight centrifugal except not suitable for pumping water containing sand or silt 2. Self-priming	1. Same as straight centrifugal except maintains priming easily	

<p>2. Deep well</p> <p>a. Vertical line shaft turbine (multi-stage)</p>	<p>Impellers submerged</p> <p>50–300 (15.24–91.44)</p> <p>100–800 (30.48–243.84)</p>	<p>1. Same as shallow well turbine</p> <p>2. All electrical components are accessible, above ground</p>	<p>1. Efficiency depends on operating under design head and speed</p> <p>2. Requires straight well large enough for turbine bowls and housing</p> <p>3. Lubrication and alignment of shaft critical.</p> <p>4. Abrasion from sand</p>
<p>b. Submersible turbine (multistage)</p>	<p>Pump and motor submerged</p> <p>50–400 (15.24–121.92)</p> <p>50–400 (15.24–121.92)</p>	<p>1. Same as shallow well turbine</p> <p>2. Easy to frostproof installation</p> <p>3. Short pump shaft to motor</p> <p>4. Quiet operation</p> <p>5. Well straightness not critical</p>	<p>1. Repair to motor or pump requires pulling from well</p> <p>2. Sealing of electrical equipment from water vapor critical</p> <p>3. Abrasion from sand</p> <p>1. 3500-rpm models, while popular because of smaller diameters or greater capacities, are more vulnerable to wear and failure from sand and other causes</p>
<p>Jet (ejector):</p> <p>1. Shallow well (4.57–6.10)</p>	<p>15–20 (4.57–6.10)</p> <p>below ejector</p> <p>15–20 (24.38–45.72)</p> <p>below ejector</p>	<p>1. High capacity at low heads</p> <p>2. Simple in operation</p> <p>3. Does not have to be installed over the well</p> <p>4. No moving parts in the well</p>	<p>1. Capacity reduces as lift increases</p> <p>2. Air in suction or return line will stop pumping</p>
<p>2. Deep well</p>	<p>15–20 (4.57–6.10)</p> <p>below ejector</p> <p>25–120 (7.62–36.58), 200 (60.26)</p> <p>below ejector</p> <p>80–150 (24.38–45.72)</p>	<p>1. Same as shallow well jet</p> <p>2. Well straightness not critical</p>	<p>1. The amount of water returned to ejector increases with increased lift—50% of total water pumped at 50 ft (15.24 m) lift and 75% lift at 100 ft (30.48 m) lift</p>

*Practical suction lift as sea level. Reduce lift 1 ft above sea level.

The rate of water delivery required depends on the time of effective pump operation as related to the total water consumption between periods of pumping.

When the well yield is low in comparison to peak demand requirements, an appropriate increase in the storage capacity is required. The life of an electric drive motor will be reduced when there is excessive starting and stopping. The water system, therefore, should be designed so that the interval between starting and stopping is as long as is practicable but not less than 1 min.

The total operating head of a pump consists of the lift (vertical distance from pumping level of the water source to the pump), the friction losses in the pipe and fittings from water source to pump, and the discharge pressure at the pump (see Figure 5.16).

Individual and Small Systems

As a result of the natural purifying capacity and protection of the soil, individual and small water supplies generally first try to develop springs 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 used without purifying the waters drawn.

Pump Capacity. Counting the number of fixtures in the home permits a ready determination of required pump capacity using Figure 5.17. For example, a home with kitchen sink, water closet, bathtub, wash basin, automatic clothes washer, laundry tub, and two outside hose bibs has a total of eight fixtures. Referring to the figure, it is seen that eight fixtures correspond to a recommended pump capacity between 9 and 11 gpm (0.03 and 0.04 m³/min). The lower value should be the minimum. The higher value might be preferred if additional fire protection is desired, or if garden irrigation is contemplated.

Pipe and Fittings. For reasons of economy and ease of construction, distribution lines for small water systems are ordinarily made up with standard threaded galvanized iron or steel pipe and fittings. Other types of pipes used are cast iron, asbestos-cement, concrete, plastic, and copper. Under certain conditions and in certain areas, it may be necessary to use protective coatings, galvanizing, or have the pipes dipped or wrapped. When corrosive water or soil is encountered, copper, brass, wrought iron, plastic, or cast iron pipe, although usually more expensive initially, will have a longer and more useful life. Cast iron is not usually available in sizes below 2 in (1 cm) in diameter; hence, its use is restricted to the larger transmission lines.

Plastic pipe for cold-water piping is usually simple to install, has a low initial cost, and has good hydraulic properties. When used in a domestic water system, plastic pipe should be certified by an acceptable testing laboratory (such as the National Sanitation Foundation) as being nontoxic and non-taste-producing. It should be protected against crushing and from attack by rodents.

Fittings are usually available in the same sizes and materials as piping, but valves are generally cast in bronze or other alloys. In certain soil, the use of dissimilar metals in fittings and pipe may create electrolytic corrosion problems. The use of nonconductive plastic inserts between pipe and fittings or the installation of sacrificial anodes is helpful in minimizing such corrosion.

Pipes should be laid as straight as possible in trenches, with air-relief valves or hydrants located at the high points on the line. Failure to provide for the release of accumulated air in a pipeline on hilly ground may greatly reduce the capacity of the line. It is necessary that pipeline trenches be excavated deep enough to prevent freezing in the winter. Pipes placed in trenches at a depth of more than 3 ft (1 m) will help to keep the water in the pipeline cool during the summer months.

Pipe Flow. The pipeline selected should be adequate to deliver the required peak flow of water without excessive loss of head, i.e., without decreasing the discharge pressure below a desirable minimum. The

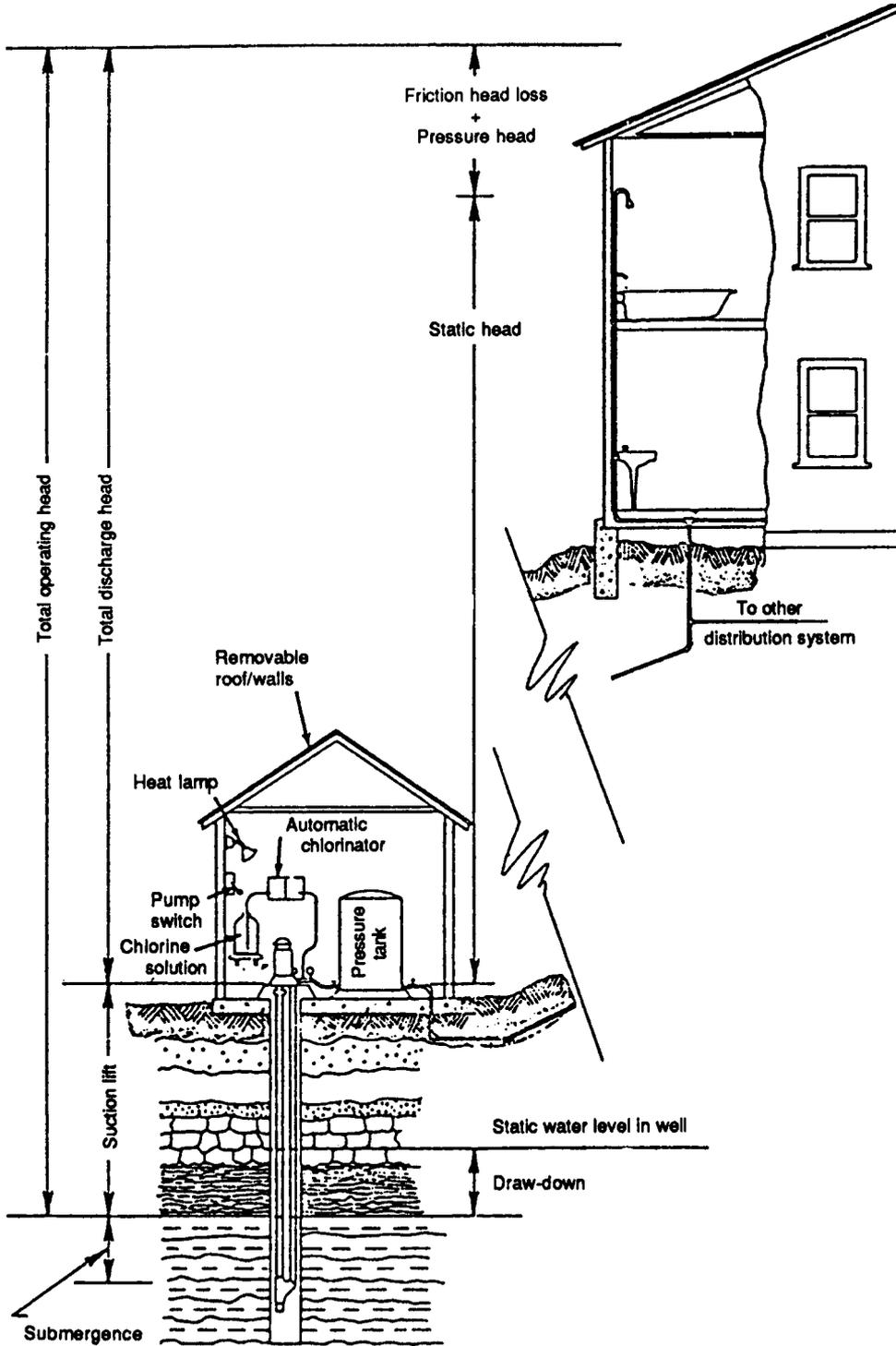


FIGURE 5.16 Components of total operating head in well pump installations (2).

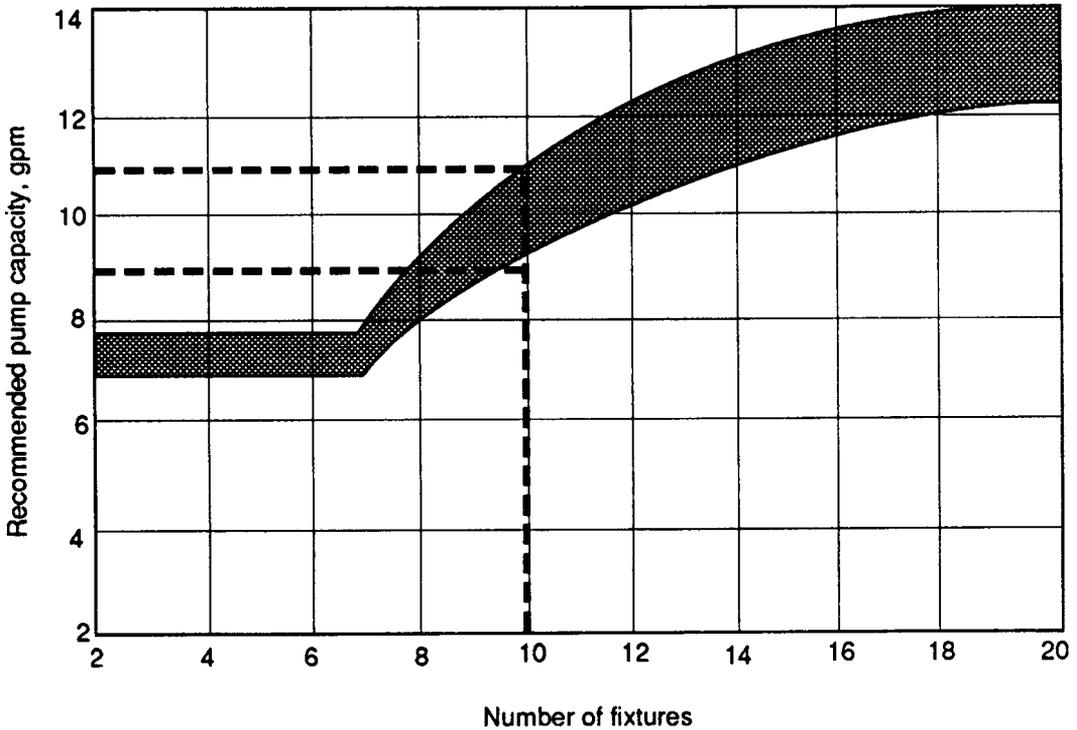


FIGURE 5.17 Determining recommended pump capacity (2).

normal operating water pressure for household or domestic use ranges from 20 to 60 lb/m² (1.41 to 4.22 kgf/cm²), or about 45 to 140 ft (13.72 to 42.67 m) of head at the fixture.

The capacity of a pipeline is determined by its size, length, and interior surface condition. Assuming that the length of the pipe is fixed and its interior condition established by the type of material, the usual problem in design of a pipeline is that of determining the required diameter.

The correct pipe size can be selected with the aid of Figure 5.18, which gives size as a function of head loss H , length of pipeline L , and peak discharge Q . As an example of the use of Figure 5.18, suppose that a home and farm installation is served by a reservoir a minimum distance of about 500 ft (150 m) from the point of use, one whose surface elevation is at least 150 ft (46 m) above the level of domestic service, and in which a minimum service pressure of 30 lb/in² (2.11 kgf/cm²) is required. It will be necessary first to determine the maximum operating head loss, i.e., the difference in total head and the required pressure head at the service.

$$H = 150 - 2.3 \times 30 = 150 - 69 = 81 \text{ ft (24.69 m)} \tag{5.12}$$

The maximum peak demand which must be delivered by the pipeline is determined to be 30 gpm (0.11 m³/min).

$$Q = 30 \text{ gpm}$$

The hydraulic gradient is 0.162 ft/ft.

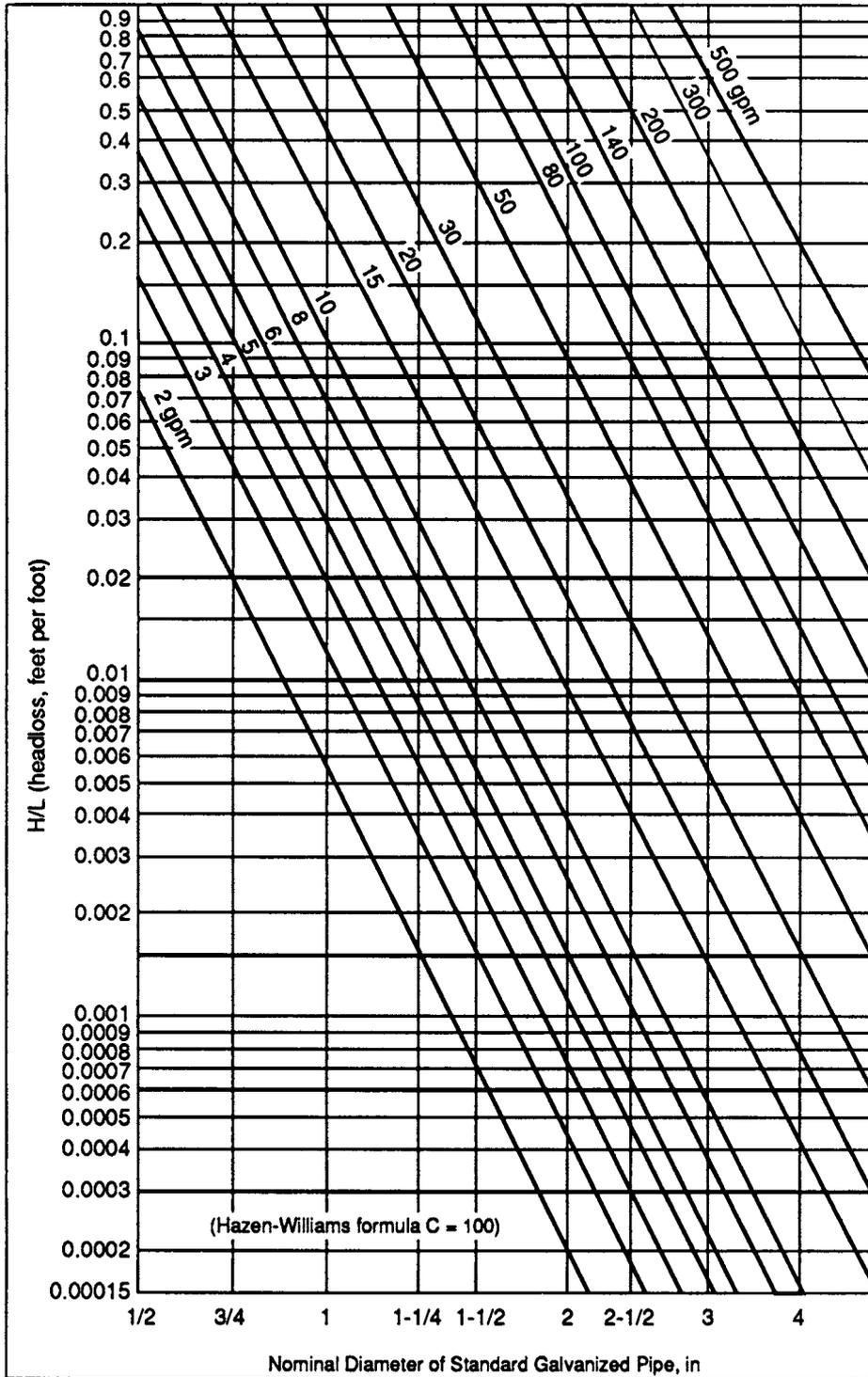


FIGURE 5.18 Head loss versus pipe size (2).

TABLE 5.15 Allowance in Equivalent Length of Pipe for Friction Loss in Valves and Threaded Fittings (2)

Diameter of fitting, in	90° std. ell, ft	45° std. ell, ft	90° side tee, ft	Coupling or straight run, ft	Gate valve, ft	Globe valve, ft	Angle valve, ft
3/8	1	0.6	1.5	0.3	0.2	8	4
1/2	2	1.2	3	0.6	0.4	15	8
3/4	2.5	1.5	4	0.8	0.5	20	12
	3	1.8	5	0.9	0.6	25	15
1 1/4	4	2.4	6	1.2	0.8	35	18
1 1/2	5	3	7	1.5	1.0	45	22
2	7	4	10	2	1.3	55	28
2 1/2	8	5	12	2.5	1.6	65	34
3	10	6	15	3	2	80	40
3 1/2	12	7	18	3.6	2.4	100	50
4	14	8	21	4	2.7	125	55
5	17	10	25	5	3.3	140	70
6	20	12	30	6	4	165	80

Note: 2.54 cm = 1 in; 0.3 m = 1 ft.

$$\frac{H}{L} = \frac{81}{500} = 0.162 \text{ ft/ft} \quad (5.13)$$

Entering Figure 5.18, with the computed values of H/L and Q , one finds that the required standard galvanized pipe size is approximately 1 ³/₈ in (3.5 cm). Since pipes are available only in standard dimensions, standard pipe of 1 1/2 in (3.8 cm) in diameter (the next size) should be used.

Additional head losses may be expected from the inclusion of fittings in the pipeline. These losses may be expressed in terms of the equivalent to the length and size of pipe which would produce an equivalent loss if, instead of adding fittings, we added additional pipe. Table 5.15 lists some common fitting losses in terms of an equivalent pipe length.

In the example given in Table 5.15, the inclusion of two gate valves (open), two standard elbows, and two standard tees (through) would produce a head loss equivalent to 15 ft (4.57 m) of 1 1/2-in (3.8-cm) pipe. From Figure 5.18, one finds that by using 515 ft (157 m) of 1 1/2-in pipe instead of the actual length of 500 ft (152 m) ($H/L = 0.157$), the capacity of the system for the same total head loss is about 38 gpm (0.14 m³/min), a satisfactory discharge.

It can be seen from this example that fitting losses are not particularly important for fairly long pipelines, say greater than about 300 ft (90 m). For pipelines less than 300 ft, fitting losses are very important and have a direct bearing on pipe selected; therefore, they should be calculated carefully.

Globe valves which do produce large head losses should be avoided in main transmission lines for small water systems.

Interior pipings, fittings, and accessories should conform to the minimum requirements for plumbing of the National Plumbing Code or equivalent applicable plumbing code of the locality.

SURFACE WATER COLLECTION

Various types of intake configurations, transmission systems, and storage facilities are described in the following sections.

Intakes

Water is generally drawn from lakes, rivers, and reservoirs through relatively simple submerged intakes or through structures that rise above the water (25). These more elaborate structures may include, in addition to gates, mechanical screens and chlorinators. Intakes should be designed and placed to draw as clean and palatable water as the source of supply can provide.

Intake Locations. In the following sections, various locations of intakes are described.

River Intakes. River intakes should be constructed upstream from points of sewage and industrial waste discharge if possible. Often, towers are used. They should be placed to advantage of deep water, a stable bottom, and the best available water quality, and should be protected against floods, debris, and river traffic.

Lake and Reservoir Intakes. These intakes should be placed to avoid shore waters where possible, and are usually towers or pipes. Placement should take into consideration sources of pollution, prevailing winds, surface and subsurface currents, and shipping lanes. Intakes can be designed with top and bottom ports. Top ports can be opened when winds drive clean surface water to the intake structure and the bottom ports opened when the wind is offshore and clean bottom water can be drawn in.

Intake Types. In the following sections, the principal types of intakes are described.

Exposed Intake Towers. Exposed intakes may be towerlike structures located in dams, on streams and lake banks, near the shore, and connected by a bridge or causeway, or located at such distances that they can only be reached by boat.

Submerged Intakes. Submerged intakes can take the form of a crib or screened bellmouth. The crib, constructed of heavy timbers and weighted with rocks, protects the conduit against damage. An intake pipe within the crib draws water through a grating.

Intake Design. The following factors should be considered in intake design.

Velocity. Ice troubles are reduced if entrance velocities are kept between 3 and 4 in/s (8 and 10 cm/s). Low velocities will not transport ice and will keep the entraining of leaves and debris to a minimum. At low velocities, too, fish can escape from the intake current (26).

Depth. Ice troubles are reduced in frequency if intake ports are placed as deep as 25 ft (8 m) below the water surface. Bottom sediments are kept out of the intake if entrance ports are raised 4 to 6 ft (1 to 2 m) above the lake or reservoir floor. Ports should be provided at numerous other levels as well, in order to permit shifting the draft to optimum water quality. A vertical interval of 15 ft (4.6 m) is often chosen.

Grating Screens. Submerged gratings are given openings of 2 to 3 in (5 to 7.5 cm). Screens are commonly 6 to 8 meshes to the inch (two to three meshes to the centimeter) and have face velocities of 3 to 4 in/s (7.6 to 10.2 cm/s).

Ice Protection. Three types of ice are encountered in cold climates: sheet, frazil, and anchor ice. Frazil ice, depending upon the conditions of its formation, may take the shape of needles, flakes, or formless slush. Carried to intakes or produced in them from supercooled water, frazil ice attaches itself to metallic racks, screens, conduits, and pumps. Anchor ice behaves like frazil ice, but is derived from ice crystals that have developed on the bottom and on submerged objects in much the same way that frost forms on vegetation during a clear night. Neither frazil nor anchor ice is normally encountered below sheet ice, which thus offers some protection against their occurrence. Difficulties can also be prevented by heating metallic surfaces or raising the temperature of supercooled water by about -17.7°C (0.1°F), the common order of magnitude of maximum supercooling. Compressed air, backflushing, and light explosives 0.25 lb (0.11 kg) of 60% dynamite have also been used successfully to free ice-clogged intakes.

Conduits. Constructed as pipes (often with flexible joints) or as tunnels, intake conduits are designed to carry water to the shore at self-cleansing velocities of 3 to 4 ft/s (1 to 1.2 m/s). Pipelines are generally laid in trenches that are dredged and backfilled. Where they reach the shoreline, the pipes must be well protected

against disturbance by wave and ice. Conduits on earth foundations beneath dams are subjected to high loads and to the stresses incident to the consolidation of the foundation.

Pump Stations. Pumps that draw water through intake conduits are generally placed in a well on shore. Since the suction lift, including friction, should not exceed 15 to 20 ft (4.5 to 6 in), pump wells are often quite deep in order to be of service when river, lake, or reservoir levels are lowered appreciably in times of drought. This creates problems of hydrostatic uplift and seepage in times of flood. Large pumping units are generally placed in dry wells.

Transmission Systems

Transmission systems are composed of supply conduits and pumping facilities.

Supply Conduits. Supply conduits, or aqueducts, transport water from the source of supply to the community. They form the connecting link between the raw-water collection works and the finished-water distribution system. Depending upon topography and available materials, conduits are designed to carry the water in open-channel flow or under pressure.

Hydraulic Design. The hydraulic design of supply conduits is concerned chiefly with the estimation of frictional resistances to flow and with the pressures that are maintained or created in the conduit.

The most nearly rational relationship between the velocity of flow and the head loss in a conduit, also one of the earliest, is that proposed by Darcy and later modified by Weisbach and others (25). As now used for calculating the head loss in round pipes flowing full, it has the form:

$$h_f = f \frac{l}{d} \frac{v^2}{2g} \quad (5.14)$$

where h_f is the head loss, l is the length of pipe, d is the diameter of the pipe, v is the mean velocity of flow, g is the gravity constant, and f a dimensionless friction factor.

Because of difficulties or inconveniences inherent in the use of this formula, engineers more commonly make use of so-called exponential equations, which relate loss of head to flow. Among them, the Hazen-Williams formula is most widely used to express flow relations in pressure conduits, whereas the Manning formula is favored to express flow relations in free-flow conduits.

The Hazen-Williams formula is

$$v = Cr^{0.63}s^{0.54} \times 0.001^{-0.04} \quad (5.15)$$

where v is the mean velocity of flow (ft/s), r is the hydraulic radius (ft), s is the hydraulic gradient, or loss of head h_f (ft) in a conduit of length l (ft), 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 Chezy formula.

For circular conduits

$$v = 0.550CD^{0.63}s^{0.54} = 0.115Cd^{0.63}s^{0.54} \quad (5.16)$$

and

$$Q \text{ (mgd)} = 0.279CD^{2.63}s^{0.54} \quad (5.17)$$

or

$$Q \text{ (gal/day)} = 405Cd^{2.63}s^{0.54} \quad (5.18)$$

Here D is the diameter of large pipes (ft), d is that of small pipes (in) and Q is the rate of discharge as indicated. Solution of Eqs. (5.14) to (5.18) for Q , v , r , D , d , s , or C requires the use of logarithms, a log-log rule, and tables. Figure 5.19 is a Hazen-Williams nomogram (27).

The weakest element in the use of the Hazen-Williams formula is the estimate of the value of C when its magnitude has not been determined by actual measurements of loss of head and discharge or velocity. Values of C vary for different materials of construction and with the relative deterioration of these materials by length of service. They vary also somewhat with size and shape. The values in Table 5.16 reflect more or less general experience.

When the rate of water consumption and fire demand is known, the capacity of supply conduits is dependent upon their position in the waterworks system and the designer's preferences.

Conduit Selection. Analysis of the cost of construction of masonry aqueducts and tunnels shows ordinarily that it is cheapest to build them to the full projected capacity of the system. Pipelines, however, are sometimes more economical if a first line of limited capacity is built to be followed by a second line that will round out the total requirements at the time the capacity of the first line has been reached. Multiple supply lines may be constructed simultaneously, however, under a number of special conditions, as follows:

1. When the size of a single line would exceed the maximum size of pipe that is, or can be, manufactured satisfactorily. Centrifugal cast iron pipe, for example, is not manufactured in sizes greater than 36 in (91 cm).
2. When the pipe is known to fail in such a way that much damage is done and that repairs cannot be made within a reasonable length of time. Cast iron pipe, for example, has been known to fracture suddenly and to open cracks that release enough water to wash out a number of lengths of pipe before the water can be shut off.
3. When the location of the line presents special hazards. River crossings, for example, are endangered by floods, ice, and ships' anchors; conduits in mining areas by cave-ins of the ground.

The shape of supply conduits is varied to ensure their best hydraulic performance while balancing against important structural needs. The following shapes of open conduits are common:

- For canals in earth—trapezoids approaching half a hexagon as nearly as the maintainable side slopes permit
- For canals in rock—rectangles twice as wide as they are deep
- For flumes of masonry or wood—rectangles twice as wide as they are deep
- For flumes of wood staves or steel—semicircles
- For pressure aqueducts, pressure tunnels, and pipelines—circles
- For grade aqueducts and grade tunnels—horseshoe sections

Closed conduits are called upon to resist some of the following forces:

- Water pressure within the conduit equal to the full head of water to which the conduit can be subjected.
- Water hammer or transient internal pressure due to sudden reduction in the velocity of the water, by the rapid closing of a gate, for example. Common allowances for water hammer in cast iron pipes are shown in Table 5.17 (25).
- External loads in the form of backfill and traffic.
- Expansion and contraction with changes in temperature that are normally confined to fluctuations in the temperature of the water carried in the conduit.

Locating Conduits. Procedures for locating a supply conduit are similar to those used in the location of railroads or highways. Grade aqueducts and tunnels are held rigidly to the hydraulic grade lines. In long supply

Design of Distribution Systems

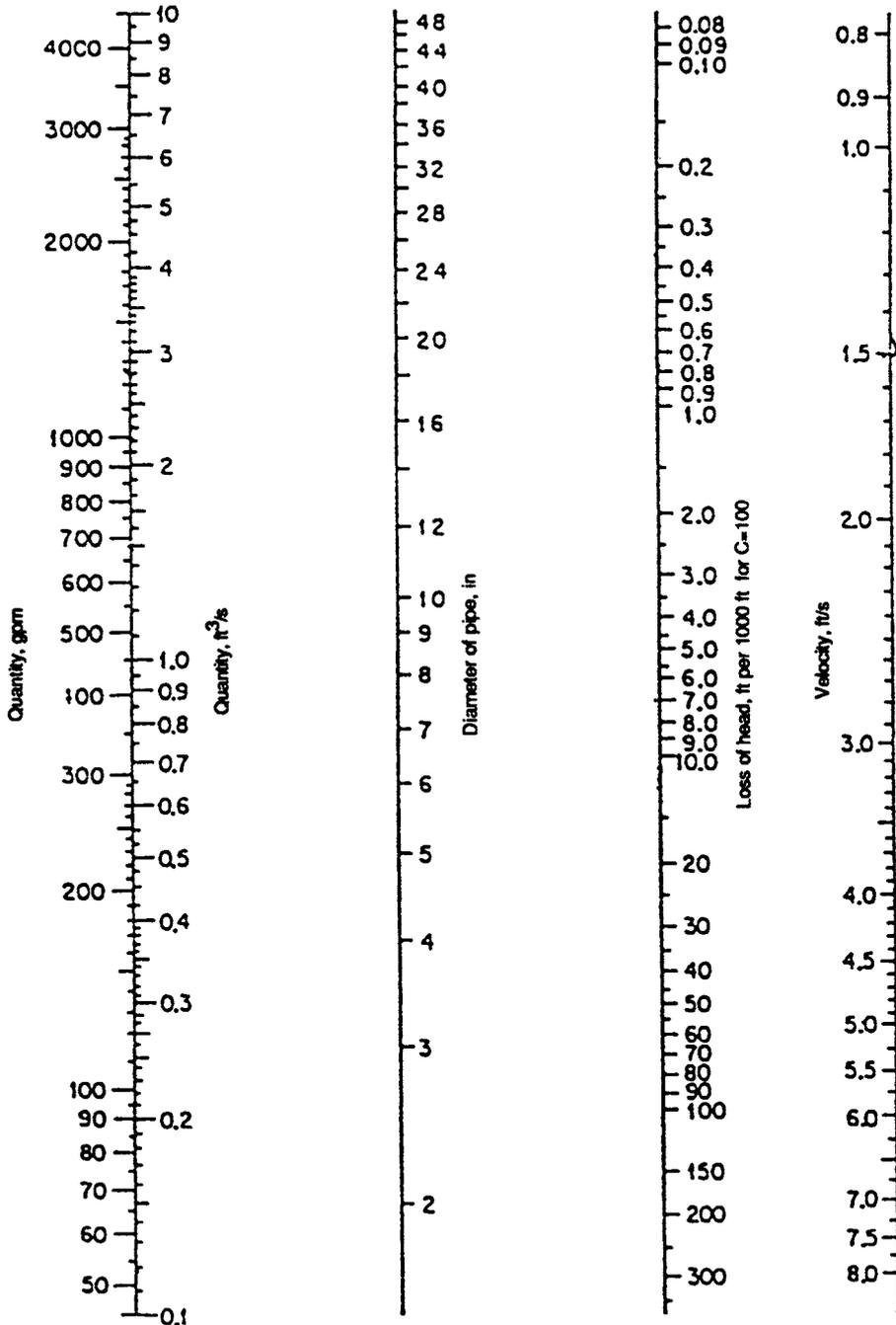


FIGURE 5.19 Nomograph for the Hazen-Williams formula for the flow in pipe (27).

TABLE 5.16 Values of the Hazen-Williams Coefficient C (25)

Conduit material	New	Uncertain
Cast iron pipe, coated with coal tar	130	100
Cast iron pipe, lined with cement or bituminous enamel	130*	130*
Steel, riveted joints, coated with coal tar	110	90
Steel, lock-bar joints, coated with coal tar	130	100
Steel, welded joints, coated with coal tar	140	100
Steel, welded joints, lined with cement or bituminous enamel	140*	130*
Concrete conduits	140	130
Wood stave pipe	130	130

*For use with the nominal diameter, i.e., diameter of the unlined pipe.

lines, changes in direction and grade should be made gradually. The angular deflection in successive lengths of cast iron and bell-and-spigot pipe should not exceed the values listed in Table 5.18 (25).

For riveted field joints on steel pipe larger than 30 in (76 cm) in diameter, the allowable angular direction is shown in Table 5.19 (25). Conduits that follow the surface of the ground are generally laid below the frost line. The following equation approximates Shannon's observations of the depth of the frost line (28):

$$d = 1.6SF^{-0.468} \quad (5.19)$$

where d is the depth of frozen soil in inches and F , the freezing index, is the algebraic difference between the maximum positive and negative cumulative departures ($T_d - 32$), of the daily mean temperature (T_d) from 0°C (32°F) (28). Accumulation begins on the first day on which a freezing temperature is recorded.

Material Selection. The selection of materials for pipelines and evaluation of their cost must take into consideration:

1. The initial carrying capacity of the pipe and its change with use, expressed, for example, in terms of the coefficient C in the Hazen-Williams formula.
2. The strength of the pipe, i.e., its ability to resist internal pressures and external loads.
3. The length of life or durability of the pipe, i.e., the resistance of cast iron and steel pipe to corrosion and of concrete pipe to erosion and disintegration.
4. The ease or difficulty of transporting, handling, and laying the pipe under different conditions of topography and geology.
5. The safety, economy, and availability of manufactured sizes.

TABLE 5.17 Allowances for Water Hammer in Cast Iron Pipes (28)

Diameter, in (cm)	Water hammer, lb/in ² (kgf/cm ²)
4–10 (10.2–25.4)	120 (8.44)
12–14 (30.5–35.6)	110 (7.73)
16–18 (40.6–45.7)	100 (7.03)
20 (50.8)	90 (6.33)
24 (81)	85 (4.12)
30 (76.2)	80 (3.88)
36 (91.4)	75 (3.63)
42–60 (106.7–152.4)	70 (4.92)

TABLE 5.18 Angular Deflections in Cast Iron Bell-and-Spigot Pipe (28)

Diameter, in (cm)	Angle	
	deg.	min.
4 (10.2)	4	0
6 (15.2)	3	30
8 (20.3)	3	14
10 (25.4)	3	7
12 (30.5)	3	0
16 (40.6)	2	41
18 (45.7)	2	26
20 (50.8)	2	9
24 (61)	1	47
30 (76.2)	1	26
36 (91.4)	1	12
42 (106.7)	1	2
48 (121.9)	0	55

6. The availability of skilled labor for the construction of the pipeline.
7. The requirements of maintenance and repair, losses of water by leakage, and other matters of pipe behavior and suitability.

The allowable leakage of bell-and-spigot and cast iron pipe that has been carefully laid and well tested during construction is given by the following empirical relationship (25):

$$Q = \frac{ndp}{1850} \tag{5.20}$$

where Q is the leakage in gallons per hour, n is the number of joints in the length of pipe tested, d is the nominal diameter of pipe in inches, and p is the average pressure during test in pounds per square inch, gage.

Appurtenances. To isolate and drain sections of supply conduits for tests, inspection, and/or repair or removal of silt, a number of appurtenances or auxiliaries may be needed as shown in Figure 5.20. These include:

- *Gates:* In pressure conduits, gate valves and generally placed at major summits.

TABLE 5.19 Angular Deflections in Riveted-Steel Pipes, Diameter Larger than 30 in (28)

Thickness of plate, in (cm)	Angle of bend, deg.
¼ (0.64)	6
⅝ (0.79)	5
¾ (0.95)	4
⅞ (1.11)	3½
1½ (1.27)	3

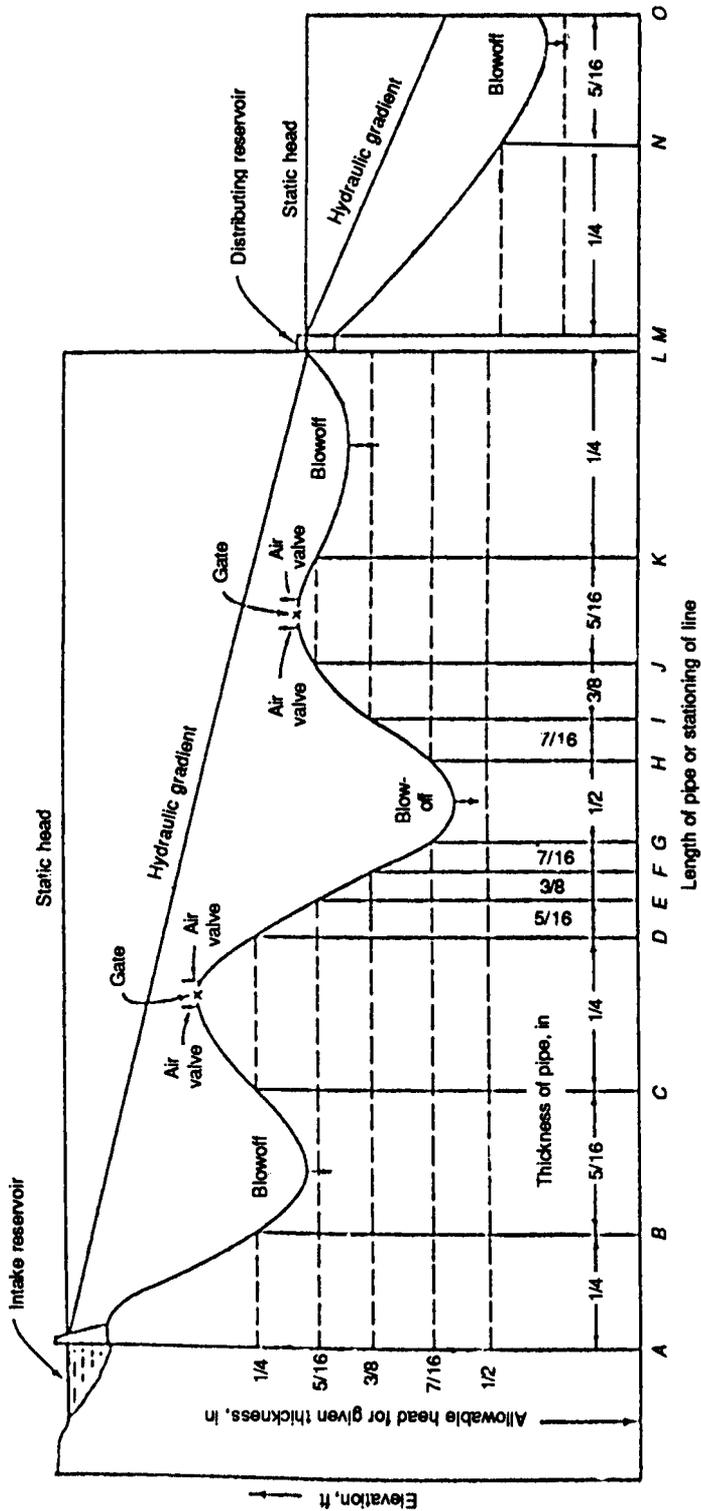


FIGURE 5.20 Profile of pipeline (25).

- *Blowoff valves*: In pressure conduits, small grated takeoffs are provided at low points in the line.
- *Air valves*: Cast iron and other rigid pipes and pressure conduits require air valves at all high points.
- *Manholes*: To provide access openings, manholes are spaced 1000 to 2000 ft (300 to 600 m) apart on large conduits.
- *Insulation joints*: Insulation joints are confined to metallic pipes.

WATER TREATMENT

With relatively few exceptions, drinking water sources require some kind of treatment before distribution to the consumer. This is particularly true for surface waters. Contaminants resulting from land erosion, dissolution of minerals, and the decay of organic vegetation have always been present in widely varying proportions in streams and have required removal to make the water potable. The need for such treatment is ever increasing because of the additional pollution contributed by an expanding industrial complex agricultural activities, and human population.

Objectives of Water Treatment

Four general objectives of drinking water treatment are to:

- Remove any toxic or health-hazard materials
- Remove or inactivate disease-producing organisms
- Provide a water of consistent quality
- Enhance the aesthetic acceptability of the water

Good engineering practice requires that these objectives be achieved with a reasonable factor of safety, and at reasonable cost. Defining treatment objectives and selecting appropriate control technologies involves eight basic considerations:

- Effluent requirements
- Adequate quantity
- Influent characteristics
- Existing system configuration cost
- Operating requirements
- Pretreatment and posttreatment components
- Waste management
- Future needs of the service area

Effluent Requirements and Influent Characteristics. Effluent requirements are usually specified by drinking water standards and regulations. Comparisons between influent characteristics and effluent requirements provide the basis for identifying treatment needs. The influent is properly characterized by a historical profile of the biological and chemical constituents of the source water. Parameters, such as pH, also are useful in characterizing the influent because they impact treatment process efficiency.

Existing System Configuration. The adaptability of an existing system to larger or different processes is an important consideration in assessing treatment requirements. Frequently, simple modifications to an existing system can substantially improve effluent as well as overall quality.

Cost. Treatment process costs are usually divided into capital and operating and maintenance costs. Frequently unit processes with high capital costs are associated with low operation and maintenance costs. Smaller systems usually have higher unit costs because they do not benefit from economies of scale.

Operating Requirements. Consistency of the quality and volume of the influent is a critical operating factor. If the influent is highly variable, increased monitoring and operator skills are required to maintain acceptable quality

Pretreatment and Posttreatment Processes. Water treatment systems perform differently with different pre- and posttreatment processes. The compatibility of all unit processes in the treatment train is important in achieving individual treatment goals. For successful implementation of a treatment technology, all elements of the treatment train must interact efficiently and effectively.

Waste Management. Waste management is a major concern associated with the removal of contaminants in drinking water. Most conventional treatment processes concentrate contaminants into sludge that frequently requires special handling. Extensive regulations cover the residuals that result from water treatment operation.

Future Service Area Needs. Population and economic forecasts of the service area are basic tools for evaluating future demands. These forecasts are used to predict the future of the service area and are a major factor in selecting a treatment technology.

Water Treatment Unit Processes

Natural waters, polluted either by human being or by nature, are likely to contain dissolved inorganic and organic substances, biological forms, such as bacteria and plankton, and suspended inorganic material. To remove these substances, the usual unit processes include plain sedimentation, removal by coagulation, generally followed by filtration, and chemical precipitation, which is used generally to remove dissolved minerals like hardness components and iron and manganese. Other processes, such as adsorption, aeration, ion exchange, oxidation, and distillation, are also important for the removal of dissolved substances.

The principal water treatment unit processes will be discussed in the following order: rapid mix, coagulation and flocculation, sedimentation, filtration, chemical feed and handling, disinfection, softening, sludge handling, ion exchange, adsorption, reverse osmosis, and aeration.

The standard unit processes that make up a conventional water treatment plant are: chemical feed, rapid mix, flocculation, sedimentation, filtration, and disinfection. Table 5.20 provides a summary description of these unit processes and identifies some of the problems associated with their application (145). A schematic flow diagram for a conventional water treatment process using the coagulation process is shown in Figure 5.21.

The water to be treated is brought to the rapid-mixing tank where destabilizing chemicals are added; vigorous mixing occurs for a short time, normally 1 mm or less. Usually one mixing tank is used, although, occasionally, two tanks may be used in series when two coagulants requiring separate addition are employed. Destabilization is fast and essentially complete after this rapid mixing. The water and its destabilized particles are then introduced into the flocculation tank, where gentle fluid motion brings the particles into contact so that aggregates can form. This gentle mixing is usually done mechanically, although hydraulic mixing is sometimes employed using baffled tanks. Detention times of 1 h are typical.

The rapid-mixing and flocculation tanks together bring about aggregation and make up the coagulation process. No materials are removed from the water in these tanks. In fact, materials are added in the form of coagulant chemicals. Solids are removed in subsequent settling and filtration facilities. These solid-liquid

TABLE 5.20 Conventional Water Treatment Unit Processes

Process	Process description	Potential causes of problems
Chemical feed	The three types of chemicals usually applied to the raw water at the beginning of the conventional treatment train are coagulants, coagulant aids, and pH control substances. Coagulants are chemicals used to remove turbidity and organic substances from raw water by precipitation. Coagulant aids are added to the influent after or simultaneously with the primary coagulant to improve flocculation performance. 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. Disinfectants, such as chlorine, are sometimes used at the front end of the treatment train, although in order to minimize the formation of disinfection by-products, chlorine is usually added at the end of treatment. More recently, ozone application as a pre-oxidant has been used to oxidize some organics to lower molecular weight.	Choice of chemical(s) Choice of chemical dose and pH Choice 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.
Coagulation/Rapid mix	Rapid mixing is generally the first stage of the treatment process. It is essential for quick and complete dispersion and mixing of the coagulant because the absence of complete mixing could result in excessive effluent turbidity. While some plants only have one rapid mixer to add coagulants, other plants have multiple rapid mixers to achieve optimum performance.	Type of rapid mix; in-line versus mechanical mix Number of rapid mixers Method of chemical addition Mixing speed/detention time
Flocculation	Flocculation usually follows the rapid mixing step in conventional treatment plants. It provides multiple opportunities for particles suspended in water to collide and aggregate through gentle and prolonged agitation. This process takes place in a basin equipped with a mixer.	Optimum detention time Optimum mixing intensity Number of stages flow conditions
Sedimentation	Sedimentation follows flocculation and precedes filtration, and its purpose is to enhance the filtration process by removing particulates. Sedimentation requires that water flow through the basin at a velocity slow enough to permit the particles to settle to the bottom before the water exits the basin.	Surface loading rates Short circuiting due to wind, temperature, density differences, inlet and outlet design Amount and rate of accumulation of sludge Sludge removal
Filtration	Filtration is the process of passing settled water through a porous medium to remove any remaining matter held in suspension. In water purification, the matter to be removed is colloidal in size and includes suspended silt, clay, organic colloids, and microorganisms, including algae, bacteria, and viruses.	Filter rate and rate control Hydraulics Chemical pretreatment of water reaching the filter Inadequate backwashing
Disinfection	Disinfection is a process that kills or inactivates pathogenic microorganisms that occur in drinking water. The most common disinfectant used in the United States is chlorine. Other types of disinfectants are chloramines, chlorine dioxide, and ozone.	Adequate dosage Adequate contact time

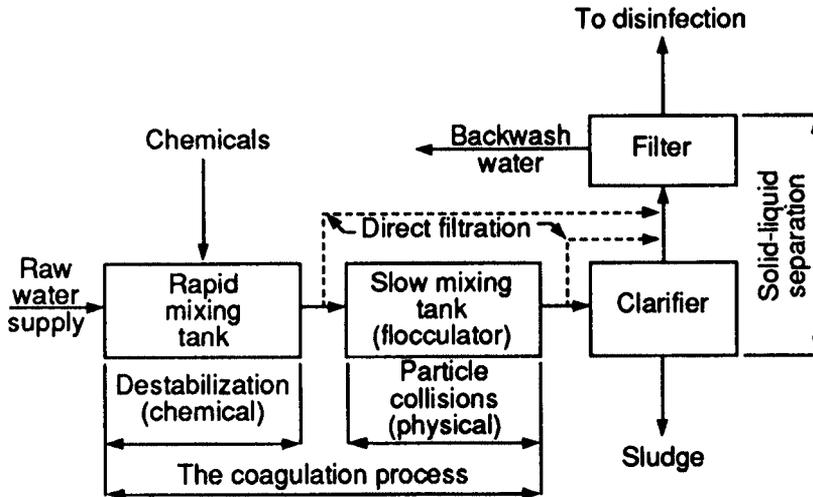


FIGURE 5.21 Schematic diagram of a conventional water treatment (29).

separation processes must remove the particles present in the original raw water and the chemicals added to bring about coagulation. The solids leave the treatment system in sludge from the settling tanks and in backwash water from the filters. Disposal of these water treatment plant wastes is a problem in itself. Here it is important to note that the characteristics of these wastes (solids concentration, quantity, dewatering capability) are functions not only of the raw water supply but also of the materials used as coagulants. Disinfection provides another barrier to waterborne disease.

Rapid Mix

Rapid, flash, or quick mix is the unit process used to generate a homogeneous mixture of raw water and coagulants which results in the destabilization of the colloidal particles in the raw water to enable coagulation. Typically, pumps, venturi flumes, air jets, or rotating impellers (paddles, turbines, or propellers) are used to achieve the high-energy mixing conditions required in this process. The vertical shaft turbine impeller is widely used for rapid mixing. It is composed of a vertical shaft driven by a motor with one or more curved blades. Table 5.21 contains typical design parameters for rapid mix.

Rapid or flash mixing is the process by which a coagulant is rapidly and uniformly dispersed through the mass of the water. This process usually occurs in a small basin immediately preceding or at the head end of the “coagulation basin.”

TABLE 5.21 Design Parameters for Rapid Mix

Item	Value
Mixing intensity	700–1000 s ⁻¹
Detention times	1–2 min
Power/unit volume	1–2 hp/ft ³ /s (26.3–52.6 kw/m ³ /s)
Basin dimensions (diameter)	3–10 ft (0.9–3 m)

It is necessary to examine rapid mixing in terms of the coagulation reactions. Figure 5.22 summarizes in schematic form the predominant mechanisms in coagulation. An analysis of the kinetics of the reactions is especially important. Coagulation in water treatment occurs predominantly by two mechanisms: (1) adsorption of the soluble hydrolysis species on the colloid and destabilization, and (2) sweep coagulation where the colloid is entrapped within the precipitating aluminum hydroxide. The reactions in adsorption-destabilization are extremely fast and occur within microseconds without formation of polymers, and within 1 s if polymers are formed (30, 31). Sweep coagulation is slower and occurs in the range of 1 to 7 s (32).

For the adsorption-destabilization process, it is imperative that coagulants be dispersed in the raw water stream as rapidly as possible. A plug flow tubular reactor, such as in-line blenders, with the chemicals injected into the fluid stream will be most efficient. For sweep coagulation, extremely short dispersion times are not as important as in adsorption-destabilization, since coagulation is due to colloid entrapment amidst the aluminum hydroxide precipitate.

Chemical Stream

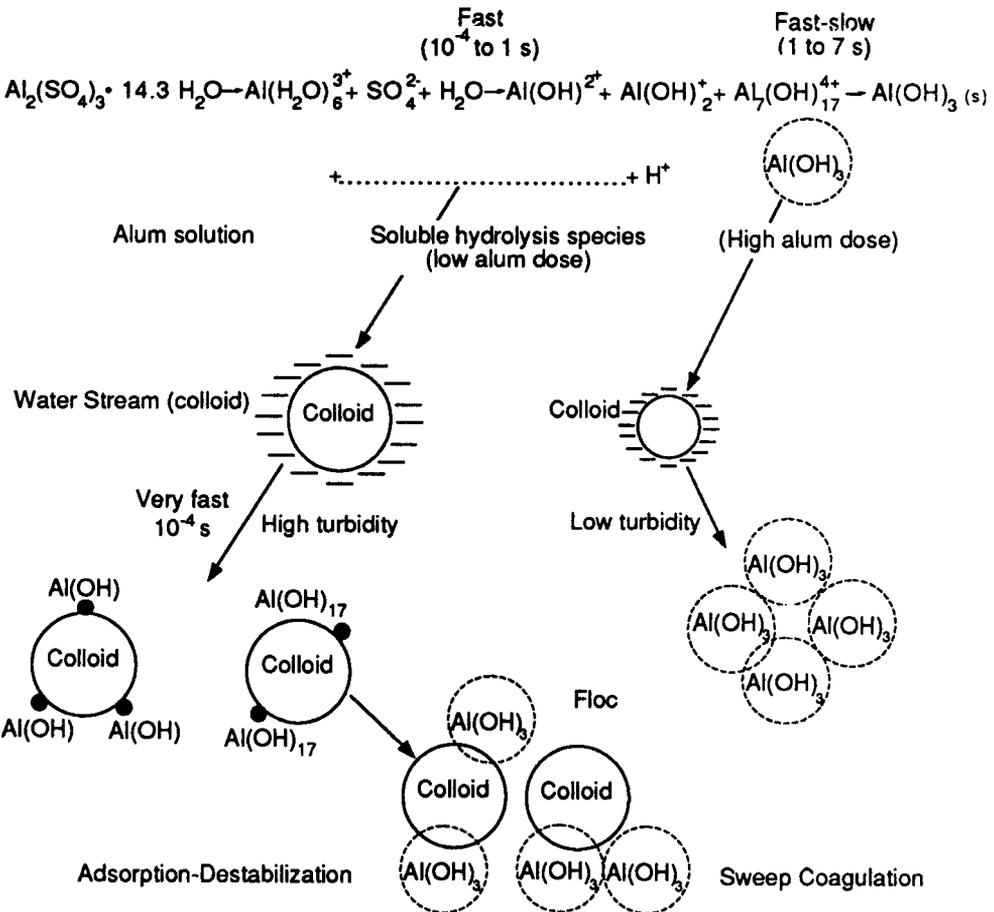


FIGURE 5.22 Reaction schematics of coagulation (39).

Mixing for the rapid dispersion of chemicals and other materials throughout the water is best accomplished by generating intense turbulence in the water. In this turbulence, small-scale eddies form frequently and disappear rapidly. Their frequent formation and disappearance provides the means for rapid transportation of the added material to all parts of the water; and the small scale of the turbulence assures a uniformity of dispersion not accomplished with large-scale eddies which can transport large masses of water from one area to another without effecting substantial mixing. Through the years, many devices have been relied upon to provide the mixing needed for chemical dispersion: baffled chambers, hydraulic jumps, and mixing chambers containing impeller or propeller mixers. As might be expected, these devices have not proven themselves to be equally effective. Generally speaking, the less-effective devices are found wanting in one or more of the following key elements: high rate of power dissipation, small-scale turbulence, and flexibility of operation.

No clear-cut guidelines exist for determining the power dissipation or detention time required to disperse chemicals in the flow of water. However, detention times of 20 s or less are not uncommon, and mixing units providing 1 to 2 hp (0.75 to 1.49 kW) for each cubic foot (cubic meter) per second of flow rate are not unreasonable (33).

A parameter frequently used to express the power input is the temporal mean velocity gradient G (which has the dimension of 1/time):

$$G = \left(\frac{P}{\mu V} \right)^{1/2} \quad (5.21)$$

Here P is the power dissipated in the water, W (ft·lb/s); V is the volume of the tank or basin, or the volume of water to which the power is applied, ft³ (m³), and μ is the absolute viscosity of the water, lb·s/ft² (N·s/m²) (34).

The required degree of turbulence for rapid mixing is provided by a number of different mechanisms. The importance of flexibility in operation cannot be overstressed. Reliance upon conduit flow, process pumps, baffled chambers, etc., to accomplish mixing leaves the plant operator with little or no opportunity to adjust the operation to suit the treatment needs. Instead, mixing conditions are determined by the rate at which the water is processed.

Because of their many desirable features, propeller, impeller, or turbine mixers have gained the widest acceptance in treatment-process design. These units are often placed in small chambers or tanks in which the agitation is well confined and the detention time short. In some instances, impeller or turbine mixers have been inserted in pipes or conduits, thereby eliminating the need for a separate tank or chamber. Three arrangements for rapid mixing are shown in Figure 5.23.

Coagulation

Coagulation, generally followed by filtration, is by far the most widely used process to remove the substances producing turbidity in water. These substances normally consist largely of clay minerals and microscopic organisms and occur in widely varying sizes, ranging from those large enough to settle readily to those small enough to remain suspended for very long times.

Coagulation is the chemical process in which particle charge is satisfied and flocculation is the physical process that agglomerates particles (too small for gravitational settling) so that they may be successfully removed during the sedimentation process.

Coarser components, such as sand and silt, can be removed from water by simple sedimentation. Finer particles, however, will not settle in any reasonable time and must be flocculated to produce the larger particles that are settleable. The long-term ability to remain suspended in water is basically a function of both size and specific gravity. The importance of size is illustrated in Table 5.22, which shows the relative settling times of

WATER SUPPLY

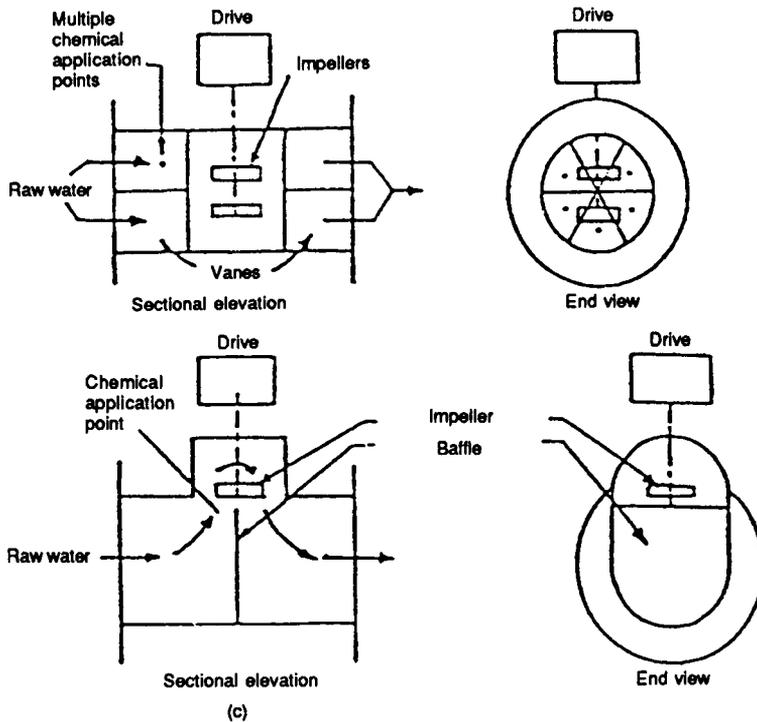
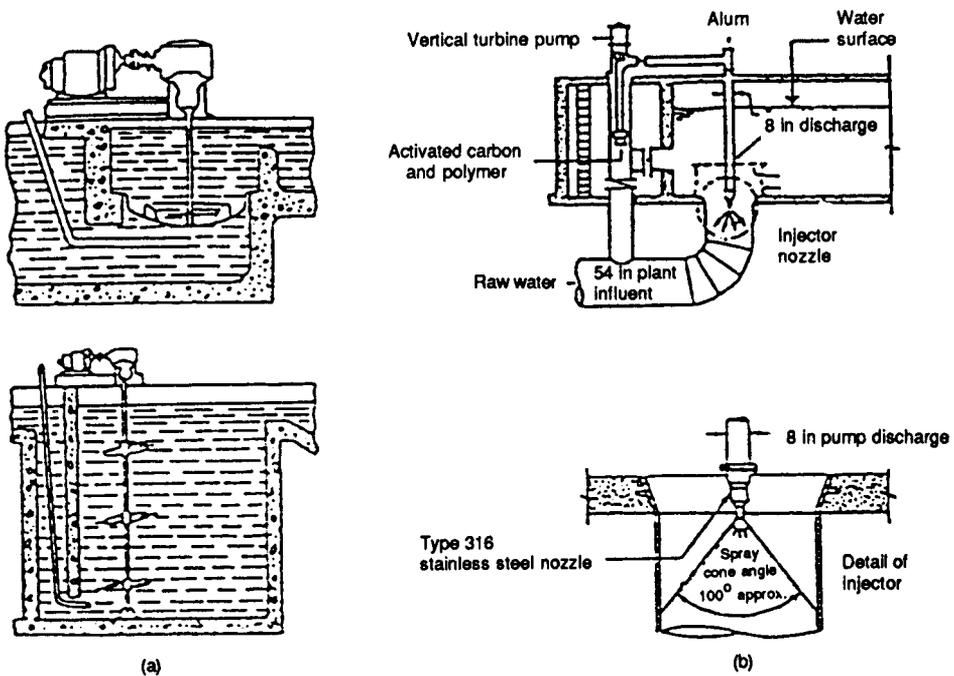


FIGURE 5.23 Rapid mixers—design alternatives (a) Mechanical mixers; (b) injection mixer; (c) inline blenders (39).

5.74

TABLE 5.22 Effect of Decreasing Size of Spheres

Diameter of particle, mm	Order of size	Total surface area *	Time required to settle [†]
10	Gravel	0.487 in ² (3.14 cm ²)	0.3 s
1	Coarse sand	4.87 in ² (31.41 cm ²)	3 s
0.1	Fine sand	48.7 in ² (314.12 cm ²)	38 s
0.01	Silt	3.38 ft ² (0.31 in ²)	33 min
0.001	Bacteria	33.8 ft ² (3.14 in ²)	55 h
0.0001	Colloidal particles	338 ft ² (31.40 in ²)	230 days
0.00001	Colloidal particles	0.7 acre (0.28 ha)	6.3 yr
0.000001	Colloidal particles	7.0 acres (2.83 ha)	63 yr minimum

*Area for particles of indicated size produced from a particle 10 mm in diameter with a specific gravity of 2.65

[†]Calculations based on sphere with a specific gravity of 2.65 to settle 1 ft.

Source: From Ref. 35.

spheres of different sizes (35). It can be seen that the settling rates of the colloidal and finely divided (approximately 0.001 to 1 μm) suspended matter are so slow that removing them from water by plain sedimentation in tanks having ordinary dimensions is impossible. The enormous increase of surface area for a given weight of solids as the particles become smaller and more numerous is an important property of colloids, which is discussed later.

Substances producing color, as distinct from turbidity, consist either of colloidal metallic hydroxides, iron for example, or of organic compounds having a much smaller particle size. These substances, too, can be removed by coagulation, which serves to agglomerate the very small particles into sizes that are settleable or can be removed by filters.

The process of coagulation may also find use, although not always, in the softening of hard water with lime or lime and soda ash. Softening is more properly a precipitation process, and coagulation is used to obtain a more rapid and complete settling of the precipitated hardness components.

A turbidity limit of 0.5 ntu is accepted as water quality for high quality finished water. This limit is readily obtainable by properly performed coagulation and filtration.

Even a high-quality treated water that meets the recommendation of residual turbidity of 0.1 unit still contains enormous numbers of particles. These either represent uncoagulated primary clay particles or fragments of coagulant floc that have passed through the filters.

Optimum coagulation treatment of a raw water represents the attainment of a very complex equilibrium in which many variables are involved. Thus, for a given water, there will be interrelated optima of conditions, such as pH, turbidity, chemical composition of the water, type of coagulant, and such physical factors as temperature and mixing conditions. These interrelations are so complex that at the present time it is impossible to predict from purely theoretical bases the optimum coagulant dose for any particular water. The proper dose and physical conditions for coagulation must be determined empirically for each water by methods that will be discussed later. Notwithstanding these difficulties, a knowledge of the effects on coagulation of the several variables can lead to a more intelligent application of empirical methods.

Destabilization of charged particles in water occurs as a result of the chemical added. The selection of type and dosage of the chemical coagulant must be made by experimentation, most commonly with jar tests. The main chemical factors that have been found to influence coagulation of natural waters are type of coagulant, pH of raw water, concentration of humic substances (especially colloids), and alkalinity.

Commonly used coagulants include those which are iron- or aluminum-based, lime, and polymers. Aluminum sulfate, commonly known as alum, is effective for pH values of 5.5 to 8.0. Sodium aluminate is used in special cases or as an aid for secondary coagulation of highly colored surface waters and in lime

soda softening to improve settling. Ferrous sulfate in conjunction with lime is effective in the clarification of turbid waters and other reactions which have a high pH, such as lime softening. Ferric sulfate reacts with alkalinity and is effective over a wide pH range. It removes color at a low pH and iron and manganese at a high pH. Ferric chloride reacts with alkalinity also but has limited use in water treatment. Polymers act well by producing floc.

Many coagulant aids are available to help with difficulties encountered with slow-settling flocs. Polyelectrolytes are synthetic polymers of varying charge which, when applied in dosage from 0.1 to 1.0 mg/L, can reduce primary coagulant doses with increased cost but make sludge easier to handle. Activated silica increases the rate of chemical reactions occurring during coagulation, reduces coagulant dose, extends optimum pH range, induces faster settling, and produces tougher floc. However, a precise dosage of activated silica is required. Bentonite clay treats high-color, low-turbidity, and low-mineral waters. It increases floc attraction and absorbs 10 to 15 mg/L of the organics present. Wetting agents, to speed up floc settling, which may be applied include powdered silica, limestone, and activated carbon.

Early investigators of the coagulation process in water treatment showed that pH was the single most important variable of the many that had to be considered. These investigators clearly established that there is at least one pH range for any given water within which good coagulation-flocculation occurs in the shortest time with a given coagulant dose. The extent of the pH range is affected by the type of coagulant used and by the chemical composition of the water as well as by the concentration of coagulant. Whenever possible, coagulation should be carried out within the optimum pH zone. Failure to operate within the optimum zone for a given water may be wasteful of chemicals and may be reflected in lowered quality of the water-plant effluent.

The metal coagulants, aluminum and iron salts, have been shown to precipitate and coagulate most rapidly and with minimum solubility in some characteristic pH range, depending on the particular coagulant. Extensive and continuing investigations beginning in the early 1920s and extending to the present have shown that the pH zone of least solubility for the hydrolysis products of aluminum ranges from 5.5 to 7.8. In practice, in the presence of ions and turbidity normally occurring in natural waters, the pH range is generally 6.0 to 7.8. Iron salts behave similarly, although the pH zone of coagulation is generally broader.

Interrelations between pH and required alum dosage to coagulate clay, residual aluminum, and turbidity of precipitated coagulant are shown in Figure 5.24 (36). Curve A shows that the minimum dose to remove 50 mg/L clay turbidity occurs within a pH range of 6.8 to 7.8; residual aluminum after treatment with 200 mg/L of alum, shown in curve B, is minimal at a pH of between 5.5 and 7.5; and flocculation of aluminum hydroxide in the absence of clay, shown in curve C, occurs most rapidly at a pH of about 7.2. The data represented in the curves shown in Figure 5.24 have been cited as evidence that aluminum sulfate is most efficient as a coagulant of dilute clay suspensions under conditions which produce the most rapid precipitation and flocculation of its hydrolysis products. Thus, experimental data corroborate the theoretical concept that the coagulating agent is hydrolysis products and not trivalent aluminum ion. Curve C, representing coagulation with no clay turbidity present, also demonstrates that coagulation and flocculation of alum alone will occur with no dependence on turbidity to serve as either nuclei or electrical charge sites.

Coagulation should, if possible, be carried out within the optimum pH zone for the particular water. For certain waters it may be necessary to adjust the pH with acid, lime, soda ash, etc., to obtain the proper conditions. Addition of excess coagulant may in some plants be a more practical way to reduce pH to the favorable range than separate acid addition. With optimum pH conditions, the soluble residual aluminum concentration in the effluent will be exceedingly small. However, under practical conditions of coagulation, colloidal-insoluble particles of aluminum hydroxide may appear in the effluent in concentrations of aluminum much higher than the upper value indicating incomplete flocculation or penetration of alum floc through the filters.

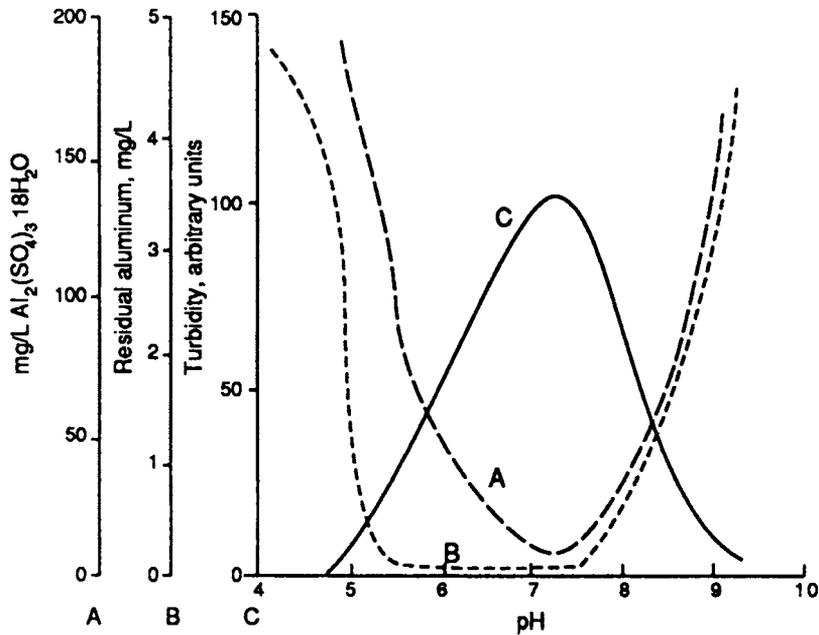


FIGURE 5.24 Relationship of pH to coagulation (36).

Flocculation

Flocculation usually occurs by mechanical stirring, producing mass fluid motion which is commonly referred to as the orthokinetic flocculation unit process. During flocculation, slow-moving paddle mixers gently stir a mixture of water and coagulant to generate floc. Due to the increased fragile nature of the floc as it grows in size, a series of flocculation chambers is usually employed rather than a single flocculation basin. The chambers are designed to enhance laminar flow conditions to prevent floc destruction in conjunction with sufficient mixing to achieve floc formation. A stepped-down mixing intensity is utilized in each successive chamber. Typical mixing intensities range between 10 and 100 s^{-1} . Flocculation time is also a governing factor in floc formation and is generally expressed along with mixing intensity as a product (Gt). Acceptable Gt factors range between 10^4 and 10^5 . Mixing intensity is governed by basin size and configuration, number of paddles and their size, and power input to permit floc formation and prevent floc destruction.

The flocculation stage in the water treatment process train is the aggregation or growth of the destabilized colloidal suspension. Flocculation follows, and in some instances overlaps, destabilization. Destabilization is essentially controlled by the chemistry of the process which flocculation is the transport step that results in collisions between destabilized colloidal particles, leading to the formation of flocs.

Two major mechanisms of flocculation are generally identified: (1) perikinesis, which is aggregation resulting from random thermal motion of fluid molecules and is significant for particles less than 1 to 2 μm , and (2) orthokinesis, which is induced by velocity gradients in the fluid. The latter is the predominant mechanism in water treatment. In addition, differential and fluctuating velocities can also cause contacts between particles, leading to aggregation. This is a major mechanism in sludge blanket or solids contact clarifiers. A knowledge of the kinetics of the process of orthokinetic flocculation forms the basis for the

design of the flocculators in a water treatment plant. The parameters that determine the rate at which aggregation occurs define the design dimensions of the process tank and its equipment.

Figure 5.25 shows various types of clarifiers with sludge blanket flocculation (37). Figures 5.26 and 5.27 show typical horizontal and vertical flocculation basins (38).

Typical design data for flocculators are given in Table 5.23.

Optimum mixing intensity can be calculated using the relationship shown below (39):

$$T(G^*)^{2.8} = \frac{44 \times 10^5}{C} \quad (5.22)$$

Where G^* = optimum mixing intensity, s^{-1}

C = alum or equivalent alum dose, mg/L

T = flocculation time, min

44×10^5 = empirical constant

If the calculated G value is less than $1 s^{-1}$, adequate mixing will not be accomplished. This condition may be corrected by decreasing detention time or through modification of chemical dose.

Sedimentation

Sedimentation or clarification is the removal of particulate matter, chemical floc, and precipitates from suspension through gravity settling (40). This makes water clarification a vitally important step in the treatment of surface waters for potable supply and in most cases the main factor in determining the overall cost of treatment. Poor design of the sedimentation basin will result in reduced treatment efficiency that may subsequently upset other operations (41).

Surface water containing high turbidity may require sedimentation prior to chemical treatment. Presedimentation basins are constructed in excavated ground or out of steel or concrete. Steel and concrete tanks are often equipped with a continuous mechanical sludge removal apparatus. The minimum recommended detention time for presedimentation is 3 h, although at many times of the year, this may not be adequate to remove fine suspensions. Chemical feed equipment may be provided ahead of presedimentation to provide prechlorination or partial coagulation for periods when water is too turbid to clarify by plain sedimentation (42).

Settling basins are usually provided for chemical coagulation or softening. These basins may be constructed of concrete or steel in a wide variety of shapes and flow mechanisms. Some possibilities of shapes and usual dimensions are presented in Table 5.24.

To minimize the effects of short-circuiting and turbulent flow, care is taken in the effective hydraulic design of inlet and outlet structures in all tanks. Inlet structures are expected to: (1) uniformly distribute flow over the cross section of the settling zone, (2) initiate parallel or radial flow, (3) minimize large-scale turbulence, and (4) preclude excessive velocities near the sludge zone. Flow through a sedimentation basin enters at the top of the basin. In some circular basins, the flow may enter a central flocculating chamber of the basin. Effluent flows vertically out over perimeter weirs. In rectangular tanks water flows horizontally to an end weir for discharge. An efficiently designed vertical tank is more stable than a horizontal one. The rating of a vertical tank as related to the bulk settling velocity is usually 3.3 to 9.8 ft/h (1 to 3 m/h) (39).

Recent developments have been made with the implementation of tube settling in the design of horizontal-flow settling tanks, which are said to overcome some of the previous difficulties and enable higher rates of sedimentation. Upflow solids contact clarifiers, and horizontal-flow basins may increase their volumetric capacity 50 to 150% or more with the introduction of inclined tubes into their basins. Similar or improved effluent quality is observed in these modified processes.

Effluent quality for any sedimentation process is dependent upon previous water quality, coagulants

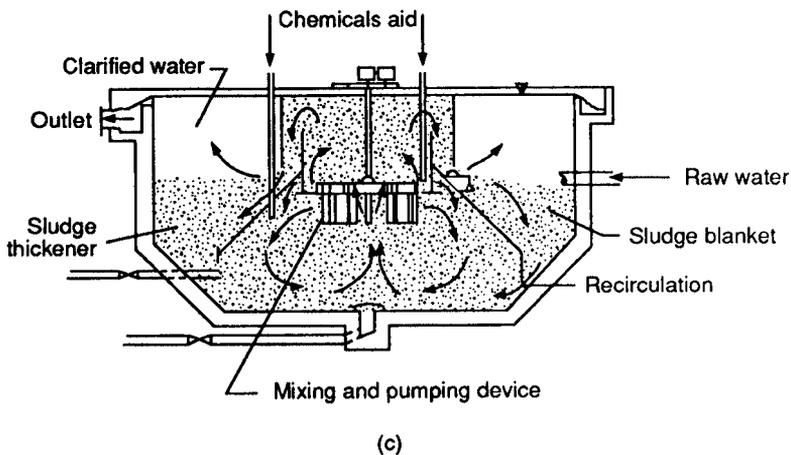
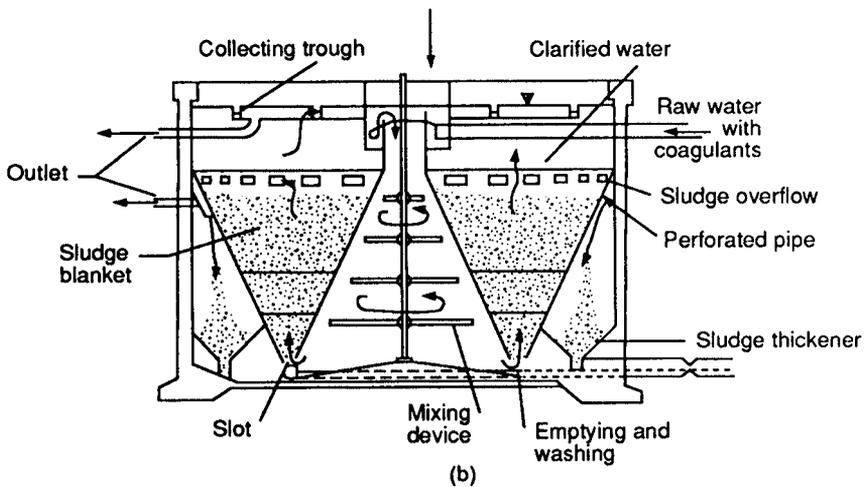
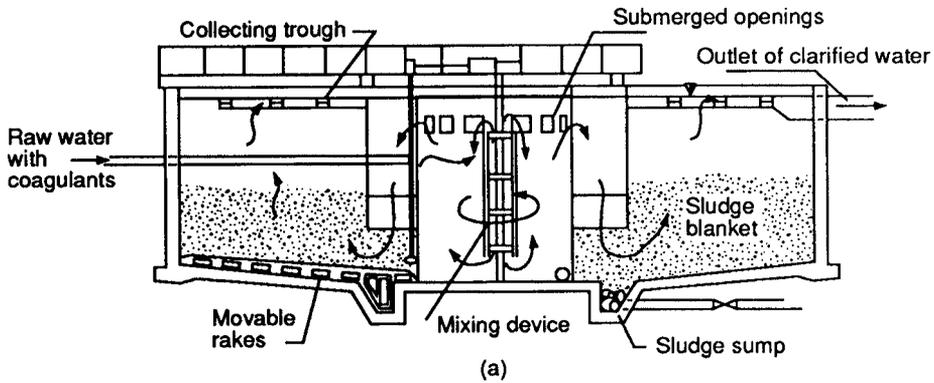
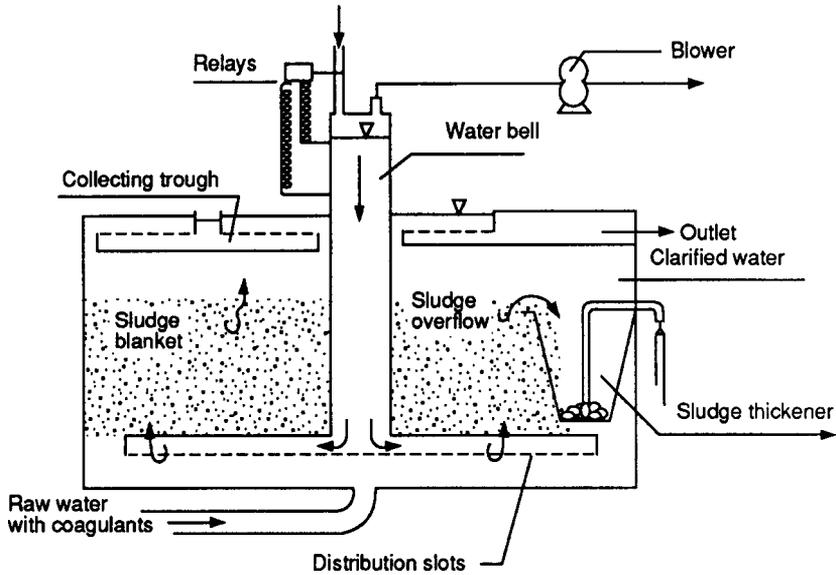
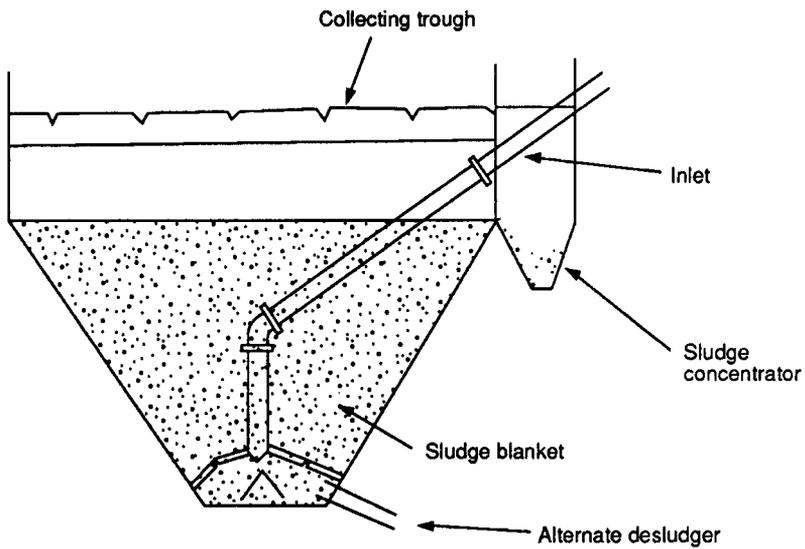


FIGURE 5.25 Clarifiers with sludge blanket flocculation. (a) Mechanically agitated bed clarifier; (b) hydraulically fluidized bed clarifier; (c) sludge circulation clarifier (37).



(d)



(e)

FIGURE 5.25 (continued) (d) Unsteady discharge clarifier; (e) hopper-bottom hydraulically fluidized clarifier (37).

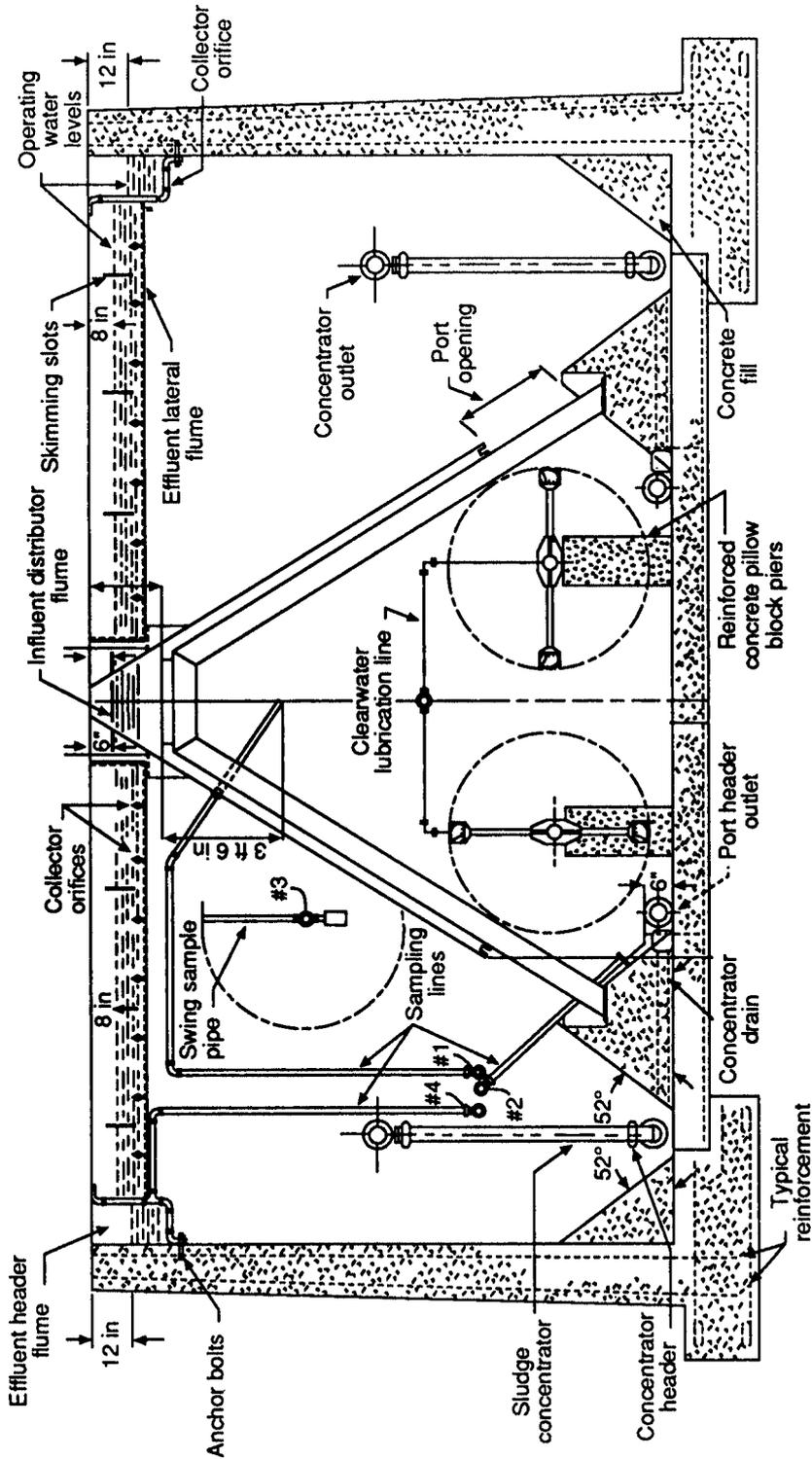


FIGURE 5.26 Horizontal sludge blanket clarifier (37)

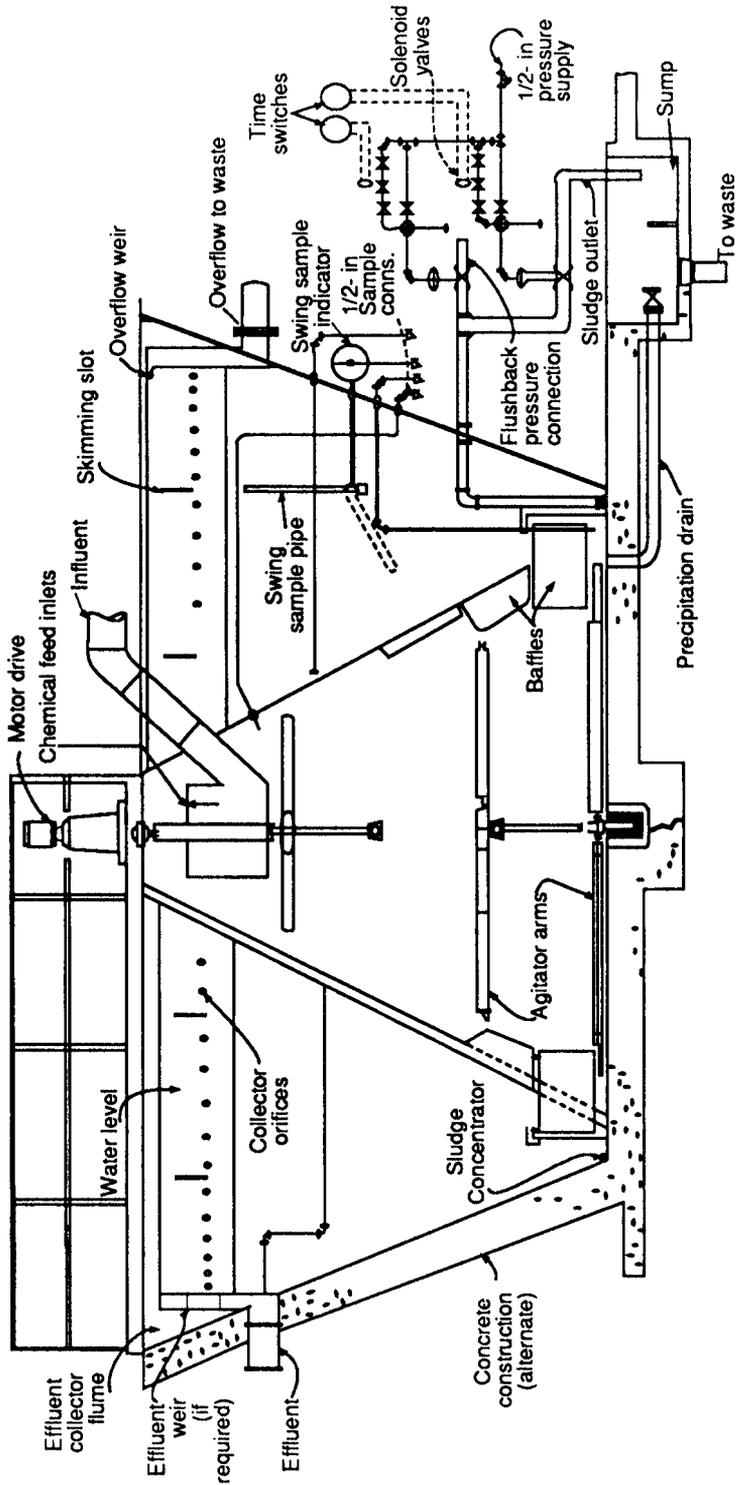


FIGURE 5.27 Vertical sludge blanket clarifier (38)

TABLE 5.23 Flocculation Basin Design Data

Parameter	Typical value
Configuration	Rectangular
Length to width ratio	4:1
Minimum number	2 basins
Maximum stage volume	12,500 ft ³ (304 m ³)
Depth	12 ft (3.6 m)
Common wall construction	For up to 80 basin stages
Horizontal mixing	Volume between 1860 and 1×10^6 /ft ³ (53 and 28×10^3 m ³)
Vertical mixing	Volume between 18,000 and 25,000 ft ³ (509 and 707 m ³)
Mixing intensity (<i>G</i>)	20 to 80 s ⁻¹
Variable-speed motors	60% efficient
Freeboard and mixing apparatus	Require 20% of tank volume
Vertical mixers	3-blade propeller impeller with Reynolds number max of 10,000
Horizontal (paddles) mixers	8 (4 arms with 2 paddles)
Paddle tip velocity	Less than 2 ft/sec (0.61 m/sec) for weak floc and 4 ft/sec (1.22 m/sec) strong floc

added, mixing, flocculation, and filtration. With the exception of presedimentation, sedimentation processes are usually preceded by coagulation and followed by filtration. Therefore, typical removals of regulated contaminants by sedimentation and filtration are reported according to coagulants used in conjunction with these processes for removal of a specific contaminant.

Basically, the theory of sedimentation is the theory of the effect of gravity on particles suspended in a liquid of lesser density. Under the influence of gravity, any particle having a density greater than 1.0 will settle in water at an accelerating velocity until the resistance of the liquid equals the effective weight of the particle. Thereafter, the settling velocity will be essentially constant and will depend upon the size, shape, and density of the particle, as well as the density and viscosity of the water. For most theoretical and practical computations of settling velocities in sedimentation basins, the shape of the particles is assumed to be spherical. Settling velocities of particles of other shapes can be analyzed in relation to spheres.

Number of Basins. Probably the most important considerations in the selection of the number of sedimentation basins are: (1) the effect upon the production of water if one basin is removed from service, and (2) the largest size that can be expected to produce satisfactory results. Plant layout and site conditions also may influence the number of basins. For any supply which requires coagulation and filtration for the production of safe water, a minimum of two basins should be provided. The greater the number available,

TABLE 5.24 Design Data for Sedimentation

Parameter	Value
Depth	8 to 12 ft (2.4 to 3.6 m)
Circular configuration	30 to 200 ft (9 to 61 m)
Square configuration	<100 ft (30.48 m) in length
Length to width ratio	3:1 to 5:1
Bottom slopes	1% in rectangular tanks; 8% in circular or square tanks

the less will be the effect upon velocity and detention period in the remaining basins should one be out of service for cleaning, repairs, or any other reason.

Size and Shape of Basins. The most common forms of sedimentation basins are circular, square, or rectangular. The selection of the particular form or shape for a given plant depends upon area available, conformity with adjacent structures, and the theories and experience of the design engineer. The majority of sedimentation basins now in use are rectangular in shape and of reinforced concrete construction. Circular basins of either concrete or steel have been used widely for both coagulation and softening.

Most sedimentation basins are provided with sloping bottoms to facilitate the removal of deposited sludge. Although the optimum depth of a basin is a controversial matter, a review of the design of the larger rapid-sand filtration plants indicates a range of 8 to 16 ft (2.43 to 4.88 m) in depth.

Multiple-story basins, in which the water travels horizontally along one level of a basin and then passes vertically (preferably upward) to another level of the basin for more horizontal flow, have been used successfully to compress a large amount of basin surface (bottom) area into a small area of the plant site. In this way, low values of the surface-loading parameter can be attained, producing high removal efficiencies. Since long, narrow tanks tend to have better hydraulic stability characteristics (less tendency to short-circuit) than short, wide ones, some designers place the several stories or levels in series rather than parallel. Although the same value for the surface-loading parameter can be attained with either flow arrangement, the parallel arrangement avoids the upsets caused by series arrangements in the area where flow direction reverses.

A recent innovation, the “tube” settler, uses parallel flow upward through tubes about 2 in (5 cm) square inclined at an angle of 60° from the horizontal to attain extremely high surface areas(39). The tubes are short, 2 to 4 ft (0.6 to 1.2 m), and the detention time for water in the “settler” is between 3 and 6 min. The 60° angle inclination permits accumulated sludge to slide down the tube wall to a sludge collector area. An alternate design uses 1-in (2.5-cm) hexagonal tubes inclined at 5° from the horizontal. In this case, the tube settler requires separate provision for sludge removal, commonly by backwashing with filter washwater. The two arrangements are shown in Figure 5.28. The steeply inclined tubes are constructed of plastic in modules which can be readily installed in existing sedimentation basins.

Inlet Arrangements. The purpose of inlets is to distribute the coagulated water uniformly among the basins and uniformly over the cross section of each sedimentation basin. Properly designed inlets and outlets are necessary to avoid short-circuiting through a basin. The most efficient and satisfactory inlet is one that permits water to enter directly into the sedimentation basin without the use of pipelines or flumes. As treatment plants operate over a relatively wide range of flow, velocities occurring in the inlet pipeline or flume may be either so low that settling takes place or so high that a floc is broken before it reaches the basin. The permissible velocity range for any water and floc can be determined by tests, but velocities between 0.5 and 2 ft/s (0.15 and 0.61 m/s) are usually satisfactory.

Where inlet pipelines or flumes are required, several methods have been used to give uniform distribution of the floc, as shown in Figure 5.29. To obtain uniform flow through each of the inlet pipes or orifices, the head loss at each opening should be large compared to the maximum difference of energy head available at the inlets. Velocities, however, must be maintained sufficiently low to prevent breaking up of floc.

Short-Circuiting. The use of baffles in sedimentation basins to reduce shortcircuiting and improve settling efficiency is common to many plants. Although such an arrangement does tend to correct short-circuiting, it may form dead spaces, produce eddy currents, and cause disturbance of the deposited solids. Another expedient for reducing short-circuiting is the use of longitudinal round-the-end baffles, which also serve the purpose of bringing water in at one end of the basin and returning it to that end before it goes on to the next step in the treatment process. Intermediate baffling has been less popular in recent years because of increased costs of construction and the use of mechanical mixing and flocculation to produce a floc having superior settling characteristics.

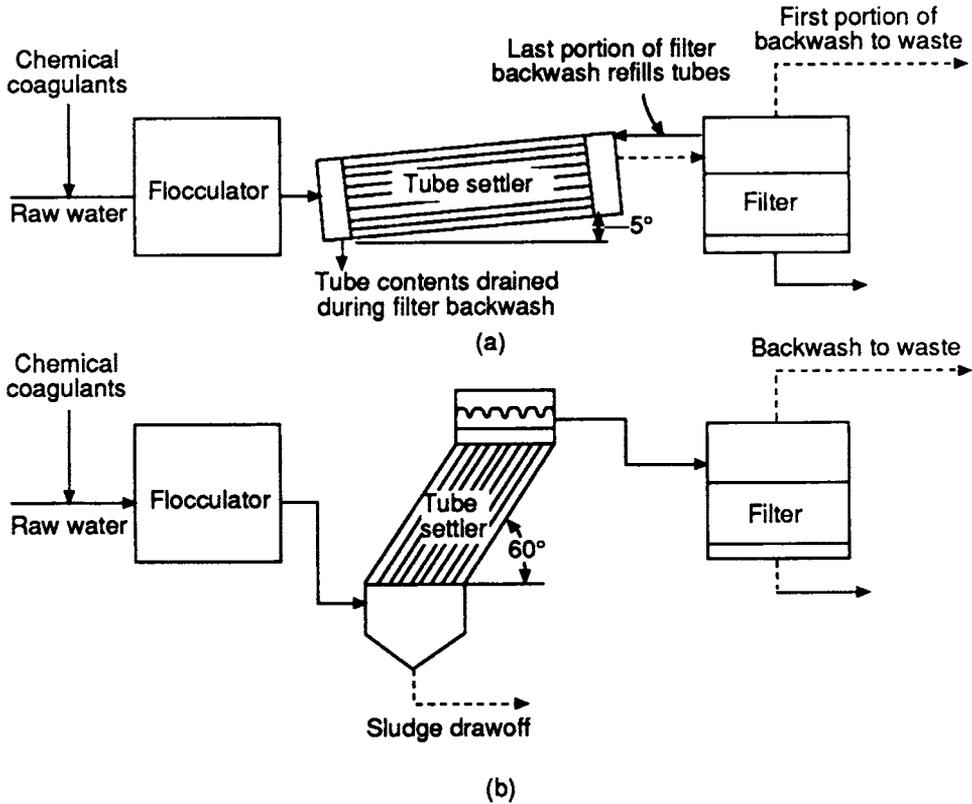


FIGURE 5.28 Basic tube settler configurations (40).

Outlet Arrangements. Water leaving the sedimentation basin should be collected uniformly across the width of the basin to prevent high velocities of approach and consequent lifting of the settled sludge over the weir.

Weirs may be constructed across the basin, or slots or effluent ports provided. Combinations of effluent orifices ahead of the submerged weir provide efficient outlet arrangements and reduce short-circuiting. In some designs, collecting weirs and flumes which begin at the middle of the basin in order to give great weir

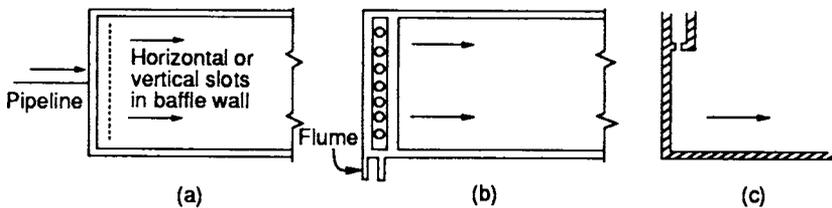


FIGURE 5.29 Inlet arrangements, (a) Plain view of basin with pipeline inlet; (b) plain view of basin with flume inlet; (c) section view of (b) (33).

length have been constructed. Insufficient length results in excessive overflow rates and consequent high velocity of approach to the weir. Frequently, in basins of insufficient weir length, settled solids are pulled from the floor of the basin by high velocity of approach, nullifying the good settling which had been achieved.

Detention Time. The time theoretically required for a unit volume of water to flow through a mixing or sedimentation basin is called the *detention time*. It is equal to the time required to fill the basin at a given rate of flow and can be computed by dividing the volume of the basin by the rate of flow through it.

Detention time must be distinguished from the “retention” time, the minimum time required for a particle of water to pass through the basin. The detention period may vary from nearly 1 to 20 times the retention period, depending upon the shape and dimensions of the basin, the design of the inlet and outlet structures and baffles, and other factors that affect the degree of short-circuiting that occurs in the basin.

The desirable retention time is affected by the purpose of the basin. In a mechanically cleaned presedimentation basin, the detention time may be sufficient for the removal of only the coarse, rapid-settling sand and silt. In a plain sedimentation basin, depended upon for removal of fine suspended solids, the detention time must be long—in fact, the longer the better—has small particles settle very slowly.

In a properly designed settling basin which handles well-coagulated solids, a detention time of from 2 to 4 h is usually ample to prepare the water for filtration. When economy is not a governing factor or where a high degree of suspended-solids removal is desired, the settling basins may be somewhat larger than required for minimum detention.

Velocity. Theoretical velocities through sedimentation basins vary in rapid sand-filtration plants from a minimum of 0.5 ft/min (0.15 in/min) to a maximum of 5.7 ft/min (1.74 in/min), averaging 1.9 ft/min (0.58 in/min). The velocity in a sedimentation basin, however, is not uniform over the cross section, even though the inlet and outlet arrangements are designed for uniform distribution. The drag on the walls and floor results in zero velocity at these points, and the velocity exceeds the average at other points in the basin. The velocity is not stable because of varying density of the water, eddy currents, and volume of sludge. These factors are important in the determination of the dimensions of the basin.

Sludge Storage. Unless sludge is removed continuously, allowance must be made in the basin volume for sludge accumulation between cleanings. In manually cleaned basins, coagulated matter may be allowed to accumulate until it tends to impair the settled-water quality, at which time the sludge is flushed out. Where the water is fairly turbid, the greatest depth of sediment is near the inlet end of the basin. There is no specified depth of sediment beyond which it should not be allowed to accumulate, but in the design of the basin, an arbitrary depth is usually assumed as unavailable for settling.

The time between cleanings varies from a few weeks, in plants that have short periods of settling and handle very turbid water, to a year or more, where the basin capacity is large and the water is not very turbid. Sometimes decomposition of organic matter in the sediment makes it desirable to clean a basin more often than would otherwise be necessary. Decomposition of certain types of sludge can be controlled by prechlorination or other chemical treatment.

Sedimentation basins may be equipped with sludge collection and removal mechanisms to maintain volumetric efficiency and reduce the need to shut down for cleaning.

Surface Loading Rate. The surface overflow area is the most important parameter for basins clarifying flocculent solids. The surface overflow rate for any given sedimentation tank could be determined by jar test studies wherein the best coagulant, optimum dosage, and the best flocculation are used. But, usually, the design engineer must rely on past empirical experience and estimate a safe basin overflow rate based on representative water analyses and on the contemplated coagulant use. There remain the problems of changing seasons and changing water quality.

Table 5.25 gives a few common overflow rates. Medium-sized plants should use rates 15 to 20% less to

TABLE 5.25 Typical Surface Loading Rates

Application	Surface loading rate	
	gal/day/ft ²	m ³ /day/m ²
Plain sedimentation without chemicals	150	6.1
Lime softening		
Low magnesium	2000	81
High magnesium	1600	65
Alum coagulation		
Turbidity removal	1200	49
Color removal	900–1000	37–41

provide an additional factor of safety, and small-sized plants need even lower rates. These conservative rates assume good effluent diffusion and well-spaced effluent overflow weirs. They apply to both circular and rectangular basins.

Solids Loading Rate. When clarifying flocculated water carrying 3000 mg/L of solids or more, the solids loading rate is apt to be limiting when compared to the surface loading rate. Hence, the overflow rate needs to be limited to keep within bounds of solids loading on the floor of the clarifier. In water works practice, the basin influent rarely bears more than 1000 mg/L of flocculent solids. Thus, the solids loading rate is seldom limiting, but nevertheless it should not be ignored. Rectangular basins handle solids loading better than circular basins do. As a general average, long rectangular basins can handle up to 5 lb/ft² per day (244 kg/m² per day) of floor area because much of the density current (which is a function of solids loading rate) is emitted from the hopper blowdown or the sludge suction pipe. A limit of about 30 lb/ft² per day (150 kg/m² per day) applies to circular clarifiers.

Sludge Handling

No clarifier design discussion is complete without a plan for the collection and disposal of the unwanted underflow product—sludge. Traditionally, the sludge was disposed of simply by discharge of a basin into a nearby stream, but this is now forbidden.

Sludge Collection. As indicated previously, sedimentation basins that are not equipped for mechanical removal of sludge usually have sloping bottoms, so that when they are drained for cleaning, most of the sediment flows out with the water. Because the bulk of the material settles near the inlet end, the floor slope should be greater at this point. In some plants, sluice gates are arranged to deliver raw water to the sedimentation basin to flush out the sludge. The balance is generally washed out with a fire hose fitted with a small nozzle.

Circular tanks may be provided with rotating mechanisms driven by motors mounted at the center of the tank or, in some plants, by motor carriages running on the wall of the tank and pulling the arms around. The rotating apparatus—equipped with rakes or blades that travel slowly around an axis just above the floor of the basin—is designed to push the sediment to the center of the basin, where it is flushed out with a small stream of water. The bottom of the basin itself is given a fairly steep slope toward the center to facilitate the movement of the sludge. Square tanks may be equipped with a similar mechanism, except that two of the arms are often provided with outriggers that sweep into the corners of the tank outside of the radius of the circular area and convey the solids in those corners into the region of the main arms.

Two types of sludge-removal mechanisms are available for rectangular basins. One kind consists of a

series of chains and sprockets, with the latter mounted at the top and bottom of the tank. Wood or steel cross-members or flights are fitted between the chains. The flights scrape along the tank bottom, dragging the sludge to an outlet at one end. The second type of mechanism, mounted on a carriage that travels along the wall of the tank, does not have wearing parts under water and is thus less subject to shutdown. The sludge blades may be lifted and inspected at will without emptying the tanks.

Removal equipment must be designed to withstand the highly abrasive action of the sludge. In presedimentation basins where sand and grit are a problem and in softening plants where lime sludge is removed, careful design of blades is highly important, and wearing parts should be located above the waterline.

The force required to move mechanical sludge collectors can be calculated according to the equation (42):

$$F = fWn = fA \tag{5.23}$$

- where F = the force in the cable or chain, kg (lb)
- f = the load factor, lb/ft (kg/m)
- w = the width of the basin (length of header, blade, or flight), ft (m)
- n = number of header, blades, or flights in contact with sludge
- a = total area of all blades in contact with sludge, ft² (m²)

The load factors are given in Table 5.26 (42).

The designer should realize that, for any given category of sludge, there is bound to be a wide variation in the resistance to move a scraper or fluidizing blade through that sludge. The extent of the resistance also depends a great deal on the dwell time and actual density of the sludge encountered. Also, to be practical, the resistances enumerated must lie in the upper quartile of empirical experience, but the “average” resistance may not be more than one-half of these values.

Sludge Disposal. In large plants, the method of disposal is still a complex problem and at present somewhat of a dilemma. The cost of sludge disposal is sometimes apt to exceed, in both construction and operating cost, that associated with the treated water product.

In lime softening plants, slurry blowdown is further thickened for disposal as cake and the filter wash water is frequently returned to the flocculation and clarifier sections. In a very large softening plant, the number of units tends to spread out this filter wash water return flow, which is intercepted in a holding tank and pumped

TABLE 5.26 Load Factors for Sludge Collectors

Type of sludge	Load-factor (f) lb/ft of width		
	Suction header units	Bladed carriage units*	Plastic chain drag units [†]
Alum or iron	5	9	6
Lime (cold lime softening)	— [‡]	22	13
Rethickened lime	—	—	50

*For blades 12 in deep. For deeper blades multiply f by the ratio, blade depth/12 in. Rail friction, if any, to be added. Carriage units frequently carry transverse suction headers for which suction header values should be used.

[†]For flights 6 in deep. For deeper flights multiply f by the ratio, flight depth/6 in. Load factor includes sprocket and bearing friction, which are small for plastic chain, flights, and wearing shoes. For iron chain add 50% for friction (which tends to be a low estimate).

[‡]Lime sludge should not be collected by suction headers.

Note: 1 in = 2.54 cm; 1 lb/ft = 1.49 kg/m.

back at a uniform rate. In a smaller plant, this does not work well because of the long periods between filter washings and sporadic and relatively great flow of wash water.

In small plants, clarifier alum sludge, filter backwash sludge, and presedimentation sludge (if any) should, if at all possible, be ponded instead of dewatered by elaborate filter presses or centrifuges. When the ponds fill, they can be mucked out with land disposal of sludges or new ponds. The overflow from sludge ponds is typically collected and slowly pumped back to the plant where it joins the raw water supply. At lime softening plants, the pond overflow, which is softened water, should always be salvaged by means of a small pump installed in a pit receiving the pond overflow. The salvaged water should always be pumped back very slowly to avoid a deluge of recycle to the plant.

Filtration

Filtration is defined as the passage of fluid through a porous medium to remove matter held in suspension. Typical filter designs are shown in Figures 5.30 and 5.31.

In water purification, the matter to be removed includes suspended silt, clay, colloids, and microorganisms, including algae, bacteria, and viruses. The particles to be removed have approximate sizes as shown in Table 5.27, stated in millimicrons.

Filter Types. Table 5.28 contains a listing of filters according to various parameters. The slow sand filter was the first type of filtration used for water treatment. Table 5.29 contains typical values for sand filtration. Slow sand filters operate with a water depth of 3 to 5 ft (0.9 to 1.5 m). Removal of contaminants occurs in the upper inches of the sand. When the head loss reaches the physical limit of the plant, the filter is removed from service, drained, and cleaned. Filter runs usually last from 1 to 6 months, depending upon influent water quality, desired water quality, and filter loading rate. Cleaning of the filter is accomplished by removing the *Schmutzdecke*, the upper layer of dirty sand, and cleaning it before replacement, or by movable washing machines located at the surface of the filter. The *Schmutzdecke* formed on the sand surfaces has filtration properties of its own and desirable biological properties which may improve filter efficiency. Slow sand filters remove most suspended solids and reduce bacteria in excess of 98% (44). Prechlorination will lengthen filter runs, improve tastes, reduce odors, and free water of enteric bacteria.

Rapid sand filters are similar to slow sand filters with the exception of filter flow rates and cleaning practices. Typically, the rapid-sand filter consists of 18 to 30 in (45.7 to 76.3 cm) of sand with an effective size of from 0.35 to 1.00 mm (usually 0.5 mm) supported by gravel and underdrains. These filters are effective for raw or coagulated waters with turbidities as high as 10 NTU and are approximately 90% efficient in the removal of applied bacteria. If pretreatment with coagulation is accomplished, the overall plant using a rapid sand filter should deliver the same quality product water as an identical plant using a slow sand filter. Disadvantages of rapid sand filters as compared to slow sand filters include surface blockage, breakthrough due to coarse media, and loss of biological treatment due to high-rate backwashes.

In recent years, sand filters have been replaced, in many cases, by dual- or mixed-media filters. These filters consist of different density media of varying sizes in an attempt to approximate reverse graduation. Two of the most commonly applied schemes in potable water filtration are: (1) dual media, composed of a coarse anthracite coal (specific gravity is approximately 1.4 to 1.6) approximately 12 in (30.5 cm) deep; (2) mixed-media configuration, which utilizes coal (specific gravity is approximately 1.4 to 1.6), silica sand (specific gravity is approximately 2.6), and garnet sand (specific gravity is approximately 4.5). Both of these filters attempt to create a more ideal filtration mechanism by providing media in which the largest particles (2 mm) are on the top and the smallest particles (0.2 mm) on the bottom (i.e., reverse graduation). Granular media filters may be used with or without pretreatment by coagulation and sedimentation for removal of solids, by lime softening for precipitating hardness, or by precipitation of iron and manganese in well water supplies.

Septum or diatomaceous earth filtration is used mainly for swimming pools, military field units, and small

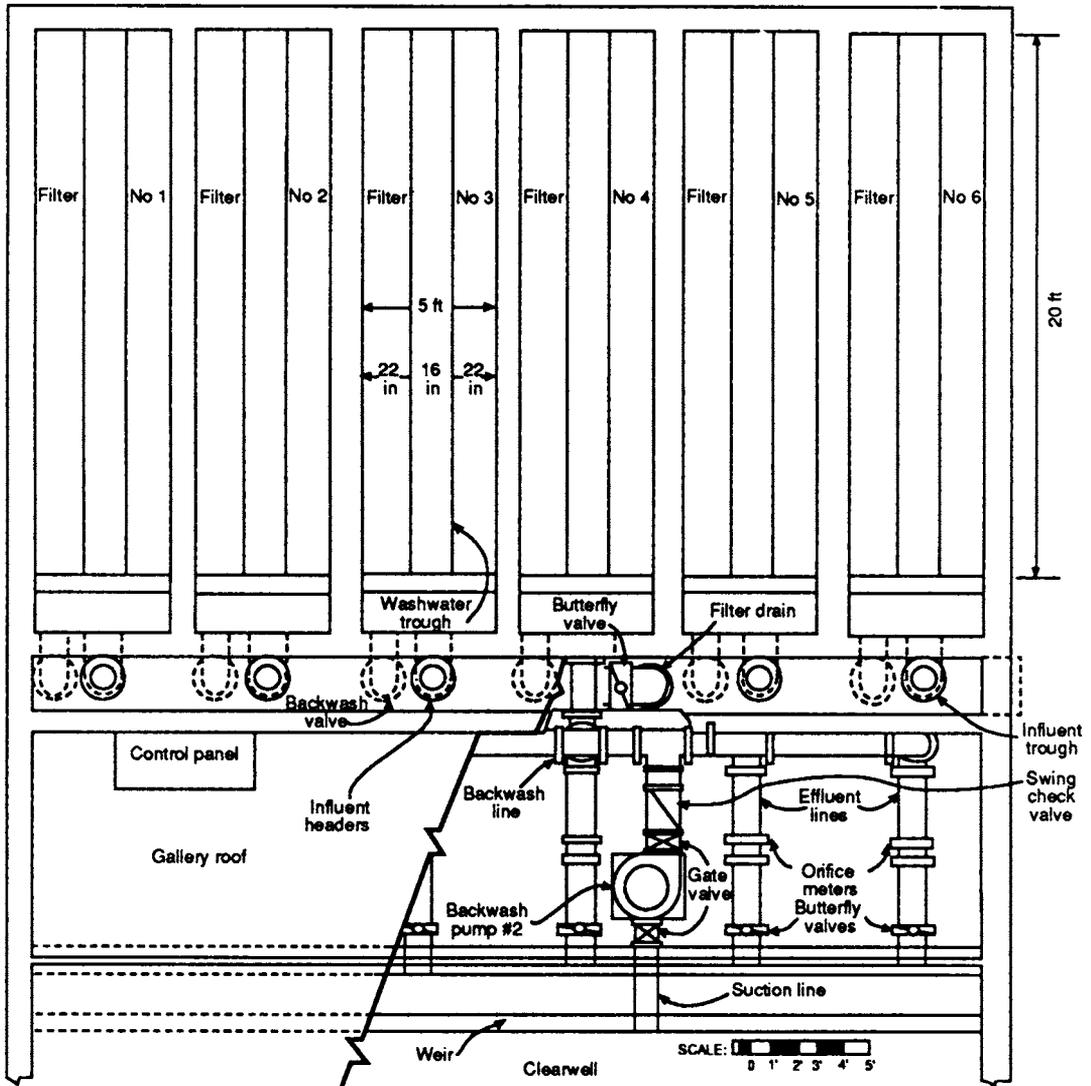


FIGURE 5.30 Plan of filter units (43).

community potable water treatment systems. The diatomaceous earth filtration cycle consists of precoating the septum with the diatom medium, filtering to a predetermined headloss, and removing the filter cake. During precoat, a slurry of diatomite (fossil skeletons of microscopic diatoms) is deposited to a thickness of approximately 0.1 in (0.25 cm) on a fine metal screen or septum to form the filter. Next the body feed, a mixture of raw water and diatomaceous earth, is applied to the filter and a filter cake develops. When the predetermined headloss across the filter cake is reached, the septum is washed and the cake discharged. Common rates of diatomaceous earth filtration range from 1 to 5 gpm/ft² (0.004 to 0.019 m³/min/m²). Diatomaceous earth filtration is a low-capital-cost system.

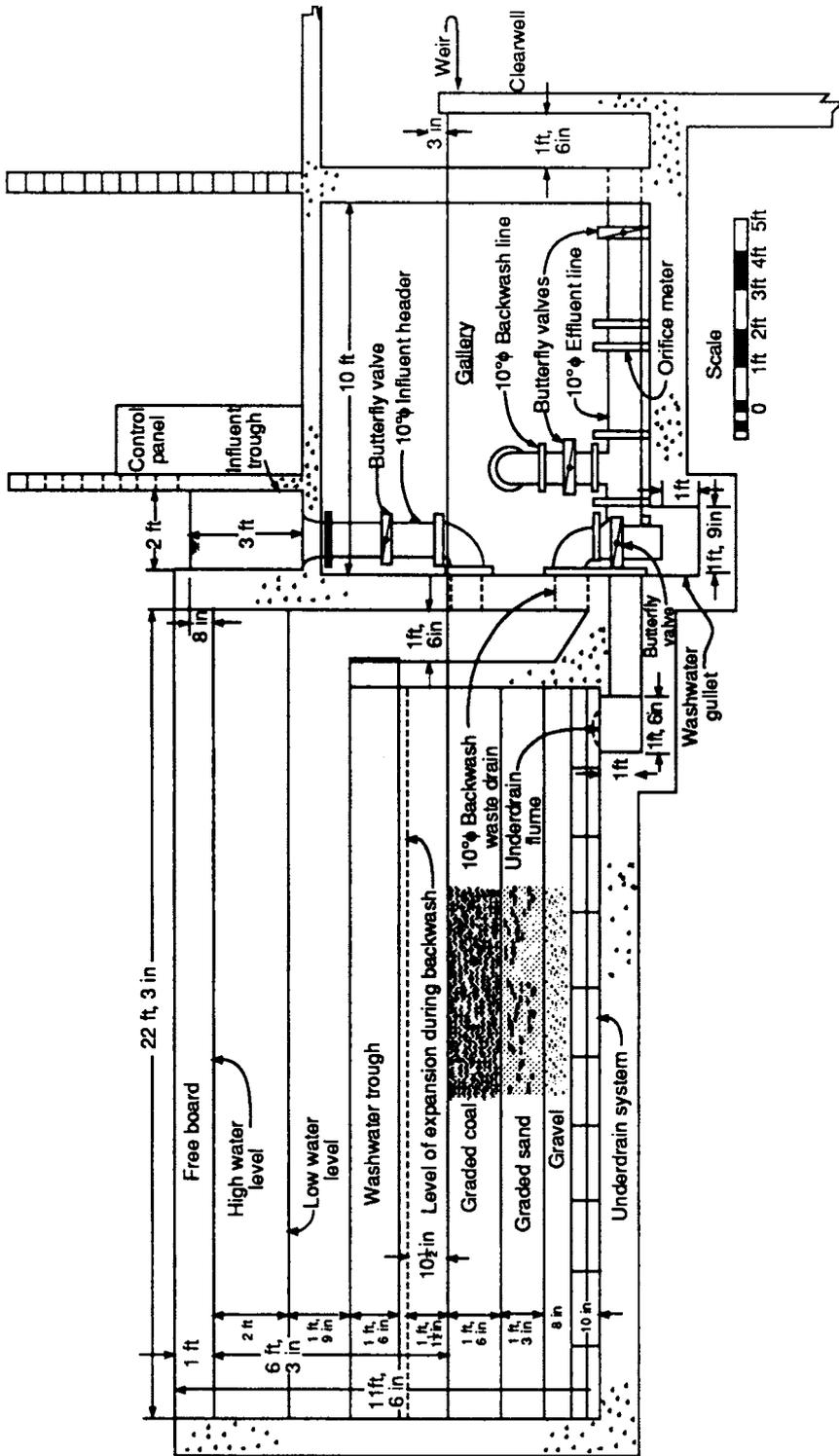


FIGURE 5.31 Section of filter units (43).

TABLE 5.27 Particle Sizes

Material	Particle size, 10^{-9} m
Silt	50,000
Bacteria	5,000
Viruses	50
Colloids	1–1,000

Filtration Process. Floc particle size and strength are of basic importance to the filtration process. In the water applied to the filters, floc particle size may range from 2 mm, down to sizes less than 0.1 mm. In a filter containing a rounded medium, the pore opening sizes will range from 15 to 40% of the particle diameter. Thus, the pore openings in a bed of 0.5-mm sand will range from 0.1 to 0.2 mm in size. In coarse, angular media, such as 1.2-mm anthracite, because of greater porosity, pore openings are larger, ranging from 0.3 to 0.6 mm.

It follows that the larger floc particles can be removed by simple straining at the bed surface, but that much of the flocculated matter will pass into the top 1 to 4 in (2.5 to 10.2 cm) of the bed and lodge within it. Unless the flocs are exceptionally strong, those that initially lodge in the bed by simple straining will subsequently break as the hydraulic gradient increases. Since breakage of the flocculated matter within the bed is inevitable, it is not necessary to handle settled water gently. Rough treatment, such as tumbling over weirs or pumping seems to have no adverse effect on filterability of the applied water. When a clean filter is placed in service, nearly all of the floc particles pass into the bed. As the flocculated matter lodges between the grains of the medium beneath the surface, the free void area is reduced and the bed offers resistance to the flow of the water. Conditions then begin to change. The rate of flow increases through the larger openings and lessens through the smaller and partly clogged openings. As a result, many of the larger passageways remain nearly free of deposited material (Figure 5.32). As filtration continues, the smaller openings become so filled with the coagulated material they are almost closed. The number of openings at the surface per unit of filter area is still very large, and there may not be an area as much as 0.5 in² (3.22 cm²) without a hole. Gradually, many of these holes stop feeding channels when the voids beneath the surface become so clogged that flow becomes impossible.

If the medium's grains are small in size, or if the floc is tough, the penetration of flocculated material beneath the sand surface may be small. This is illustrated in panel (a) of Figure 5.32. Panel (b) represents penetration of part of the coagulated material into the bed with a fairly large portion of it remaining at the surface. Panel (c) depicts continuing penetration into the bed through channels. Panels (b) and (c) represent

TABLE 5.28 Classification of Filters

Parameter	Type
Direction of flow	Upflow; downflow, biflow; radial flow
Type of media	Fine to coarse; garnet sand; multilayered; diatomaceous earth
Flow rate	
Slow	0.5–0.15 gpm/ft ² (0.02–0.01 m ³ /min/m ²)
Rapid	1–2.5 gpm/ft ² (0.04–0.10 m ³ /min/m ²)
High	3–15 gpm/ft ² (0.12–0.61 m ³ /min/m ²)
Influence of flow	
Pressure	150 lb/in ² (10.54 kgf/cm ²)
Gravity	1.3–5.2 lb/in ² or 3–12 ft head (0.09–0.37 kgf/cm ²)

TABLE 5.29 Typical Slow Sand Filtration Values(33)

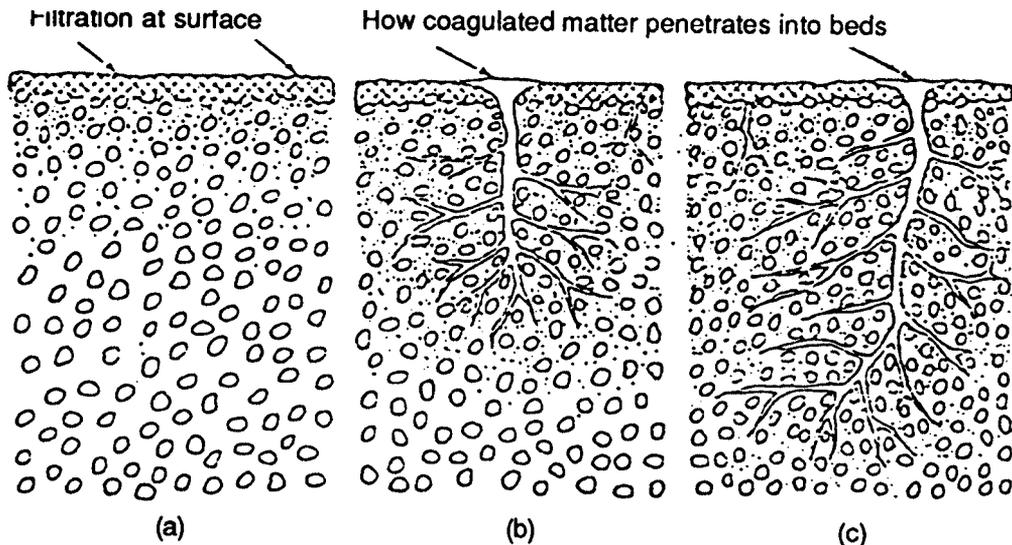
Parameter	Value
Depth of sand	3 to 5 ft (0.91–1.52 m)
Medium	Silica sand
Effective sand size	0.35 mm
Uniformity coefficient	1.75
Gravel	6–12 in (15.24–30.48 cm)
Open-joint drains	10–20 ft (3.05–6.10 m) apart on centers

conditions that are expected to take place in a rapid-sand filter bed. These filters are diagrammatic; the channels are actually quite crooked.

To obtain filter runs of reasonable length with sand beds, the penetration of coagulated material into a single-media bed should be from about 2 to 4 in (5 to 10 cm). When there is little penetration of the material into the filter bed, as shown in Panel (a), the filter loss of head increases rapidly, and the filter runs are too short for practical operation of the filters. When penetration is greater, flocs pass down into coarser underlying layers, and there is danger of breakthroughs.

There is little or no deposition of flocculated material in the channels where velocities are high. The sediment-bearing water moves down the channels until it reaches voids that have not been filled with the coagulated material. It then spreads out, the velocity reduces, and lodgement occurs again.

When the voids in the media around the end of a channel have filled sufficiently to increase the resistance to flow, previously deposited floc shears, and new channels open up through previously unperforated areas and conduct the waters to unclogged sections. These new channels are caused by the increasing hydraulic gradient, which causes increased shear stress. The process continues to repeat itself. As the loss of head through the filter increases and the flocculated material clogs to a greater depth, the channels through the

**FIGURE 5.32** Filter bed operation (45).

surface become fewer. Some water also enters the bed by seepage through the surface layer. The part that does this is small in comparison to that entering through channels.

Filter Design. Table 5.30 summarizes significant facts in filtration that are reasonably well documented and have a major impact on filter design. These concepts are of major importance in design of direct filtration plants, which are commonly the most economic treatment scheme for low-turbidity raw waters (<50 TU), such as those from lakes needing low-alum coagulant dosages.

Figure 5.33 gives an illustration of the behavior of the filter during a filter run. It shows the initial breakthrough, a transitory jump in effluent turbidity, and a terminal breakthrough. Since initial breakthrough usually lasts no more than 30 min, and therefore has no great effect on the general plant average quality, in most plants no corrective action is taken. In some plants, water is filtered to waste for 15 to 30 min at the start of each filter run. British waterworks practice usually includes a 30-min slow start for each filter run as a measure to prevent the initial breakthrough.

The transitory turbidity jump at 20 h (Figure 5.33) is the kind of event caused by a coagulant underfeed or failure. It shows up quite conspicuously on a recording effluent-turbidity monitor. The turbidity rise at the end of the run, occurring as floc penetration approaches the full bed depth, is typical of the terminal breakthrough caused by a condition of weak flocculation. To detect such conditions, it is helpful to have a turbidity monitor on an individual filter as well as another on the total plant production.

TABLE 5.30 Variables Affecting Filter Operation and Design

Item	Effect
1. Pretreatment, filtration rate, and water quality	With adequate pretreatment, filtration rates from 2–6 gpm/ft ² (0.08–0.24 m ³ /min/m ²) produce the same water quality (48, 49). Chemical destabilization is an essential prerequisite for effective filtration (50, 51, 52). Inadequate pretreatment may produce poor-quality effluent even at filtration rates less than 2 gpm/ft ² (0.08 m ³ /min/m ²).
2. Filtration rate changes, water quality and rate control for design	Any rate change during filtration causes significant deterioration of the effluent quality (53, 54). Degradation in quality can be qualitatively correlated with magnitude of rate change and inversely with the time for the rate change (50, 51).
3. Filter medium, run length, and backwashing	The finer the medium, the better the quality, but headloss increases which reduces run length. Coarse medium implies a balance between effluent quality degradation and headloss buildup, but requires higher backwash rates. Simultaneous air scour and subfluidization wash water is the most effective cleaning system, but it destratifies single medium and mixed dual media and multimedia. Restratification of dual media or multimedia requires that the filter medium be fluidized.
4. Filtration and polyelectrolytes	Polyelectrolytes can assist in improving effluent quality when added in small amounts as a filter aid (0.008–0.001 mg/L) just prior to filtration. Addition of polyelectrolytes makes backwashing more difficult due to the higher attachment forces and auxiliaries like surface wash or air scour are necessary. Optimum dosages, overdosing, and stoichiometry, which are characteristic of destabilization in coagulation, also occur with polymer-aided filtration.

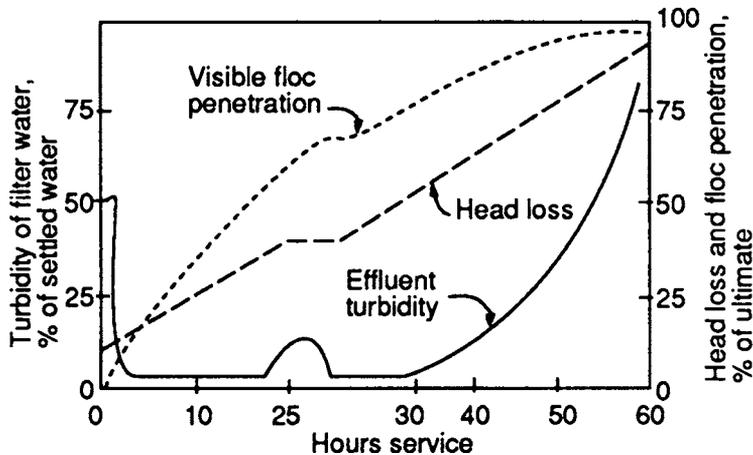


FIGURE 5.33 Behavior of filter during filter run (45).

When there are breakthroughs due to weak flocculation, and when they cannot be corrected by chemical means, good operating practice requires termination of filter runs at head losses lower than 8 ft (2.4 m) and reduction to provide the additional resistance needed to maintain the desired flow rate or water level. As filtration continues, the filter becomes clogged with solids and the flow control valve gradually opens. When the valve is fully open, the run must be terminated, since any further increase in filter resistance will not be balanced by a corresponding increase in the resistance of the flow control valve.

The disadvantages of effluent-control, constant-rate filtration include the following:

- The initial and maintenance costs of the fairly complex rate control system are high.
- The filtered water quality is not as good through gravity granular media filters as through declining-rate filters in potable water filtration (44, 46).
- The rate or level control systems frequently do not function properly, causing sudden changes in rate, with the effect described previously on water quality. In many existing works, the control systems are nonfunctional.

Filter Backwashing. Both sand and mixed-media filters are normally cleaned by reversal of the flow through the bed, or backwashing. Backwashing expands the medium's depth, allowing particulates to be released into the water which are collected above the media in wash troughs. This washing process may be improved by jets of water, air injection, or mechanical agitation at the surface. Cleasby et al. cite a method of backwashing that may be used in various situations: (1) High-rate backwash with full bed expansion and fluidization (bed expansion 20 to 50%) preceded by a lower-rate backwash (47). This method may be used for single-medium or multimedia filters. (2) Low-rate backwash without fluidization for expansion but with auxiliary scour. This method is used for single-medium filtration only. (3) Water surface wash. (4) Air and water surface wash has several options, including: (a) air with low-rate water backwash for a single medium only; (b) air with high-rate water backwash for a single media and multimedia; (c) simultaneous air and lowrate water backwash for a single medium or multimedia. Backwash efficiency is affected by medium size, shape, density, water quality, and coagulants used previous to or during the filtration process. Backwash water is either discharged to the sewer or settled with the supernatant returned to the front of the plant and the sludge treated in some acceptable manner.

The best criteria for backwash effectiveness are the quality of the filtered water and long-term absences

of dirty filters and mudball formation. Amirtharaja has indicated that few or no contacts occur between fluidized particles (47). Hence, particulate fluidization with water alone is an intrinsically weak cleaning process. Air scour, which causes abrasions between particles throughout the depth of the bed, and surface wash, which causes collisions at the top of the bed, are effective auxiliaries for cleaning (47).

For ordinary solids (low adhesive forces), water wash alone—which expands the bed by 30 to 40% to give expanded porosities around 0.70 at the top layers of the backwashed filter—provides optimum cleaning. This is economically achieved with a total wash water usage of 75 to 100 gal/ft² (3.1 to 4.1 m³/m²) or 1.5 gpm/ft² for 6 to 7 min (0.06 m³/min/m² for 6 to 7 min). An important consideration in estimating backwash requirements is to calculate the expansion of the medium with the warmest temperature of the wash water. Simultaneous air scour 3 to 5 scfm/ft² (0.9 to 1.5 m³/min/m²) and subfluidization water wash 6 to 8 gpm/ft² (0.24 to 0.32 m³/min/m²) provide the best cleaning characteristics for solids with higher adhesive forces (polyelectrolytes or wastewater solids). Dual-media filters have the ever present danger of loss of media with simultaneous air and water wash and also need a fluidization wash at the end of the backwash cycle to re-stratify the media. If supporting gravel layers are used, their movement during air scour is possible. Loss of media can be avoided by: (1) lowering the water level to the media surface prior to wash, (2) using air scour alone, and (3) providing greater depth between the surface of the media and the lip of the wash water troughs.

Filter backwash operations are important in the control of microbiological contaminants within the treatment train. The collected backwash water may be reintroduced to plant influent with or without prior treatment, discharged to a sanitary sewer, discharged to a surface water, or placed in a settling lagoon. The typical water treatment plant generates a backwash volume that is 3 to 7% of the total daily flow, and recycling has been common practice. However, recycling such a large volume of water, especially during a short time period, can significantly increase the concentration of contaminants in the plant influent. During source water events, such as periods of heavy rain and runoff, or operational upsets or difficulties, recycling filter backwash may compromise the treatment plant's ability to produce high-quality finished water.

The Safe Drinking Water Act Amendments of 1996 requires the EPA to promulgate a regulation governing the recycle of filter backwash within the treatment process of public water systems by August 2000. The regulation will apply to all systems, regardless of size, that backwash filters. Available options are banning the recycle of filter backwash, requiring that recycle flows be equalized, requiring that all recycle flows be treated to a specific level, requiring that all recycle flows be treated with a specific technology, and other options not yet identified.

Chemical Feed and Handling

Problems associated with chemical feed and handling are important throughout the water treatment process. The topic of chemical feeding and handling will be discussed in relationship to the individual unit processes requiring it.

One important concept in analyzing chemical handling requirements is whether the chemical under consideration is one whose application may be interruptible. Those materials used for corrosion control, taste and odor control, and fluoridation can have their application interrupted, and therefore less rigorous criteria apply to their storage space and handling equipment. On the other hand, substances such as coagulants and chlorine must not be interrupted under any circumstances. Adequate storage space, handling facilities, and adequate standby provisions for feeding are required for them.

Capacity and Reserve Provisions. Depending on the number of application points required for each chemical, the number of feeding units is established. For example, in a large water treatment plant, where it is necessary to have three or four application points for coagulant chemicals, it should be anticipated that a coagulant feeder would be provided for each application point, plus approximately 50% surplus

units or not less than two standby units, whichever is least. Similarly with chlorinators, an adequate reserve should be provided so that service can be maintained regardless of equipment servicing or maintenance requirements. A good general rule is that there should still be a unit in reserve, even when one of the largest units is out of service.

For interruptible materials, less rigorous standards may be applied. Some reserve equipment is desirable, but it need not meet all the criteria stated above.

Capacity of feeding equipment is based on two simultaneous requirements: (1) capability to meet the maximum dosage required, and (2) capability to feed that dosage under the maximum demand to be expected on the plant while still maintaining reserve units. When considered in connection with minimum feed rates for chemical feed equipment (usually about 10% of the maximum feed rate), these requirements may call for a greater range of chemical feed than a single unit can provide. In these circumstances, a larger number of units may be needed in order to cover the range requirement. Frequently, it is preferable to plan chemical feeding equipment for a shorter time of growth than the life of the entire plant. Additional units may then be added a few years after plant construction, as plant demands grow.

Storage Provisions. Storage provisions are outlined in Table 5.31, which states them in terms of days of supply for both interruptible and noninterruptible materials. Table 5.32 indicates the storage characteristics that apply in design (48).

The arrangement of storage and chemical handling should make these operations as convenient and easy as possible. Use of hand or forklift trucks should be maximized, with design eliminating lifting of heavy loads by operators. For example, personnel should not have to carry bagged materials upstairs.

Chemical Handling. In the infancy of water purification, most chemical handling was in liquid form. Dry feeding equipment was then developed and used intensively for many decades. In the last decade or two, there has been a return of the use of liquid chemicals rather than dry for several reasons. First the equipment for handling liquids is commonly simpler and less costly. Second, handling chemicals in liquid form eliminates numerous serious dust problems. Systems for handling dry chemicals are well known and adequately described in manufacturers literature.

Systems for handling wet chemicals are not as widely known or described. They include variable-head orifice-type feeders, usually regulated by means of float valves; constant-head orifice feeders, in which the orifices may be adjustable; and various types of rotating dipper units. Additionally, chemicals that do not cause coatings or encrustation can be metered through tapered flow tubes or rotameters. In a number of

TABLE 5.31 Chemical Storage Space Criteria (48)

Class of chemicals	Noninterruptible	Interruptible
Examples of class	All chemicals used for disinfection; chemicals used for coagulation in treatment plants where raw water is polluted; softening chemicals	Chemicals used for corrosion control; taste and odor; fluoridation
Minimum stock to be maintained, in days	30	10
Additional allowance based on shipping time, in days*	2 times shipping time	1.5 times shipping time

*Additional allowance must be large enough to accommodate maximum size shipment (truckload, carload, fractional shipload).

TABLE 5.32 Chemical Storage Type Criteria

Type of storage	Dry	Wet
Handling requirements	<ol style="list-style-type: none"> 1. Allow for access corridors between stacks of packaged chemicals. 2. Palletize and use fork lift truck only in large installations. 	<ol style="list-style-type: none"> 1. Provide agitation for slurries such as carbon or lime; not less 1 hp (0.75 kW) mixing per 100 ft³ (2.83 m³). 2. Check manufacturers of feed equipment for pumps and pipe sizing.
Safety and corrosion requirements	<ol style="list-style-type: none"> 1. Provide separated storage spaces for combustibles and for toxic chemicals, such as carbon or chlorine gas. 2. Provide ample space between stores of materials that may interact, such as ferrous sulfate and lime. 	<ol style="list-style-type: none"> 1. Double-check corrosion resistance of bulk storage linings. 2. Isolate hazardous or toxic solutions such as fluosilicic acid. 3. Prefer below-ground or outdoor storage.

Source: From Ref. 48.

recent installations, the chemical has been transferred from bulk storage solution to a pressurized day tank from which it is fed. A pneumatic drive may then be applied to the day tank to force the liquid through a rotameter or through some other type of flow device, such as a magnetic flow meter. Systems of this type lend themselves to flow automation, enabling the chemical feed to be proportioned to the dosage required and to the flow through the plant.

With either liquid or dry feed chemical equipment, it has been fairly common to pace the rate of feed of the chemical by starting and stopping the feeding equipment. This can be a disadvantage when used for coagulant or coagulant-aid feeds, since the feed may be interrupted frequently and continuously, thus overdosing one stream of water and underdosing a succeeding one. This can have serious adverse effects on chemical treatment reactions.

Most of the foregoing is less rigorously applied to most interruptible materials, with the exception of fluorides. With fluorides, tight control of dosages and measurement of residuals is important to establish that there is no overdose.

Clearly, feeding equipment should be designed to make all of the above operations convenient and easy so that they can be carried out quickly and without excessive burden on operating personnel. Tables 5.33 and 5.34. list the chemicals used in water treatment (49).

Disinfection

The destruction of pathogenic bacteria in water has always been of primary concern in the treatment of potable water. Several methods are available for disinfection, including chlorine, chlorine dioxide, ozone, and potassium permanganate.

Chlorination is the traditional disinfection technique practiced in the United States. Chlorine is shipped, in liquid form, in pressurized steel cylinders ranging in size from 100 lb to 1 ton (45 kg to 0.9 t). One volume of chlorine liquid yields 450 volumes of chlorine vapor. The moist gas is corrosive; therefore, all piping and dosing equipment must be nonmetal or resistant to corrosion.

Chlorine gas is drawn from the pressurized cylinder through a solution feeder, which controls the rate of application. The injector, in a solution feed chlorinator, dissolves the gas into the feedwater (Figure 5.34). The concentrated solution is then applied to the process water. Both surface and groundwater water supplies require a chlorine residual to ensure good bacteriological quality in the distribution system. Breakpoint chlorination, prior to coagulation of a surface water, may be practiced at certain times of the year for taste and

TABLE 5.33 Chemicals Used in Water Treatment

Coagulants and coagulant aids		
Coagulants	Coagulant aids	
Aluminum sulfate (bauxite)	Bentonite	
Ferrous sulfate	Calcium carbonate	
Ferric sulfate	Carbon dioxide (see "Softening")	
Ferric chloride	Sodium silicate	
Sodium aluminate	Organic coagulant aids as	
Hydrated lime and quick-lime (see "Softening")	approved by Technical Advisory Committee on Coagulant Aids for Water Treatment	
Aluminum ammonium sulfate		
Aluminum potassium sulfate		
Disinfection and chlorinating agents		
Disinfection agents	Dechlorinating Agents	pH adjustment
Ammonia, anhydrous	Activated carbon	Calcium carbonate (see "Coagulant Aids")
Ammonium hydroxide	Aluminum ammonium sulfate	Carbon dioxide (see "Softening")
Ammonium sulfate	Ion-exchange resins	Hydrochloric acid
Bromine	Sodium bisulfite (sodium pyrosulfite)	Lime (quicklime and hydrated lime (see "Softening")
Chlorine	Sodium sulfite	Sodium carbonate (see "Softening")
Chlorine dioxide (sodium chlorite)	Sulfur dioxide	Sodium hydroxide
Chlorinated lime		Sulfuric acid
Hypochlorites (calcium, lithium, and sodium)		
Ozone		
Silver		
Fluoridation and fluoride adjustment		
Fluoridation	Fluoride adjustment agents	
Ammonium silico-fluoride	Activated alumina	
Fluospar (CaF)	Fluo-Carb	
Hydrofluoric acid	Fluorex	
Hydrofluosilicic acid	Ion-exchange resins	
Sodium fluoride	Magnesium oxide	
Sodium silico-fluoride	Sodium bisulfate	
Tastes and odors		
Activated carbon, powdered and granulated		
Bentonite		
Chlorine		
Chlorine dioxide		
Ozone		
Potassium permanganate		
Mineral oxidation (iron and manganese)		
Chlorine		
Chlorine dioxide		
Ozone		
Potassium permanganate		

TABLE 5.33 Chemicals Used in Water Treatment (*continued*)

Stabilization and corrosion control
Quicklime and hydrated lime (see "Softening")
Sodium carbonate (see "Softening")
Sodium tri-polyphosphate
Sodium hexa-metaphosphate
Sodium hydroxide
Softening
Carbon dioxide
Ion-exchange resins
Quicklime and hydrated lime
Sodium carbonate (soda ash)
Sodium chloride
Sulfuric acid (see pH-adjustment section)
Miscellaneous
Copper sulfate
Sodium bichromate

odor control in addition to disinfection. Intermediate or postchlorination may also be required to maintain disinfection throughout the plant and distribution system.

Chlorine dioxide may be produced from sodium chlorite, and acid from sodium chlorite and gaseous chlorine or from sodium hypochlorite. After production, chlorine dioxide is fed through PVC pipe using a diaphragm pump. Safety features, such as chlorine gas detectors, floor drains, and emergency gas masks, should be available at the generation and application site. The major advantage of chlorine dioxide is for use as a residual disinfectant. It does not produce measurable quantities of by-products, such as trihalomethanes, because it does not react with many chlorine-demanding substances (possible organic precursors). Other advantages of chlorine dioxide include algae destruction; improvement in color, taste, and odors; iron and manganese removal, and residual and general disinfection properties.

Ozone is widely used in European water systems for disinfection. The ozonation system consists of four parts: (1) a gas preparation system, (2) an electrical power supply, (3) ozone-generating equipment, and (4) contacting equipment. The gas preparation system includes a filtering and drying process that removes oil, dust, and water from the air or oxygen that would interfere with and possibly damage the ozone preparation system. The air or oxygen is also cooled by refrigeration or heat exchangers before it is used. Drying is accomplished by passing the gas to be converted through a desiccant (silica gel, alumina gel, or calcium chloride).

The two types of ozone production systems in water treatment use are the Otto and Welsback systems. Both systems use charged elements arranged in groups or suspended in a glass or stainless steel chamber. The charge of 8000 to 20,000 V is provided by a step-up transformer. The Otto generator is composed of elements with high- and low-tension plates separated by a glass dielectric. The dielectric is connected to the low-tension plates and spaces provide an air gap between the high-tension plate and the dielectric. In the Welsback generator elements are tubular. Each element has a central high-tension cylindrical conductor supported coaxially within a glass cylinder (dielectric) jacketed in stainless steel which acts as the low-tension conduction. Both systems are cooled by convection on the high-tension member and by water in

TABLE 5.34 Characteristics of Chemicals Used in Water Treatment

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft ³	Solubility, lb/gal	Commercial strength	Characteristics
Coagulation								
Aluminum sulfate, $\text{Al}_2(\text{SO}_4)_3 \cdot 14\text{H}_2\text{O}$	Alum, filter alum, sulfate alumina	100–200 lb bags; 300 and 400 lb bbls; bulk (carloads)	Dry, iron, or steel; solution, lead-lined, rubber, silicon, asphalt, 316 stainless steel	Ivory-colored Powder Granule Lump	38–45 60–63 62–67	4.2 (60°F)	15–20% Al_2O_3	pH of 1% solution, 3.4
Ammonium aluminum sulfate $\text{Al}_2(\text{SO}_4)_3 (\text{NH}_4)\text{SO}_4 \cdot 24\text{H}_2\text{O}$	Ammonia alum, crystal alum	bags, bbls; bulk	Duriron, lead, rubber, silicon, iron, stoneware	Lump Nut Pea Powdered	64–68 62 65 60	0.3 (32°F) 8.3 (212°F)	11% Al_2O_3	pH of 1% solution, 3.5
Bentonite	Colloidal clay, volclay, wilkinite	100-lb bags; bulk	Iron, steel	Powder, pellet, mixed sizes	60	Insoluble (colloidal solutions used)		
Ferric chloride* a. FeCl_3 (35–45% solution) b. $\text{FeCl}_3 \cdot 6\text{H}_2\text{O}$ c. FeCl_3	Ferrichlor, chlorine of iron Crystal ferric chloride Anhydrous ferric chloride	5–13 gal carboys; trucks; tank cars 300-lb bbls. 500-lb casks; 100, 300, 400 lb kegs	Glass, rubber, stoneware, synthetic resins	Dark brown syrupy liquid Yellow-brown lump Green-black powder		Complete	37–47% FeCl_3 20–21% Fe 59–61% FeCl_3 20–21% Fe 98% FeCl_3 34% Fe	Hygroscopic (store lumps and powder in tight containers); no dry feed; optimum pH, 4.0–11.0
Ferric sulfate* $\text{Fe}_2(\text{SO}_4)_3 \cdot 9\text{H}_2\text{O}$	Ferrifloc Ferrisul	100 to 175-lb bags 400–, 425-lb drum	Ceramics, lead, plastic, rubber, 18–8 stainless steel	Red-brown or granule		Soluble in 2–4 parts cold water	90–94% $\text{Fe}_2(\text{SO}_4)_3$ 25–26% Fe	
Ferrous sulfate* $\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$	Copperas, green vitriol	Bags; bbls; bulk	Asphalt, concrete lead, tin, wood	Green crystal granule, lump	63–66	0.5 (32°F) 1.0 (68°F) 1.4 (86°F)	55% FeSO_4 20% Fe	Hygroscopic; cakes in storage; optimum pH 8.5–11.0

(continues)

TABLE 5.34 Characteristics of Chemicals Used in Water Treatment (*continued*)

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft ³	Solubility, lb/gal	Commercial strength	Characteristics
Coagulation								
Potassium aluminum sulfate, $K_2K_2O_4 \cdot Al_2(SO_4)_3 \cdot 24H_2O$	Potash alum	Bags, lead-lined; bulk (car-loads)	Lead, lead-lined rubber, stoneware	Lump Granule Powder	62–67 60–65 60	3.0 (68°F) 3.3 (86°F)	10–11% Al_2O_3	Low, even solubility; pH of 1% solution, 3.5
Sodium aluminate, $Na_2OAl_2O_3$	Soda alum	100 and 150 lb bags; 250 and 440 lb drums; solution	Iron, plas-tics, rub-ber, steel	Brown powder, liquid (27°Be)	50–60	Complete	70–80% $Na_2Al_2O_4$	Hopper agitation required for dry feed
Sodium silicate, Na_2OSiO_2	Water glass	Drums, bulk (tank trucks, tank cars)	Cast iron, rubber, steel	Opaque, viscous liquid			38–42°Be	Variable ratio of Na_2O to SiO_2 ; pH of 1% solution, 12.3
Disinfection and dechlorinating agents								
Ammonium aluminum sulfate, $Al_2(SO_4)_3(NH_4)_2SO_4 \cdot 24H_2O$	Ammonia alum, crystal alum	Bags, bbls; bulk	Duriron, lead rub-ber, sili-con, iron, stoneware	Lump Nut Pea Powder	64–68 62 65 60	0.3 (32°F) 8.3 (212°F)	11% Al_2O_2	pH of 1% solution, 3.5
Ammonium sulfate, $(NH_4)SO_4^*$	Sulfate of ammonia	100–lb bags;	Ceramics, plastics, rubber, iron (dry)	White or brown crystal		42.5	6.3 (68°F)	Cakes in dry feed; add $CaSO_4$ for free flow
Anhy-drous ammonia, NH_3	Ammonia	50, 100, and 150 lb cylinders; in bulk (tank cars and trucks)	Glass, iron, Monel metal, nickel, steel	Colorless gas		3.9 (32°F) 3.1 (60°F) 1.8 (125°F)	99–100% NH_3	
Aqua am-onia, NH_4OH	Ammonia water, ammo-nium hy-drate, am-monium	Carboys: 750–lb drums; 8000–gal tank cars or trucks	Glass, iron, Monel metal, nickel, steel	Colorless liquid		Completely	29.4° NH_2 (26°Be)	

TABLE 5.34 Characteristics of Chemicals Used in Water Treatment (*continued*)

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft ³	Solubility, lb/gal	Commercial strength	Characteristics
Disinfection and dechlorinating agents								
Calcium hypochlorite, CaOCl ₂ ·4H ₂ O	HTH, Perchloron, Pittchlor	5-lb cans; 100, 300, and 800 lb drums	Glass, rubber, stoneware, wood	White granule, powder	52.5		70% available Cl ₂	
Chlorinated lime, (CaO) 2CaOCl ₂ ·3H ₂ O	Bleaching powder chloride of lime	100, 300, and 800 lb drums	Glass, rubber, stoneware, wood	White powder	48		25–30% available Cl ₂	Deteriorates
Chlorine, Cl ₂	Chlorine gas, liquid chlorine	100 and 150 lb cylinders; 1-ton containers; 16, 30, 55, 85, and 90 ton tank cars; tank trucks about 15–16 tons)	Dry: black iron, copper, steel; wet gas: glass, hard rubber, silver	Liquified gas under pressure	91.7		0.07 (60°F) 0.04 (100°F)	99.8% Cl ₂
Chlorine dioxide, ClO ₂	Chloride dioxide	Generated as used	Plastics, soft rubber (avoid hard rubber)	Yellow-red gas		0.02 (30 mm)	26.3% available Cl ₂	
Ozone, O ₃	Ozone	Generated at site of application	Aluminum, ceramics, iron, steel, wood	Colorless gas			Dry, 67% SO ₂ 33.3% SO ₂	Sulfurous odor
Pyrosodium sulfite	Sodium metabisulfite	Bags; drums; bbls	Iron, steel, wood	White crystalline powder		Completely in water		
Sodium chlorite, NaClO ₂	Technical sodium chlorite	100-lb drums	Metals (avoid cellulose materials)	Light orange powder or flake			82% NaClO ₂ 30% available Cl ₂	Generates ClO ₂ at pH 3.0
Sodium hypochlorite, NaOCl	Sodium hypochlorite	5 and 13 gal carboy; 1300–2000 gal tank trucks	Ceramics, glass, plastic, rubber	Light yellow			12–15% available Cl ₂	

(continues)

TABLE 5.34 Characteristics of Chemicals Used in Water Treatment (*continued*)

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft ³	Solubility, lb/gal	Commercial strength	Characteristics
Disinfection and dechlorinating agents								
Sodium sulfite, Na ₂ SO ₃	Sulfite	Bags; drums; bbls	Iron, steel, wood	White crystalline powder		Complete in water	23% SO ₂	Sulfurous taste and odor
Sulfur dioxide, SO ₂	Sulfurous acid anhydride	Steel cylinders; ton containers; tank cars or trucks	Aluminum, brass, Durco D-10, stainless steel 316	Colorless gas		20% at 32°F, complete in water	99% SO ₂	Irritating gas
Fluoridation and fluoride adjustment								
Ammonium silico fluoride, (NH ₄) ₂ SiF ₆	Ammonium fluorsilicate	100 and 400 lb drums	Steel, iron, and lead	White crystals		1.7(63°F)	100%	White, free-flowing solid
Calcium fluoride, CaF ₂	Fluorspar	Bags; drums; bbls; hopper cars; trucks	Steel, iron, and lead	Powder		Very slight	85% CaF ₂ less than 5% SiO ₂	
Hydrofluosilicic acid, H ₂ SF ₆	Fluosilicic acid	Rubber-lined drums; trucks; or RR tank cars	Rubber-lined	Liquid		Approx. 1.2 (68°F)	Approx. 35%	
Hydrogen fluoride,* HF	Hydrofluoric acid	Steel drums; tank cars	Steel	Liquid			70%	Below 60%, steel cannot be used
Sodium fluoride,* NaF	fluoride	Bags; bbls; fiber; drums; kegs	Iron, lead, steel	Nile blue or white powder Light Dense	50 75	0.35 (most temps.)	90–95% NaF	pH of 4% solution, 6.6
Sodium silico fluoride Na ₂ SiF ₆	Sodium silico fluoride	Bags; bbls; fiber; drums	Iron, lead, steel	Nile blue or yellowish-white powder	72	0.03 (32°F) 0.06 (72°F)	99% Na ₂ SiF ₆	pH of 1% solution, 5.3

TABLE 5.34 Characteristics of Chemicals Used in Water Treatment (*continued*)

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft ³	Solubility, lb/gal	Commercial strength	Characteristics
Fluoridation and fluoride adjustment								
Aluminum oxide, Al ₂ O ₃	Activated alumina	Bags; drums	Iron, steel, lead	Powder, granules (up to 1 in diam)	Variable	Insoluble	100%	
Bone charcoal	Fluo-Carb	Bags; drums; bulk	Wood, iron, steel	Granules	Variable	Black, best used in beds for presolution		
Tricalcium phosphate	Fluorex	Bags; drums; bulk; bbls.	Iron, steel	Granular, technical	50–63	Insoluble		Also available as white powder
High-magnesium lime	Dolomitic lime	Bags; bbls; bulk	Wood, iron, steel	Lump, pebble, ground		Slakes slowly	58% CaO, 40% MgO ₂	
Stabilization and corrosion control								
Disodium phosphate, Na ₂ HPO ₄ · 12H ₂ O	Base sodium phosphate, DSP, secondary sodium phosphate	120-lb kegs; 200-lb bag; 325-lb bbls	Cast iron, steel	Crystal	60–64	0.4 (32°F) 6.4 (86°F)	19.5% P ₂ O ₅	Precipitates Ca, Mg; pH of 1% solution, 9.1
Sodium hexameta-phosphate, (NaPO ₃) ₆	Calgon, glassy phosphate, vitreous phosphate	100-lb bags	Hard rubber, plastics, stainless steel	Crystal, flake, powder	47	1–4.2	66% P ₂ O ₅	pH of 0.25% solution, 6.0–8.3
Sodium hydroxide, NaOH	Caustic soda, soda, lye	100 and 700 lb drums; bulk (trucks, tank-cars)	Cast iron, rubber, steel	Flake Lump Liquid		2.4 (32°F) 4.4 (68°F) 4.8 (108°F)	98.9% NaOH, 74–76% NaO ₂	Solid; hygroscopic; pH of 1% solution, 12.9
Sulfuric acid, H ₂ SO ₄	Oil of vitriol, vitriol	Bottles; carboys; drums; trucks; tank cars	Concentrated: iron, or steel; dilute: glass, lead, porcelain, rubber	Solution (60–66°Be)		Complete	77.7% H ₂ SO ₄ (60°Be); 93.2% H ₂ SO ₄ (66°Be)	Approx. pH of 0.5% solution, 1.2
Tetrasodium pyrophosphate, Na ₃ P ₂ O ₇ · 10H ₂ O	Alkaline sodium pyrophosphate, TSPP	125-lb kegs; 200-lb bags; 300-lb bbls	Cast iron, steel	White powder	68	0.6 (80°F) 3.3 (212°F)	53% P ₂ O ₅	pH of 1% solution, 10.8

(continues)

TABLE 5.34 Characteristics of Chemicals Used in Water Treatment (*continued*)

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft ³	Solubility, lb/gal	Commercial strength	Characteristics
Stabilization and corrosion control								
Trisodium phosphate Na ₃ PO ₄ ·12H ₂ O	Normal sodium phosphate, tertiary sodium phosphate, TSP	125-lb kegs; 200-lb bags; 325-lb bbls	Cast iron, steel Medium Standard	Crystal: Coarse	56 58 61	0.1 (32°F) 13.0 (158°F)	19% P ₂ O ₅	pH of 1% solution, 11.9
Softening								
Calcium hydroxide, Ca(OH) ₂	Hydrated lime, slaked lime	50-lb bags; 100-lb bbls; bulk (carloads) trucks	Asphalt, cement, iron, rubber, steel	White powder: Light Dense		0.014 (68°F) 0.012 (90°F)	85–99% (Ca(OH) ₂) 63–73% (CaO)	Hopper agitation required for dry feed of light form
Calcium oxide,* CaO	Burnt lime, chemical lime, quick lime, unslaked lime	50-lb bags; 100-lb bbls; bulk (carloads)	Asphalt, cement, iron, rubber, steel	Lump, pebble, granule		Slakes to form hydrated lime	75–99% CaO	pH of saturated solution, 12.4; detention time, temperature, and amount of water all critical for efficient slaking
Sodium carbonate, Na ₂ CO ₃	Soda ash	Bags, bbls; bulk (carloads, trucks)	Iron, rubber, steel	White powder: Extralight Light Dense	23 35 65	1.5 (68°F) 2.3 (86°F)	99.4% Na ₂ CO ₃ 55% Na ₂ O	Hopper agitation required for dry feed of light and extralight forms; pH of 1% solution, 11.3
Sodium chloride,* NaCl	Common salt, salt	Bags; bbls; bulk (carloads)	Bronze, cement, rubber	Rock Fine		2.9 (32°F) 3.0 (68°F)	98% NaCl	
Taste and odor								
Activated carbon,* C	Aqua Nuchar, Hydrodarco Norite	Bags, bulk	Dry: iron, steel; wet: rubber, silicon, iron, stainless steel	Black granules, powder	15	Insoluble (suspension used)		

TABLE 5.34 Characteristics of Chemicals Used in Water Treatment (*continued*)

Chemical name and formula	Common or trade name	Shipping containers	Suitable handling materials	Available forms	Weight, lb/ft ³	Solubility, lb/gal	Commercial strength	Characteristics
Chlorine, Cl ₂	Chlorine gas, liquid chlorine	100 and 150 lb cylinders; 1-ton tanks; 16, 30 and 35 ton tank cars	Dry black iron, copper, steel; wet gas; glass, hard rubber, silver	Liquefied gas under pressure	91.7	0.07 (60°F) 0.04 (100°F)	99.8% Cl	
Chlorine dioxide, ClO ₂	Chlorine dioxide	Generated as used	Plastics, soft rubber (avoid hard rubber)	Yellow-red gas		0.02 (30 mm)	26.3% available Cl ₂	
Copper sulfate,* CuSO ₄ · 5H ₂ O	Blue vitriol, blue stone	100-lb. bags, 450-lb bbls: drums	Asphalt, silicon, iron, stainless steel	Crystal Lump Powder	75–90 73–80 60–64	2.6 (32°F) 2.2 (68°F) 2.6 (86°F)	99% CuSO ₄	
Ozone, O ₃	Oxone	Generated at site of application	Aluminum, ceramics glass	Colorless gas				
Potassium permanganate KMnO ₄	Purple salt	Bulk: bbls: drums	Iron, steel, wood	Pure crystals		Infinite	100%	Danger of explosion on contact with organic matters.

*For list of approved organic coagulant aids, see text.

Note: 1 lb = 0.45 kg; lb/ft³ = 15.90 kg/m³; 1 lb/gal = 119.82 kg/m³; 0.555 (°F - 22) = °C.

the low-tension member. Ozone production is controlled by the air or oxygen feed rate and the transformer voltage.

Ozone must be produced at the water treatment plant. Pipes leading from the ozonator are usually stainless steel, although PVC has been used successfully. Three methods of ozone introduction into the water are commonly used: (1) injection through a filter head at the base of a column contactor; (2) the Kerag system, in which gas is jetted into an impeller at the base of a contact column; and (3) diffusion through various media such as ceramic and stainless steel diffusers. Typically, the column provides 5 to 10 min of contact time between the ozone and the water. Ozone has multiple possibilities in water treatment. Ozone oxidizes organic matter (including bacteria and viruses), cyanide, iron, and manganese.

Potassium permanganate is also used in a limited manner for disinfection, although it is not as effective as the chlorine compounds. Its introduction to water requires a chemical feed system resistant to its corrosive

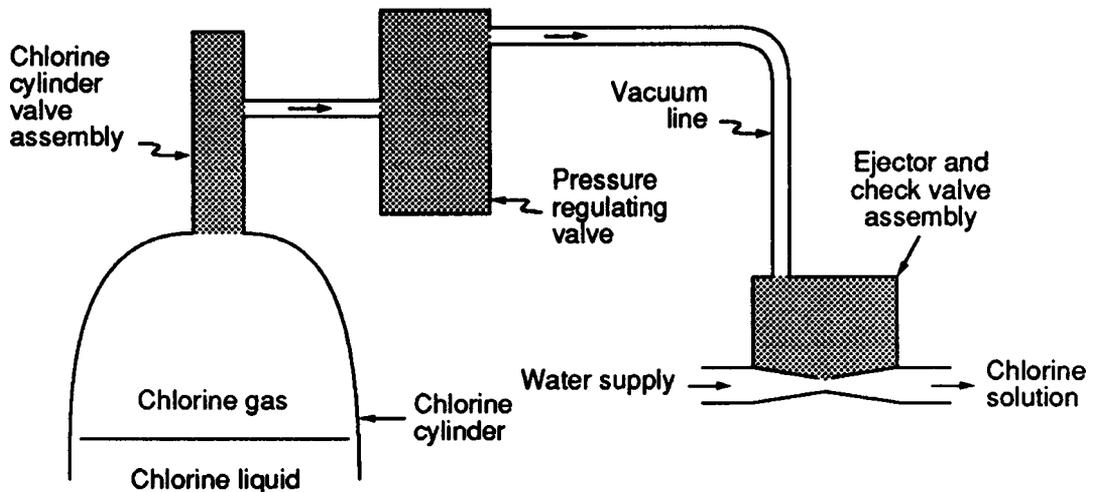


FIGURE 5.34 Chlorination flow diagram.

properties. Still potassium permanganate has several advantages which may be desirable in the water treatment process.

Because chlorination is the primary mode of disinfection in the United States, it will be discussed in more detail in the following sections.

Chemistry of Chlorination. Chlorine is used in the form of free chlorine or as hypochlorites. In either form it acts as a potent oxidizing agent and often dissipates itself in side reactions so rapidly that little disinfection is accomplished until amounts in excess of the chlorine demand have been added.

Reactions with Water. Chlorine combines with water to form hypochlorous and hydrochloric acids.



The former, being a weak acid ($K = 2.8 \times 10^{-8}$), is very poorly dissociated at pH levels below 6. In dilute solution and at pH levels above 4, the equilibrium shown above is displaced greatly to the right and very little Cl_2 exists as such in solution.

Hypochlorites are used largely in the form of modern-day high-test calcium hypochlorites. When such compounds are dissolved in water, they ionize to yield hypochlorite ion.



This ion, of course, established an equilibrium with hydrogen ions in accordance with the equation



The amounts of OCl^- ion and HOCl in the solution depend upon the pH, as shown in Figure 5.35 (50). Natural waters always contain some chloride ions, and so the hypochlorous acid and chloride and hydrogen ions present in the water exist in equilibrium with free chlorine, as shown in Eq. (5.24). Thus, it can be concluded that the same equilibria are established in water regardless of whether chlorine or hypochlorites are added. The significant difference would be in pH effects and the influence of chlorine or hypochlorites on the relative

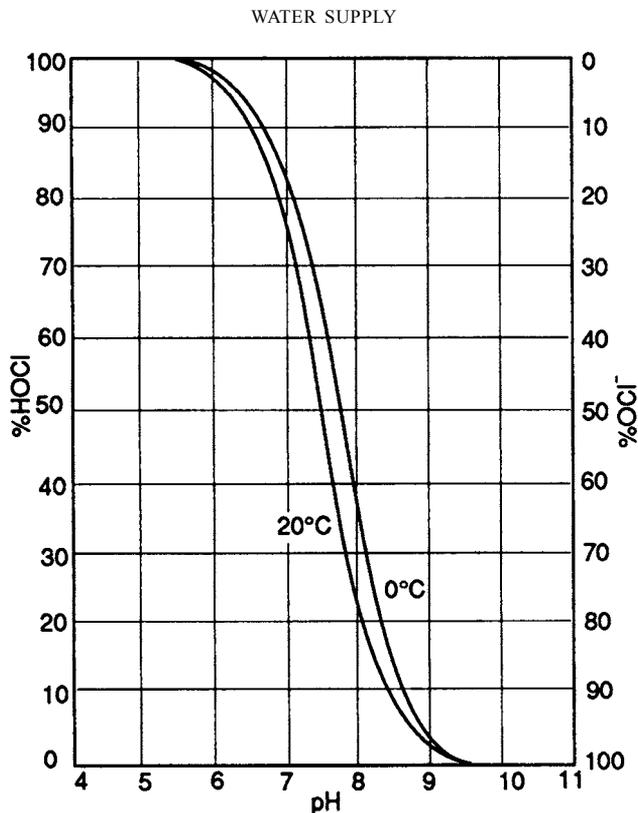
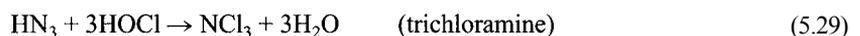
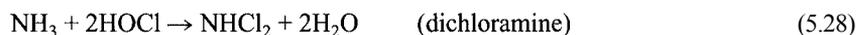
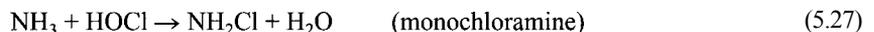


FIGURE 5.35 Effect of pH on the distribution of hypochlorous acid and hypochlorite ion in water (*Reprinted from Journal AWWA, Vol. 40, No. 10 (October 1948), by permission. Copyright © 1948, The American Water Works Association.*)

amounts of OCl^- and HOCl at equilibrium. Chlorine tends to decrease the pH, whereas hypochlorites tend to increase the pH.

Reactions with Ammonia. Ammonium ions exist in equilibrium with ammonia and hydrogen ions. The ammonia reacts with chlorine or hypochlorous acid to form monochloramines, dichloramines, and trichloramines, depending upon the relative amounts of each and to some extent upon the pH, as follows:



The mono- and dichloramines have significant disinfecting power and are, therefore, of interest in the measurement of chlorine residuals.

Extraneous Reactions. Chlorine combines with a wide variety of materials, particularly reducing agents. Many of the reactions are very rapid while others are much slower. These side reactions complicate the use

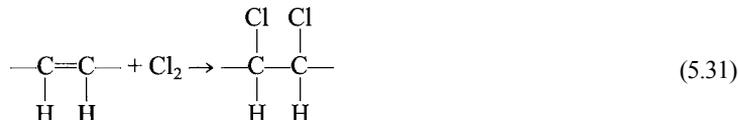
of chlorine for disinfecting purposes. Their demand for chlorine must be satisfied before chlorine becomes available to accomplish disinfection.

The reaction between hydrogen sulfide and chlorine will serve to illustrate the type of reaction that occurs with reducing agents.

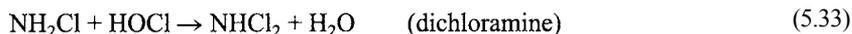
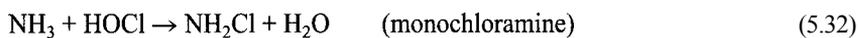


Fe^{2+} , Mn^{2+} , and NO_2^- are examples of other inorganic reducing agents present in water supplies. A few organic reducing agents may be present, but their concentrations are very low in potable waters.

Organic compounds that possess unsaturated linkages will also add chlorine and increase the chlorine demand.



Chlorine-Ammonia Reactions. The reactions of chlorine with ammonia are of great significance in water chlorination processes, especially disinfection. As previously noted, the kinetics of these reactions have been extensively studied. When chlorine is added to water containing natural or added ammonia (ammonium ion exists in equilibrium with ammonia and hydrogen ions), the ammonia reacts with HOCl to form various chloramines which, like HOCl, retain the oxidizing power of the chlorine (51–54). The reactions between chlorine and ammonia may be simply represented by



The distribution of the reaction products is governed by the rates of formation of monochloramine and dichloramine, which are dependent upon pH, temperature, time, and initial $\text{Cl}_2:\text{NH}_3$ ratio (50, 55). In general, high $\text{Cl}_2:\text{NH}_3$ ratios, low temperatures, and low pH levels favor dichloramine formation. Although reported data differ in some degree, it is evident that some dichloramine can be anticipated at pH levels below 7. Palin's work (55, 56) further suggests that at pH levels below about 7.5, some nitrogen trichloride can be anticipated. Dependent on a number of factors, such as the free ammonia and organic nitrogen content of the raw water, the level of free residual chlorination applied, contact time, pH, and type of water plant, nitrogen trichloride can pose a considerable problem which may be disposed of by various means. The subject is a complex one and has been extensively treated elsewhere (57–59).

Chlorine Residuals. Disinfection is a process designed to kill harmful organisms, and it does not ordinarily produce a sterile water. These generalizations hold for disinfection with chlorine also. Two factors are extremely important in disinfection: time of contact and concentration of the disinfecting agent. Where other factors are constant, the disinfecting action may be represented by

$$\text{Kill} = C \times t \quad (5.35)$$

The important point is that with long contact times a low concentration of disinfectant suffices, whereas short contact times require high concentration to accomplish equivalent kills. Furthermore, research has shown that a greater concentration of chloramines than of hypochlorous acid is required to accomplish a given kill in a specified time. For these reasons it is important to know both the concentration and the kind of residual chlorine acting.

From the considerations mentioned above, it has become common practice to refer to chlorine, hypochlorous acid, and hypochlorite ion as free chlorine residuals, and the chloramines are called combined chlorine residuals.

The rate of the reaction between ammonia and hypochlorous acid varies considerably, depending upon the pH and temperature. The reaction rate is most rapid at pH 8.3 and decreases rapidly as the pH is decreased or increased. For this reason, it is common to find free chlorine and combined chlorine residuals coexisting after contact periods of 10, 15, or even 60 min.

The action of excess chlorine on waters containing ammonia merits special consideration. With mole ratios of chlorine to ammonia up to 1:1, both monochloramine and dichloramine are formed, the relative amounts of each being a function of the pH. Further increases in the mole ratio of chlorine to ammonia result in formation of some trichloramine and oxidation of part of the ammonia to nitrogen gas. These reactions are essentially complete when 2 moles of chlorine has been added for each mole of ammonia nitrogen originally present in the water. Chloramine residuals usually reach a maximum when 1 mole of chlorine has been added for each mole of ammonia and then decline to a minimum value at a chlorine-to-ammonia ratio of 2:1. Further additions of chlorine produce free chlorine residuals. Chlorination of a water to the extent that all the ammonia is converted to trichloramine or oxidized to free nitrogen or other gases is referred to as *breakpoint chlorination* because of the peculiar character of the chlorine residual curve, as illustrated in Figure 5.36.

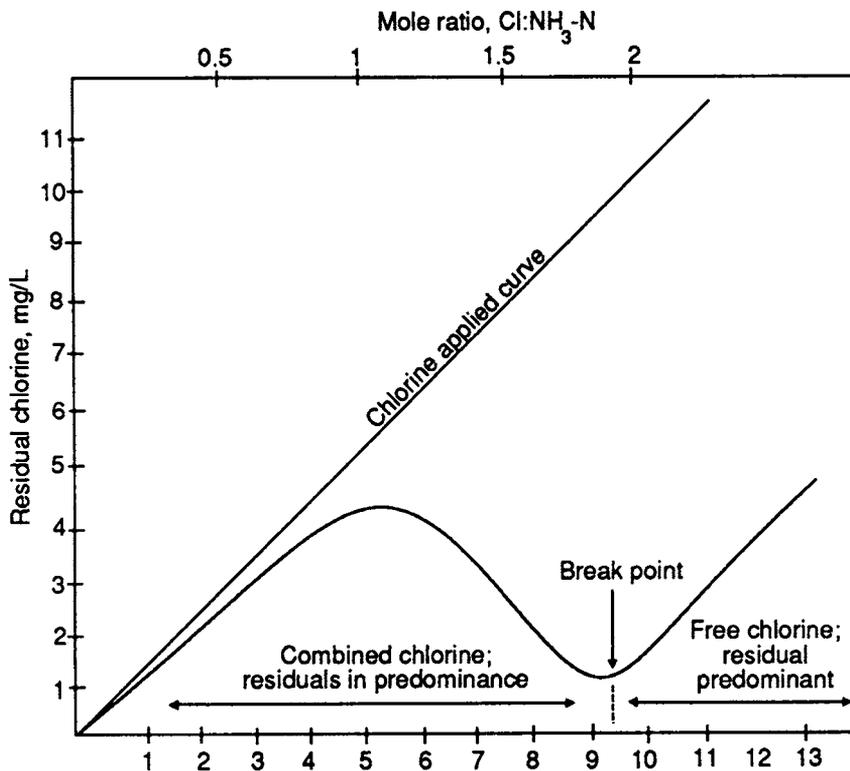


FIGURE 5.36 A residual-chlorine curve showing a typical breakpoint. Ammonia-nitrogen content of water, 1.0 mg/L. (Reprinted from *Journal AWWA*, Vol. 40, No. 10 (October 1948), by permission. Copyright © 1948, The American Water Works Association.)

Theoretically, it should require 3 moles of chlorine for the complete conversion of 1 mole of ammonia to nitrogen trichloride (trichloramine). The fact that only 2 moles of chlorine is required to reach the break point indicates that some unusual reactions occur. Nitrous oxide, nitrogen, and nitrogen trichloride have been identified among the gaseous products of the breakpoint reaction. The presence of nitrous oxide could be accounted for by the following reaction:



The total chlorine required for formation of the monochloramine, dichloramine, and the hypochlorous acid for the final oxidation step corresponds to 2 moles for each mole of ammonia. This would indicate that nitrous oxide is the major end product when ammonia is oxidized by chlorine in dilute solutions.

Bactericidal Residuals. Minimum bactericidal chlorine residuals were investigated from 1944 to 1948 by the U.S. Public Health Service. Data secured in experiments at temperatures from 20 to 25°C, using 10-min exposures to free available chlorine and 60-min exposures to combined available chlorine, are the basis for recommended minimum safe residuals illustrated in Figure 5.37 (60, 61). Table 5.35 summarizes the recommended residual levels for free and combined chlorine.

Cysticidal Residuals. Minimum cysticidal chlorine residuals recommended by the National Research Council for military water supplies, based on the investigations by Chang, Fair, and others (61–64) are illustrated in Figures 5.38 to 5.40. It is significant to note that minimum chlorine requirements for cyst destruction are greater than for bacteria destruction and that a diminished effect at reduced temperatures is more pronounced even under the best conditions of operation. Thus, more than 50-ppm free available residual after 30-min contact would be necessary when the pH is 8.3 and 9.5, respectively, at 2 to 5°C and 22 to 25°C. Waters containing significant amounts of organic matter or high bacterial or cyst densities would require either higher 30-min residuals or longer contact periods than those illustrated.

Viricidal and Other Residuals. Complete data are unavailable upon which to base a recommendation on residual requirements to ensure destruction of waterborne viruses (65). From studies using human volunteers, it is apparent that complete control of the causative hepatitis virus depends almost entirely on the proper use of a suitable disinfectant; it is not destroyed or inactivated by coagulation, settling, or diatomite filtration alone (66). According to Clark and Chang, satisfactory control is obtained by prechlorinating to maintain a 0.3- to 0.4-mg/L free available chlorine residual throughout treatment and 0.2- to 0.3-mg/L free available residual in the plant effluent (67). The chance of survival of any human enteric virus after such treatment is remote, especially when satisfactory flocculation is employed in the treatment regimen.

Nematodes reportedly are not demobilized after 2 h of contact by a free available chlorine residual of 2.5 to 3 mg/L or after 1 mm of contact by a 15- to 45-mg/L free available residual. Also, while the organisms they might have ingested might be expected to survive only a day or two, such organisms are completely protected against chlorine even when 90% or more of the nematodes have been immobilized (68). Where infested waters are a problem, it currently is recommended that supplies be prechlorinated for 6 h to maintain a free available residual of 0.4 to 0.5 mg/L; this treatment might not destroy nematodes, but it attenuates most and renders them immobile and easily removed by settling processes.

The adoption and maintenance of fixed, minimum free, or combined available chlorine residuals serve only as operating guides and should not minimize, nor can they replace, cultural or microscopic analyses. Demonstration of the absence of disease-producing organisms is the only positive means of assuring safety.

Points of Application. When chlorination of water was first employed for disinfection, terminal treatment of the purification plant effluent was almost invariably practiced. Now the use of chlorine in various stages of water treatment and even in the distribution system is common practice. Multiple- or split-chlorination schemes are increasingly employed for disinfection and other purposes and frequently enhance the efficiency of many water-treatment unit processes. Fundamentally, the points at which chlorine is applied depend on the specific objectives of chlorination and on technical, practical, safety, economic, and other considerations.

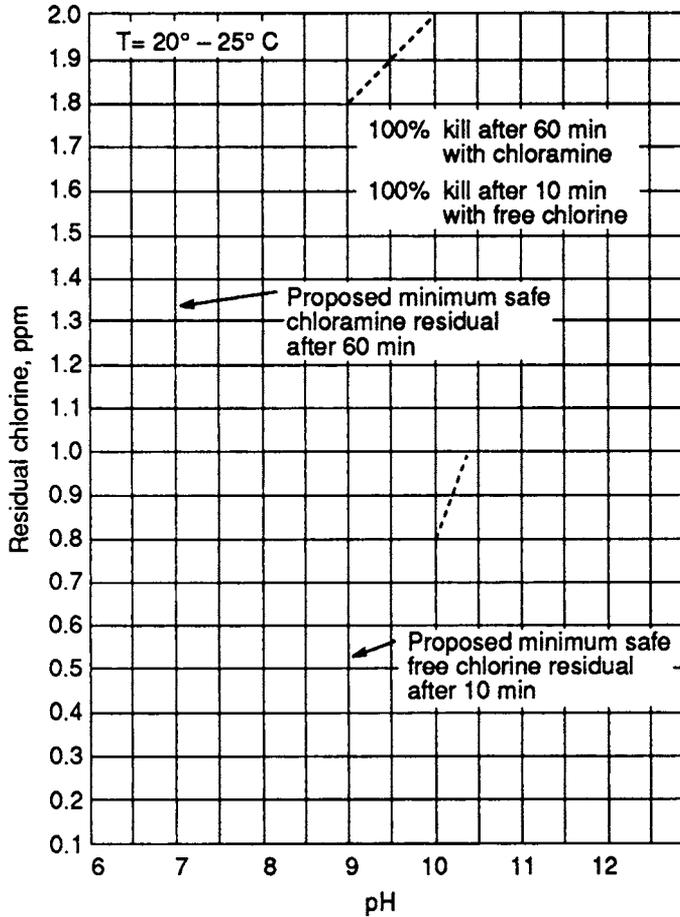


FIGURE 5.37 Minimum free and combined bactericidal chlorine residuals. (Reprinted from *Journal AWWA*, Vol. 40, No. 12 (December 1948), by permission. Copyright © 1948, The American Water Works Association.)

TABLE 5.35 Free Chlorine and Combined Chlorine Residuals

Types of chlorination	pH level	Safe residual, mg/L
Free*	6.0–8.0	0.2 or greater
Free*	8.0–9.0	0.4 or greater
Free*	9.0–10.0	0.8 or greater
Free*	10.0–greater	1.0 or greater
Combined†	6.0–7.0	1.0 or greater
Combined†	7.0–8.0	1.5 or greater
Combined†	8.0–9.0	1.8 or greater

* 10-min contact time.

†160-min contact time.

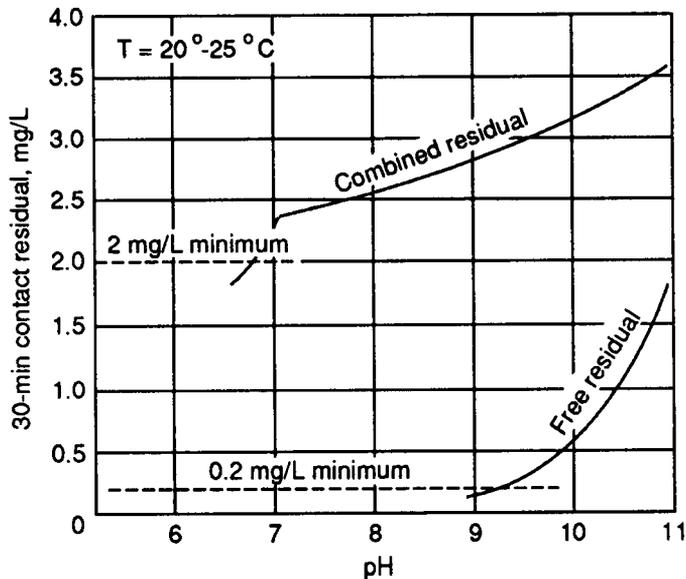


FIGURE 5.38 Minimum free and combined bactericidal chlorine residuals. (Reprinted from *Journal AWWA*, Vol. 40, No. 12 (December 1956), by permission. Copyright © 1948, The American Water Works Association.)

in practice, various terms have come into common usage to specify the point and location at which chlorine is applied.

Plain or simple chlorination involves the application of chlorine to water that receives no other treatment. More than half of all existing water treatment facilities in the United States fall into this category and, thus, this treatment represents the sole public health safeguard. When applied to impounded or naturally elevated surface systems, chlorine usually is injected into the pipeline leading from such a gravity supply. When applied to water pumped into a system, chlorine usually is added at the pump suction using pressure beyond the pump to operate the chlorine feeders.

Disinfection and Disinfection By-Products

The U.S. Environmental Protection Agency (EPA) is in the process of developing a sophisticated regulatory strategy in an attempt to balance the complex trade-offs in risks associated with controlling disinfectants and disinfection by-products (D/DBP) in drinking water (146). To accomplish this goal, the EPA published a proposed two-stage rule for disinfectants/disinfection by-products and an enhanced surface water treatment rule (ESWTR) on July 29, 1994. A stage-one D/DBP rule and an interim ESWTR (IESWTR) were to be published first. An initial information collection rule (ICR) was to be developed and the data from that rule were to be used to make decisions regarding stage one and the IESWTR. This was especially true for the IESWTR. The ICR has experienced significant delays. The final expedited rule for both (stage one of the D/DBP rule and the IESWTR) is expected in November 1998. Stage two of the D/DBP rule and the final ESWTR will be based on ICR data, so it will probably be a long period of time before they are developed.

The EPA first attempted to control disinfection by-products in 1974, after trihalomethane (THM) formation in drinking water was identified as a by-product of chlorination. Based on available toxicologic data,

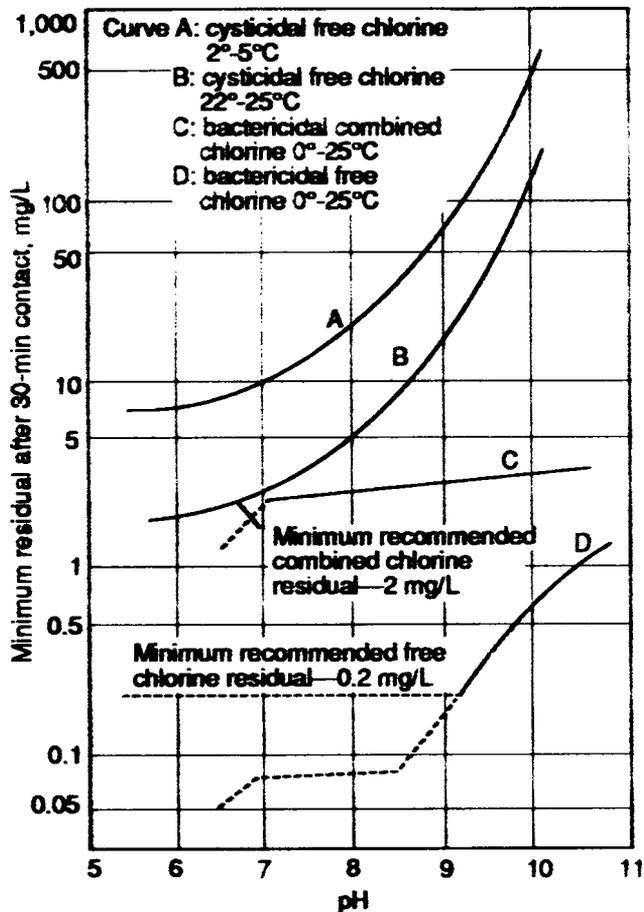


FIGURE 5.39 Minimum free and combined bactericidal and cysticidal chlorine residuals. (Reprinted from *Journal AWWA*, Vol. 40, No. 12 (December 1956), by permission. Copyright © 1948, The American Water Works Association.)

chloroform, one of the THMs was labeled as a suspect carcinogen. Epidemiological studies also suggested a human risk. Because of these suspected health effects and the potential that a large number of drinking water consumers would be exposed to these by-products, a Total Trihalomethane (TTHM) Regulation was promulgated on November 29, 1979, at a level of 0.10 mg/L. Since that time, many other objectional by-products of chlorination have been identified as well.

Section 1401 (1)(D) of the Safe Drinking Water Act, Public Law 93-523, requires that for any regulated contaminant a "Treatment Techniques" document must accompany a Maximum Contaminant Level (MCL) to provide compliance guidance. In order to satisfy this requirement, the Drinking Water Research Division of EPA's Office of Research and Development prepared a comprehensive document entitled "Treatment Techniques for Controlling Trihalomethanes in Drinking Water," which was published in September 1981 (147).

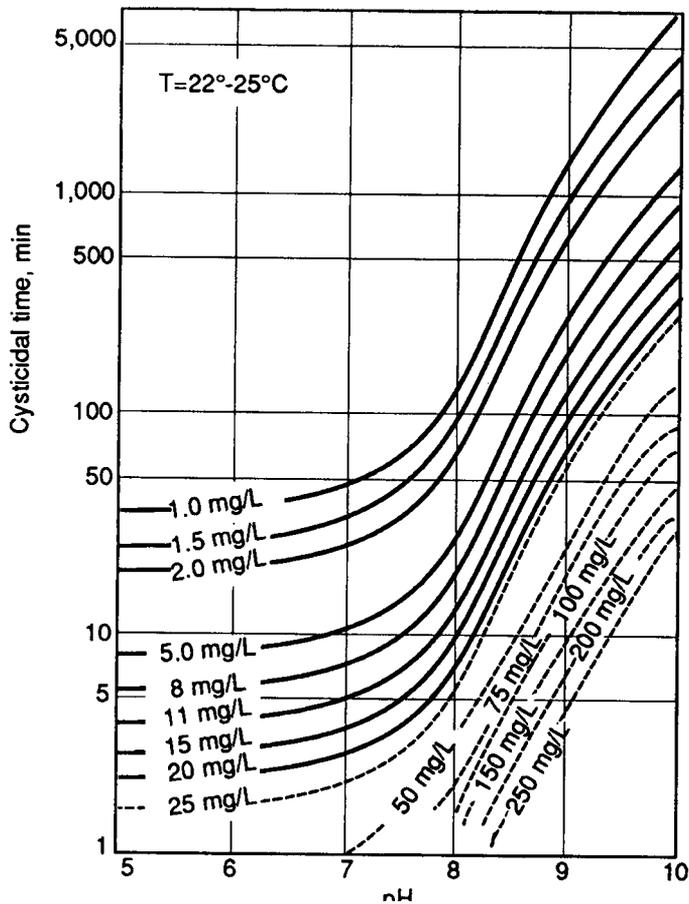


FIGURE 5.40 Minimum contact time for free residual chlorine destruction of *E. histolytica*. (Reprinted from *Journal AWWA*, Vol. 40, No. 12 (December 1956), by permission. Copyright © 1948, The American Water Works Association)

Disinfection Practice. Chlorine has been the most commonly used disinfectant in the United States, where an estimated 220,000,000 people receive disinfected drinking water (146). Chlorine has been the disinfectant of choice for many utilities, with more than 50% of the systems using surface water and chlorinating before settling and filtration. Many utilities in the United States have explored the use of alternate disinfectants other than chlorine to lower their THM levels below the 0.10 mg/L MCL.

Some utilities have considered switching to chloramines as an alternate disinfectant to chlorine. A survey conducted by the American Water Works Association showed that the vast majority of utilities that changed disinfection practices have switched to chloramines (148). Some utilities are considering the possibility of using ozone as a disinfectant followed by chlorine or chloramines.

Ozone is drawing increasing interest but concern with using ozone includes the need for biostabilization of the treated water and possible formation of by-products such as aldehydes, ketones, and acids. Chlorine

dioxide is an effective disinfectant but there are concerns over the formation of the inorganic by-products chlorite and chlorate when it is used.

Microbiological. Prior to the discovery that protozoan cysts (*Giardia* and *Cryptosporidium*) were a prime cause of waterborne disease outbreaks, the apparent attainment of adequate disinfection was fairly easy. Many of the commonly available chemical disinfectants (chlorine, ozone, chlorine dioxide, and even chloramines) are successful in reducing coliform bacteria to acceptable levels, which is generally an indication of pathogen-free water. However, it is now known that pathogens can exist even in the presence of elevated free chlorine. The EPA has, therefore, evaluated the common chemical disinfectants for their efficacy in inactivating *Giardia* cysts and viruses.

The EPA has adopted the CT approach (concentration in mg/L \times time in minutes) concept. In comparing the biocidal effectiveness of disinfectants, major considerations are the disinfectant concentration and the time needed to attain inactivation of a certain microbial population exposed under specific conditions. The CT concept can be expressed as an empirical equation as shown below:

$$K = C^n \cdot t$$

Where C = disinfectant concentration in mg/L

n = coefficient of dilution

t = contact time in minutes required for a fixed percent of inactivation

K = constant for a specific microorganism

CT values have been developed for inactivation of various microorganisms for the major disinfectants. An example of these values is shown in Table 5.36 (146).

Ozone shows the highest efficiency, inactivating 99% of most types of microorganisms at very low CT values. Chloramine shows the lowest efficiency. Chloramine CT values for viral agents, in particular, are very high. Free chlorine and chlorine dioxide at pH 6 to 7 are approximately equivalent for polio virus-1 inactivation. Free chlorine appears to be more effective than chlorine dioxide for inactivation of rotavirus and *E. coil*, whereas chlorine dioxide appears more effective than free chlorine for *G. muris* and *G. lamblia* cysts.

TABLE 5.36 Summary of CT Value Ranges for Inactivation of Various Microorganisms by Disinfectants

Microorganism	Free chlorine, pH 6 to 7	Preformed chloramine, pH 8 to 9	Chlorine dioxide, pH 6 to 7	Ozone, pH 6 to 7
<i>E. coli</i>	0.034–0.05	95–180	0.4–0.75	0.02
Polio virus-1	1.1–2.5	768–3740	0.2–6.7	0.1–0.2
Rotavirus	0.01–0.05	3806–6476	0.2–2.1	0.006–0.06
Phage f ₂	0.08–0.18	ND	ND	ND
<i>G. lamblia</i> cysts	47–>150	2200 ^a	26 ^a	0.5–0.6
<i>G. muris</i> cysts	30–630	1400	7.2–18.5	1.8–2.0
<i>Cryptosporidium parvum</i>	7200 ^b	7200 ^c	78 ^d	5–10 ^e

Note: All CT values are for 99% inactivation at 5°C, except for *Giardia lamblia* and *Cryptosporidium parvum*.

^aValues for 99.9% inactivation at pH 6–9.

^b99% inactivation at pH 7 and 25°C.

^c90% inactivation at pH 7 and 25°C.

^d90% inactivation at pH 7 and 25°C.

^e99% inactivation at pH 7 and 25°C.

ND—No data.

Other Treatment Goals. Oftentimes, disinfection also can perform other treatment tasks in a drinking water treatment plant, for example: oxidation of metals, taste and odor control, and enhancement of turbidity removal. Probably the greatest need for an oxidant, other than disinfection, is for precipitation of metals, such as iron and manganese. These metals occur in the reduced inorganic state and in metal-organic complexes, which are the most difficult forms to remove. Ozone, the most powerful oxidant, performs extremely well. Chlorine dioxide also oxidizes metals very well. Both are more effective at removing metals than chlorine. Chemical costs can be considerably higher, however, particularly if there is a high oxidant demand.

Softening

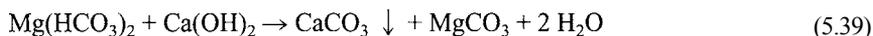
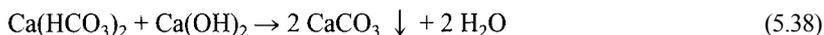
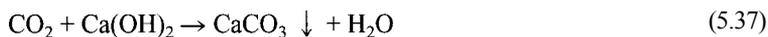
Softening is the removal of ions that cause hardness in water. Hardness is caused mainly by calcium and magnesium ions and at times by iron, manganese, strontium, and aluminum ions. Hardness causes excessive soap consumption for washing operations and scale formation in pumps, boilers, and pipes. Public water supplies should not exceed 300 to 500 mg/L of hardness; however, aesthetically, a hardness greater than 150 mg/L is unacceptable.

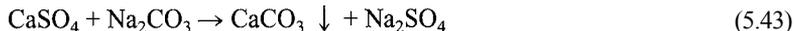
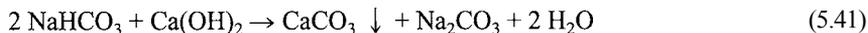
There are two methods to soften water: precipitation and ion exchange. The lime-soda ash precipitation process removes most of the available hardness. Lime will precipitate calcium oxide, calcium bicarbonate, and magnesium bicarbonate, in that order. These compounds are commonly identified as carbonate hardness. Noncarbonate hardness may then be removed with soda ash. The sulfate and chloride anions associated with calcium and magnesium comprise the noncarbonate hardness. After all hardness is removed, the water is recarbonated using carbon dioxide to reduce the pH to near 7 and thereby preclude production of a corrosive product water. Lime-soda ash softening is capable of reducing total hardness to 30 to 40 mg/L as calcium carbonate and 10 mg/L magnesium hydroxide as calcium carbonate.

Lime-Soda Processes. The hardness of almost all water supplies is caused by the presence, in solution, of calcium and magnesium compounds. Other bivalent ions, such as strontium, ferrous iron, and manganese, may contribute to the hardness to a lesser degree. The anions with which these bivalent ions are associated are principally the bicarbonate and sulfate ions, although chlorides and nitrates of calcium and magnesium do occur in some natural waters. The softening process consists of removing a part of these salts from the water to reduce the hardness to a predetermined value consistent with the reduction of detergent consumption, the control of scale formation, the prevention of corrosion, and other factors that make for a high-quality water. Calcium and magnesium bicarbonates are designated as *carbonate hardness*, and calcium and magnesium sulfates (or chlorides and nitrates) are called *noncarbonate hardness*.

The alkalinity determination on raw water ordinarily measures the carbonate hardness, but in some raw and softened waters the alkalinity may also include some sodium alkalinity if the total alkalinity exceeds the total hardness. Sodium alkalinity is often termed *negative noncarbonate hardness*, since on reacting with lime, sodium carbonate is formed, which is used for removing noncarbonate hardness. The noncarbonate hardness is measured by the difference between the total hardness and the carbonate hardness, and requires soda ash for its reduction or removal. The sum of the alkalinity plus the noncarbonate hardness equals total hardness if the water contains no sodium alkalinity.

The chemistry of water softening is probably best explained or illustrated by showing the chemical reactions that take place when lime and soda ash are added to water containing calcium and magnesium salts. The reactions in the lime-soda process, then, are





These equations show all of the reactions taking place in softening a water containing both carbonate and noncarbonate hardness by the lime–soda process. It should be noted that in Eq. (5.37), the carbon dioxide is not hardness as such, but in proportion to its content in the water will consume lime and must, therefore, be considered in calculating the amount of lime required. Similarly, in Eq. (5.41), the sodium bicarbonate or sodium alkalinity, if present, is not part of the hardness, but since it is included in the total alkalinity, it will consume lime. Equations (5.39) and (5.41) show the removal of carbonate hardness by lime. Although only one molecule of lime is required for one molecule of calcium bicarbonate (Eq. (5.39)), two molecules of lime are required for the removal of one molecule of magnesium bicarbonate (Eqs. (5.39) and (5.40)). Equation (5.42) shows the removal of magnesium noncarbonate hardness, usually magnesium sulfate, by lime. No softening is effected by this reaction because, for each molecule of magnesium noncarbonate hardness removed, an equivalent amount of calcium noncarbonate hardness is formed. Equation (5.43) shows the removal of calcium noncarbonate hardness, usually calcium sulfate, either originally in the water or formed as shown in Eq. (5.43).

From these reactions, it is apparent that the amounts of lime and soda ash required to soften a water may be calculated from the concentrations of free carbon dioxide, bicarbonate (usually the total alkalinity), magnesium hardness, and noncarbonate hardness. Various equations have been proposed for calculating dosages of lime and soda ash. However, since free carbon dioxide is usually determined as milligrams per liter of CO_2 , and the alkalinity and various kinds of hardnesses are determined as milligrams per liter of equivalent calcium carbonate, it is simpler to leave these terms in these forms.

To remove carbonate hardness and magnesium:

$$\text{lb 100\% CaO per million gal} = 10.6x \text{ mg/L CO}_2 + 4.7x \text{ mg/L alkalinity} + 4.7x \text{ mg/L Mg hardness} \quad (5.44)$$

where x is the desired excess hydroxide alkalinity.

To remove noncarbonate hardness:

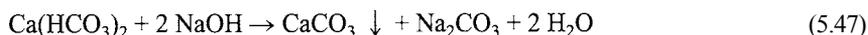
$$\text{lb 100\% Na}_2\text{CO}_3 \text{ per million gal} = 8.8 \text{ mg/L noncarbonate hardness} - x \quad (5.45)$$

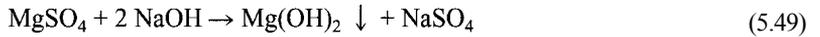
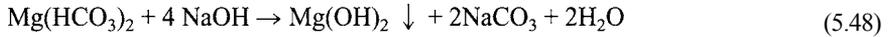
where x is the noncarbonate hardness to be left in the water.

These formulas are based on pure chemical compounds. The better commercial grades of high-calcium quicklime contain about 88 to 95% available CaO; high-calcium hydrated lime contains approximately 68 to 70% CaO. Commercial soda ash contains 99 to 100% Na_2CO_3 . The results from Eq. (5.44) should be divided by the percent CaO in the lime used and multiplied by 100. If the exact analysis of the quicklime is not known, the results may be divided by 0.90, and for hydrated lime, by 0.68.

Temperature, reaction-zone efficiencies, time, and catalytic effect of retained slurries will influence the completeness of reactions, and the stoichiometric calculations shown above will have to be adjusted to meet plant conditions.

An alternative method in the lime–soda process that should be mentioned here is the use of caustic soda instead of soda ash. The reactions of caustic soda with the carbonate and noncarbonate hardness are





Lime-Soda Treatment Processes. Softening can be selective with reference to the removal of calcium and magnesium hardness. Water relatively low in magnesium hardness can be treated with lime to remove the calcium carbonate hardness. If magnesium is to be removed, excess lime is added. If noncarbonate hardness is to be reduced, soda ash is added. Caustic soda (NaOH) may be substituted for lime (Ca(OH)₂) and soda ash (Na₂CO₃). The softened water needs to be stabilized before the water is applied to filters. The several processes can be accomplished in a number of flow patterns, as illustrated in Figures 5.41 to 5.43 (69).

Figure 5.41 shows single-stage conventional softening. Depending upon the kinds and amounts of chemical added, the calcium and magnesium carbonate and noncarbonate hardness can be removed. Figure 5.42 depicts two-stage treatment and two-stage recarbonation. This arrangement permits greater removal of the magnesium hardness and promotes greater control of the effluent characteristics. It also provides the lowest practical level of hardness in the effluent. Figure 5.43 shows a “split-treatment” arrangement. This system also may be tailored to provide selective calcium carbonate hardness reduction or calcium and magnesium hardness reduction. Feeding excess lime to the main stream removes the magnesium hardness. The softened and unsoftened waters are mixed prior to recarbonation, with CO₂ added in the minimum amount necessary to provide stable water. This process flow arrangement frequently obviates the need for CO₂ addition.

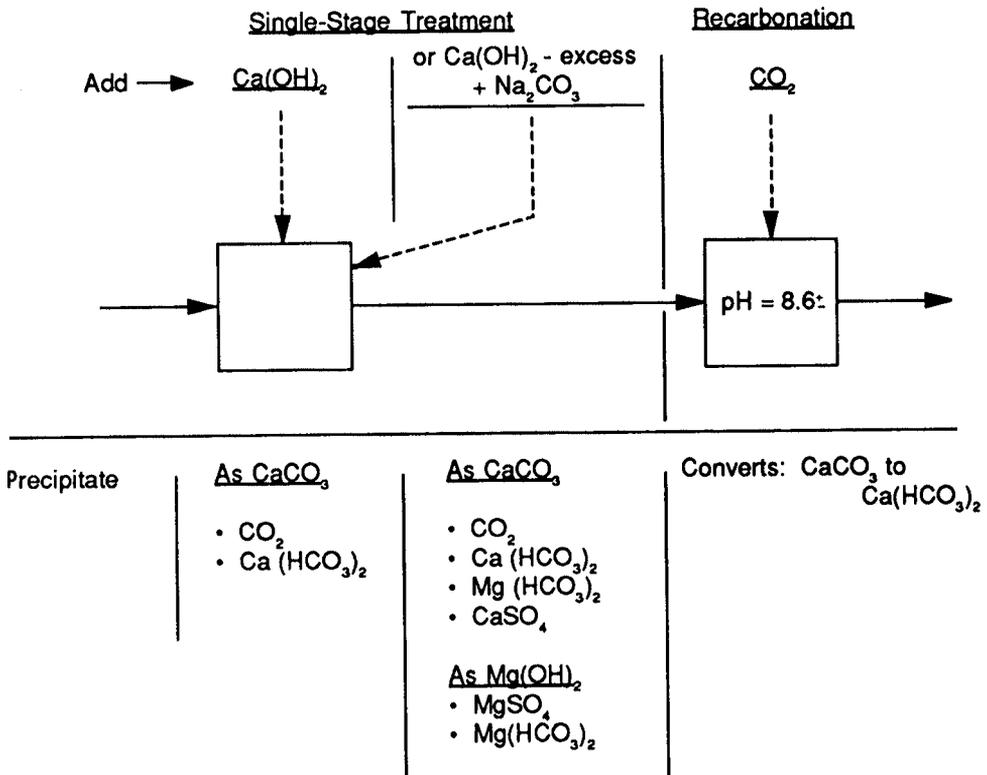


FIGURE 5.41 Single-stage softening and recarbonation (69).

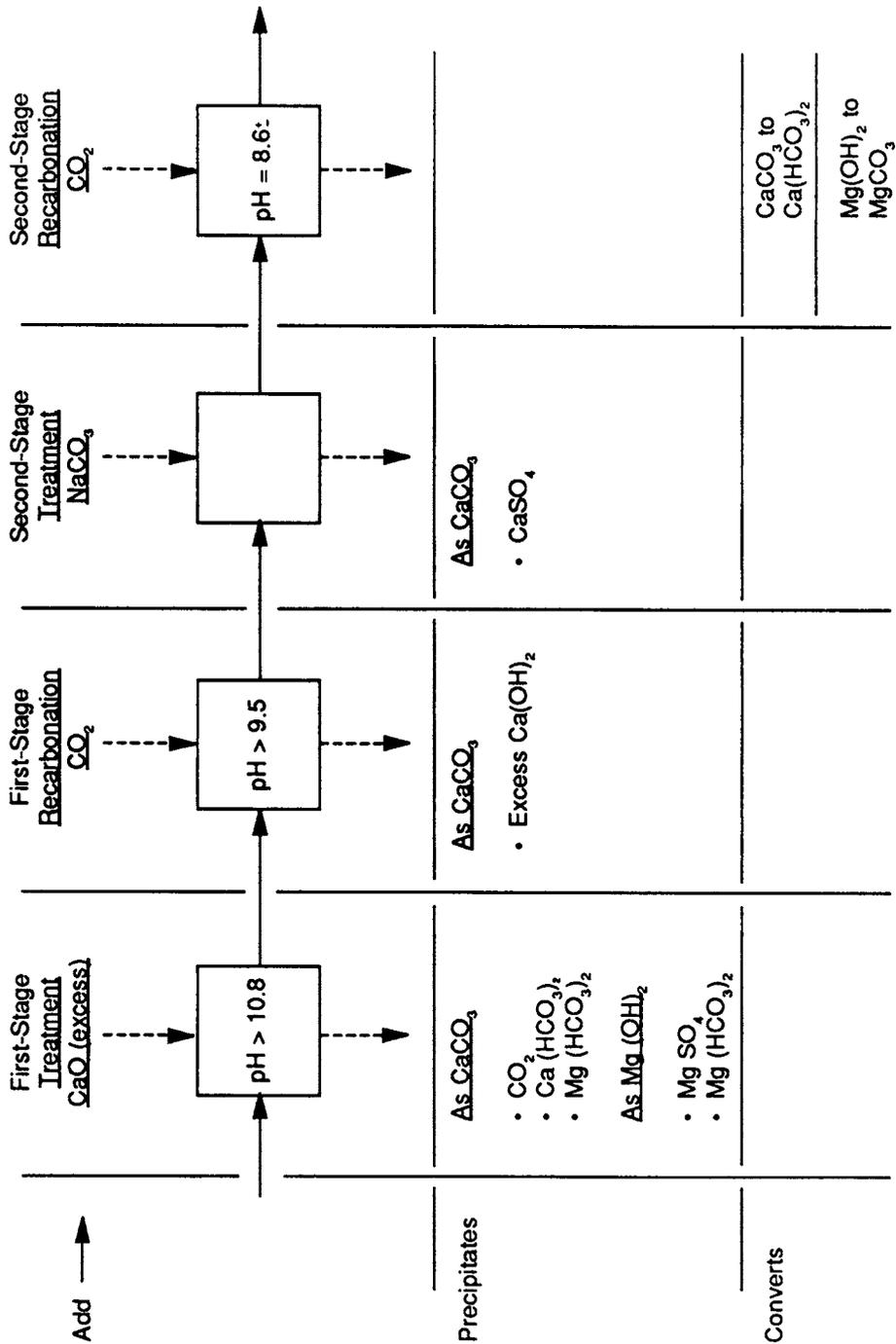


FIGURE 5.42 Two-stage softening and recarbonation (69).

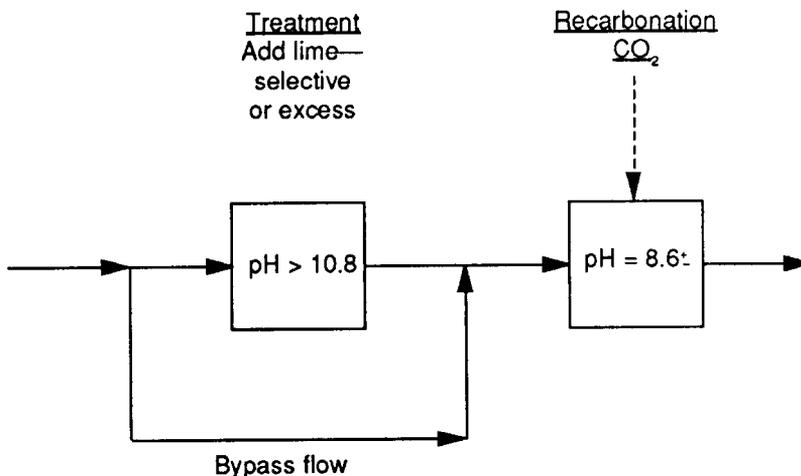


FIGURE 5.43 Split treatment and recarbonation (69).

Other modifications have been used for specific problems. Where efficient lime recovery and maximum hardness reduction are necessary, three-stage processes have been proposed (70). This process involves the precipitation of CaCO_3 in the first stage, followed by the addition of excess lime in the second stage to precipitate magnesium hydroxide, followed by two-stage recarbonation.

Although not shown on any of the diagrams, aeration may be considered to be a part of the softening process.

Choice of Reagents. The choice of reagents, lime-soda ash versus caustic, is affected by two parameters (Table 5.37).

TABLE 5.37 Chemical Dosage Requirements for Softening (67)

Source of demand	Chemical dosage required,* lb/million gal [†]		
	Lime-soda softening		
	CaO as CaO	Na_2CO_3 as Na_2CO_3	Caustic softening, NaOH as NaOH
CO_2 [‡]	10.61	None	15.16
Ca CH	4.67	None	6.67
Mg CH	9.34	None	13.34
Mg NCH	4.67	8.84	6.67
Ca NCH	None	8.84	None
Na-Alk	4.67	-8.84	None

*Comparison of lime soda and caustic softening: stoichiometric dosages of pure chemicals for 1 mg/L (as CaCO) of demand source.

[†]lb/million gallons = 1.2×10^{-4} kg/m³.

[‡]As CO_2 all others expressed as CaCO_3 .

Note: Ca CH = calcium, carbonate hardness; Mg CH = magnesium, carbonate hardness; Mg NCH = magnesium, noncarbonate hardness; Ca NCH = calcium, noncarbonate hardness.

- *Total dissolved solids*: The total dissolved solids increase with the use of caustic because all reactions produce soluble soda ash (Na_2CO_3), sodium sulfate (Na_2SO_4), or sodium chloride (NaCl).
- *Sludge production*: Reaction with caustic soda produces less sludge than reaction with lime–soda ash, as follows:
 - No sludge is produced for CO_2 or Na-alk.
 - About 30 to 40% as much sludge is produced for magnesium removal.
 - Half as much sludge is produced for calcium carbonate hardness removal.
- The same amount of sludge is produced for calcium noncarbonate hardness removal.

The comparative quantities of sludge production are summarized in Table 5.38.

Design Factors. The softening process is influenced by the following factors:

1. The rate of dissolution of chemical reagents
2. The rate of precipitate formation
3. The growth rate of precipitated particles to a size that can be removed
4. The settling rates of the floc

CaO and $\text{Ca}(\text{OH})_2$ dissolve relatively slowly. Although 30 s may be adequate to dissolve coagulant in the process stream, 5 to 10 min may be required for lime (67). Similarly, while high G levels are required to disperse coagulants, softening reactions generally require mixing only to keep the lime particles in suspension and distributed uniformly during the time of mixing.

The rate of precipitate formation of CaCO_3 is temperature-dependent. Cole reports that water at a temperature of 1°C took about 90 min to reach its minimum hardness; at 22°C , it was reached in about 30 min (71). The data are summarized in Figure 5.44.

The rates of formation and growth of precipitate are usually addressed only in qualitative terms. The recirculation of sludge enhances the growth of the precipitate. Sludge recirculation is an integral part of solids contact basin system design. The reaction produced by lime in raw water is very slow in the absence of crystallization nuclei. However, if the lime and the water are mixed in the presence of precipitated CaCO_3 , the reaction time is reduced and the stability of the softened water is enhanced.

Settling rates are a function of particle size and particle density. As either increases, the rate of settling increases. Inasmuch as the water density increases with lowering temperatures and the rate of formation of

TABLE 5.38 Softening Sludge Production

Source of demand	lb Dry solids, lb/million gal*	
	Lime soda	NaOH
CO_2^*	18.95	None
Ca CH	16.68	8.34
Mg CH	16.68	4.84
Mg NCH	13.18	4.84
Ca NCH	8.34	8.34
Na-Alk	8.34	None

Stoichiometric amounts from 1 mg/L as CaCO_3^\dagger of demand source.

*lb/million gallons = 1.2×10^{-4} kg/m³.

†Except for CO_2 , which is expressed as mg/L CO_2 .

Note: CH = carbonate hardness; NCH = noncarbonate hardness.

Source: From Ref. 66.

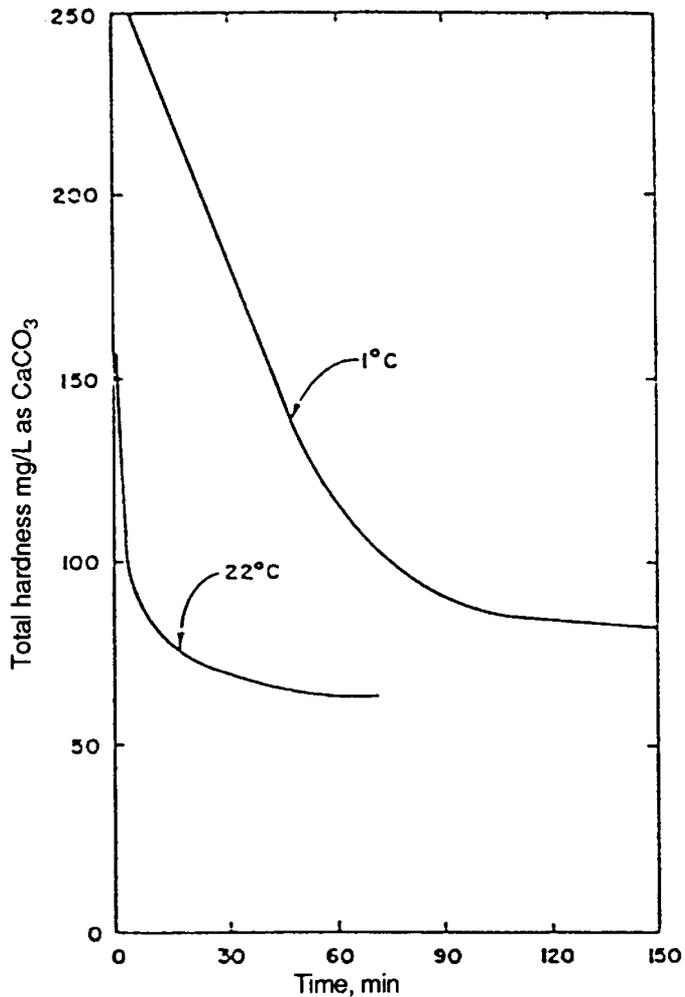


FIGURE 5.44 Lime-softening rate. (Reprinted from *Journal AWWA*, Vol. 68, No. 1 (January 1976), by permission. Copyright © 1976, The American Water Works Association.)

CaCO₃ decreases with lowering temperatures, low-temperature operations usually require larger settling tanks and the use of coagulant aids to increase the size and density of the floc.

Selective Calcium Carbonate Removal. When it is not necessary to remove practically all the magnesium hardness or when the raw water is low in magnesium hardness so that special methods are not needed for its reduction, treatment with just enough lime to precipitate the carbonate hardness may be used. This is often called *selective calcium carbonate removal*. If it is desired to reduce the carbonate to the minimum, enough lime should be added to obtain the pH of maximum carbonate concentration. This pH value varies with temperature and total alkalinity as shown in Figure 5.45, and at this point twice the phenolphthalein alkalinity equals the total alkalinity, or $2P = T$ (72). Soda ash may or may not be required, depending on the amount of

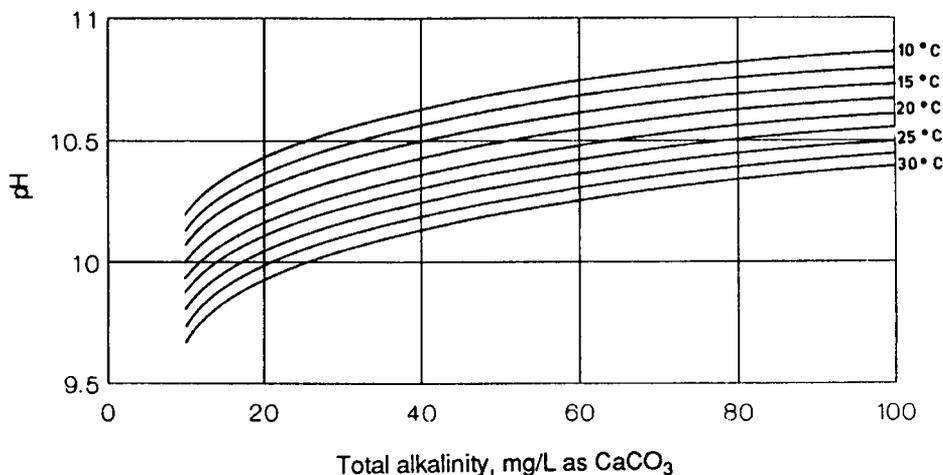


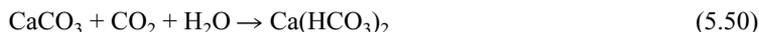
FIGURE 5.45 Total alkalinity versus maximum carbonate ion concentration at several tempera-

noncarbonate hardness present in the raw water. Sludge recirculation may be used to aid in calcium carbonate precipitation, and recarbonation is usually practiced to produce a stable effluent. Often sodium polyphosphates are also used to inhibit incrustation of the filter sand and scale formation in the distribution system. Although extremely low hardnesses are not usually required or desirable for municipal use, by this method the calcium carbonate hardness may be reduced to 15 or 20 mg/L.

Excess-Lime Treatment. With waters containing more than about 40 mg/L magnesium hardness (as CaCO_3) it is usually necessary to resort to excess-lime treatment to obtain the desired magnesium reduction. Since it is not good practice to pump highly caustic water to consumers, some means must be used to neutralize the excess lime remaining in solution after settling out the precipitated magnesium hydroxide and calcium carbonate in the primary basin. The easiest method is to convert it to calcium carbonate by recarbonating or by the use of bypassed raw water (split treatment).

Figure 5.46 illustrates the magnesium reduction obtained in the primary basin in one plant with various hydroxide alkalinities. These hydroxide alkalinities are total, not excess. Thus, while a reduction to 40 mg/L is obtained by a total hydroxide alkalinity of 54 mg/L, the excess hydroxide alkalinity required is $54 - 40$, or only 14 mg/L. Also, it should be noted that a curve based on the solubility product K_{sp} for magnesium hydroxide would be appreciably lower, and other plants may obtain a closer approach to the theoretical solubility than the plant results illustrated in Figure 5.46.

Recarbonation. Recarbonation is used for two purposes in a lime-softening plant: (1) for pH adjustment, and (2) for the neutralization and precipitation of the calcium hydroxide in the primary basin effluent in cases where high-magnesium waters are softened by application of excess lime followed by recarbonation. When a low-magnesium hard water is treated with the amount of lime necessary to react with the free CO_2 and bicarbonates, as in the process of selective calcium carbonate removal, the water remains supersaturated with calcium carbonate and may have a pH as high as 10.4 or more. This water would be highly encrusting, and the pH should be adjusted downward before filtration. Carbon dioxide gas is bubbled through the water for this purpose. The carbon dioxide combines with all or a part of the unprecipitated calcium carbonate to form some soluble bicarbonate, the amount depending on the final pH desired. The reaction is



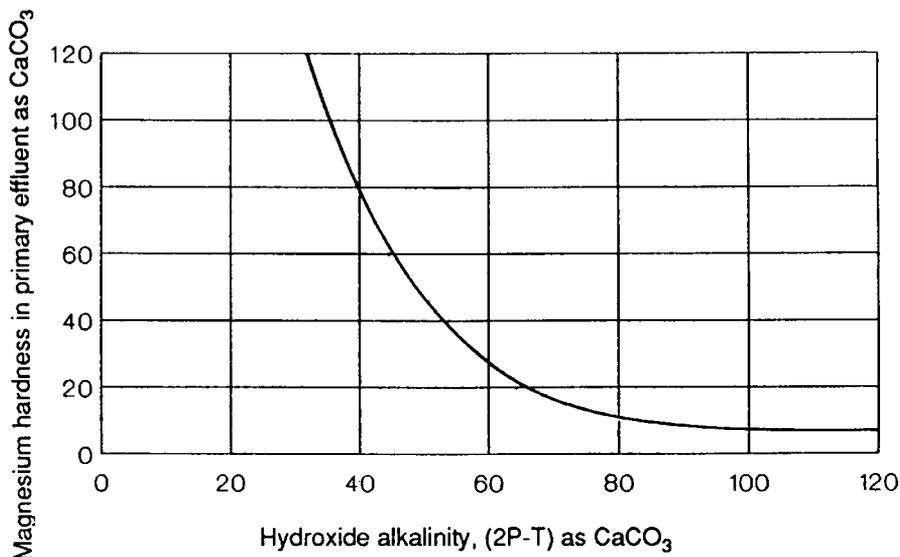
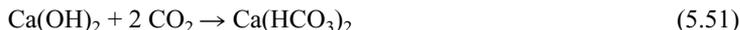
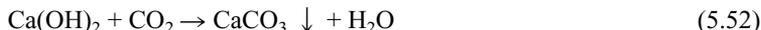


FIGURE 5.46 Magnesium hardness versus primary hydroxide alkalinity.

If enough lime has been added to precipitate some magnesium hydroxide, the hydroxide alkalinity which remains must also be converted to bicarbonate according to the following reaction:



If, however, the method of excess lime followed by recarbonation is used, there is too much calcium hydroxide present after sedimentation, and converting it to the soluble bicarbonate would unduly increase the hardness. It is necessary to precipitate it as calcium carbonate according to the reaction



Sometimes it is possible to add enough additional carbon dioxide to convert a portion of the carbonate to the bicarbonate in the recarbonation basin between the primary and secondary basins, and thus lower the pH to the desired figure. But because of the danger of redissolving some of the precipitated calcium carbonate, it is better to make a final pH adjustment by a final recarbonation just after filtration.

Sludge Disposal. Disposal of the large quantities of sludge formed in plants where lime-soda ash is used to soften water is a serious problem and must be planned in advance. The principal methods used are: (1) discharge into water courses, (2) discharge into sewers, (3) ponding or lagooning, (4) filtering in a vacuum filter and using the resultant cake for landfill or as agricultural lime, and (5) recalcination of the dewatered, dried sludge.

Ion-Exchange Softening. Three methods may be used for water softening by ion exchange. They are sodium cycle, split stream, and demineralization. Only sodium cycle will be given major attention in this section. The major thrust in softening is the use of sodium cycle operation (cation exchange). Softening of water by ion exchange consists essentially of passing hard water through a bed of ion exchanger. There the hardness components of the water, calcium and magnesium ions, are removed from solution and replaced in solution by sodium. In practice the exchange rate is so rapid that almost any water can be easily and completely softened at normal flow rates regardless of hardness.

Cation-Exchange Reactions. It is easy to indicate by chemical equations the reactions which occur in cation-exchange water softening and in regeneration. The exact mechanism of the reaction, however, is not clearly understood. Whatever the final explanation of the cation-exchange reaction may be, it is evident that its successful completion is dependent upon securing adequate contact of the water with the exchange material. Other conditions being equal, adequate contact is probably the most important requisite in successful cation exchange water softening, and to secure this contact requires experienced correlation of all such important influencing factors as hardness of water, depth of material, rate of flow, and character of the exchanger. Thus, it follows that if continued maximum contact is to be secured, reasonably clear water must be applied to the exchanger to avoid coating of its particles with colloidal or suspended matter from the water.

The following schematic equations, in which R represents the anionic component of the ion exchanger, indicate the reactions which take place in cation-exchange softening:



Carbonate hardness	Soluble nonhardness
-----------------------	------------------------



Noncarbonate hardness	Soluble nonhardness
--------------------------	------------------------

And when the exhausted exchanger is regenerated, essentially the reverse action takes place:



These equations emphasize the base-exchange nature of cation-exchange softening, indicating that the only change in the composition of the water is the replacement of the hardness-producing bases—calcium and magnesium—by sodium, which does not produce hardness. In this respect, it is worth noting that the softening reaction may usually be carried to such a degree of completion that no calcium or magnesium can be found in the softened water by ordinary analytical methods; in other words, with the majority of waters, a properly designed and operated cation-exchange softener is capable of delivering water with a hardness that is actually zero, even though, in industrial and municipal practice, the term “zero hardness” may actually mean only “insignificant hardness.”

Water Composition. The composition of the influent water may have a marked effect upon the capacity and hardness leakage of the resin. As noted in the chemistry of softening, the calcium and magnesium ions exchange for sodium ions on the resin. In regeneration the reverse reaction takes place. It can be assumed that at some theoretical sodium-hardness ratio neither softening nor regeneration can take place. Hardness alone in the influent water has little effect upon the capacity of the resin. As the sodium content increases, competition develops between the sodium and hardness ions for the active sites on the resin. In most water supplies, the sodium-hardness ratio is low, and the effect upon capacity is hardly noticeable. In Figure 5.47, the effects of increasing hardness in a water with a sodium-hardness ratio of 1:3 are plotted as a function of

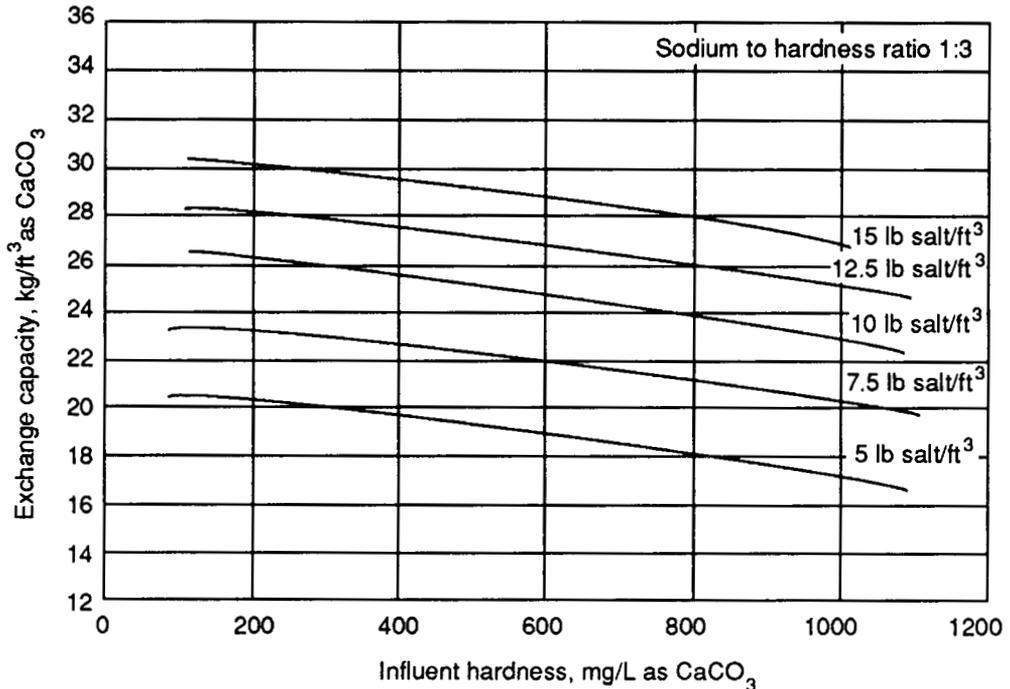


FIGURE 5.47 Ion-exchange capacity versus water composition.

capacity. Only when the sodium-hardness ratio exceeds 2:1 will there be any real need to be concerned about the lowering of capacity due to decrease in influent water quality (73). As this latter limit is approached, there should be more concern about the quality of the finished water than in the process.

An increase in the sodium-hardness ratio causes an increase in the hardness leakage of the softener. In those installations where the final water needs to be extremely soft, this may be a problem, as indicated in Figure 5.48. In municipal plants, where the softener effluent is mixed with unsoftened water to produce an intermediate hardness, leakage is no problem. Softening of problem waters alone without additional treatment should be thoroughly investigated before the final design is made.

Regeneration Cycle. On completion of the softening cycle, the softener should be backwashed to loosen the ion-exchange resin, flush out any particulate matter which may have collected on top of the bed, and re-grade the exchanger to make it more receptive to the regeneration cycle in preventing channeling of brine through the bed. Two to five minutes is sufficient for the backwash cycle. Manufacturers' engineering data give the time required and the recommended upflow rates to obtain maximum cleaning of the ion-exchange resin. However, final adjustments must be made on the individual installation to suit the conditions encountered. With gravity upflow softeners, the backwash operation is eliminated.

In regenerating the pressure softener, brine is introduced to the bed at a controlled rate through a distributor system set immediately above the exchanger surface. To obtain maximum softening capacity from the ion exchanger, it is essential that the brine pass uniformly through and contact all parts of the resin bed. Several different techniques are used in introducing brine to the softeners. In some small domestic softeners, the tank is drained down several inches, and rock salt is dropped directly onto the top of the resin. Water is passed, at a controlled rate, through the salt to produce a brine of the proper concentration. In some cases,

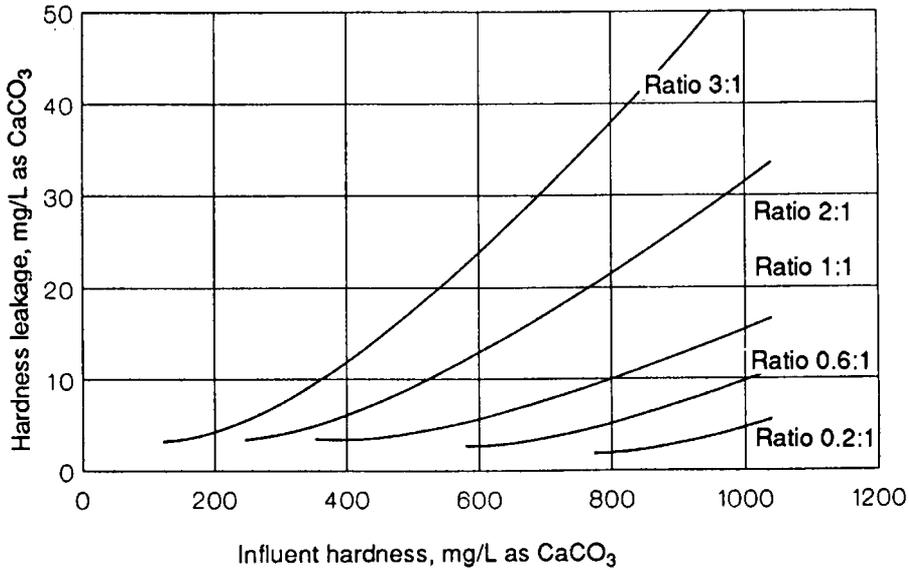


FIGURE 5.48 Hardness leakage versus influent hardness at various sodium-to-hardness ratios.

the salt is allowed to stand for several hours to dissolve, after which water is introduced at the top to force the brine down through the bed. In better quality domestic softeners and industrial units up to 28.38 m³ (1000 ft³), concentrated brine is pulled from the storage tank by water eductors and delivered to the brine distributor in the softener shell. The eductor water serves not only as the motive force but also as a dilutant to bring the brine to the proper concentration. The time of education determines the quantity of brine delivered to the unit.

Design Factors. Design of a softening system must be based upon the flow, influent water quality, effluent water quality requirements, resin exchange capacity, resin hydraulic characteristics, period of time between regeneration, type of operation (manual or automatic), and the number of units to be installed.

In the sizing of softener units, it is essential to first determine the volume of ion exchanger required (27):

$$E = \frac{KQH}{G} \tag{5.61}$$

where E = volume of exchange material to be used, ft³ (m³)

Q = volume of water to be treated between regenerations, ft³ (m³)

H = hardness of water, grains per gal (m³)

G = hardness to be removed between regenerations per unit of volume of exchange material, grains per gal (m³)

K = a factor dependent on the units (7.48 for U.S. Customary units and 1000 for SI units)

In manual operation consideration should be given to calculating Q to make the regenerations fall during the day, when labor can be used most effectively. Shell diameter and bed depth should be adjusted to maintain a unit rate in the range of 4 to 8 gpm/ft² (0.22 to 0.32 m³/min/m²).

Sludge Disposal

Sludge is a waste by-product generated in potable water treatment. Specifically, sludges are generated in the coagulation, softening, and settling operations and are also generated as settled particulates from rapid-sand-filter backwash waters. Sludge disposal in water treatment must therefore be considered if any of the aforementioned unit processes is used.

Disposal may be accomplished by discharge into a sanitary sewer, discharge into a stream, or landfilling. Since the latter of these alternatives has been adversely affected by recent environmental legislation, storage has become a more viable option. Sludge may be permanently lagooned; landfilled with prior temporary lagooning or prior mechanical dewatering by vacuum filtration, centrifugation, etc.; or used to fill natural or man-made depressions, such as strip mine areas, borrow pits, and quarries.

Sludge has other possible uses, with or without prior dewatering and drying, such as farm- or pasture-land application. Dewatered sludge may be used in road stabilization. Some lime softening sludges may be recycled by recalcination if the sludge is of sufficient quality and the process is economically feasible. Quicklime is recovered for later use in softening during recalcination.

Local conditions, the type of sludge generated, and regulatory requisites define strategies applied to waste sludge storage and treatment. Figure 5.49 describes the sources and possibilities for disposal of water treatment sludges. As can be seen in the figure, alum sludge is difficult to dewater and thus requires pressure filtration after it has been conditioned with lime. The process producing the alum sludge could be modified by adding a polyelectrolyte coagulant aid to enhance presedimentation. This would likely improve sludge dewatering. In large plants with high lime concentrations, recovery of coagulants may be possible by recalcination. This process involves gravity thickening, mechanical dewatering, removal of magnesium hydroxide, and heating.

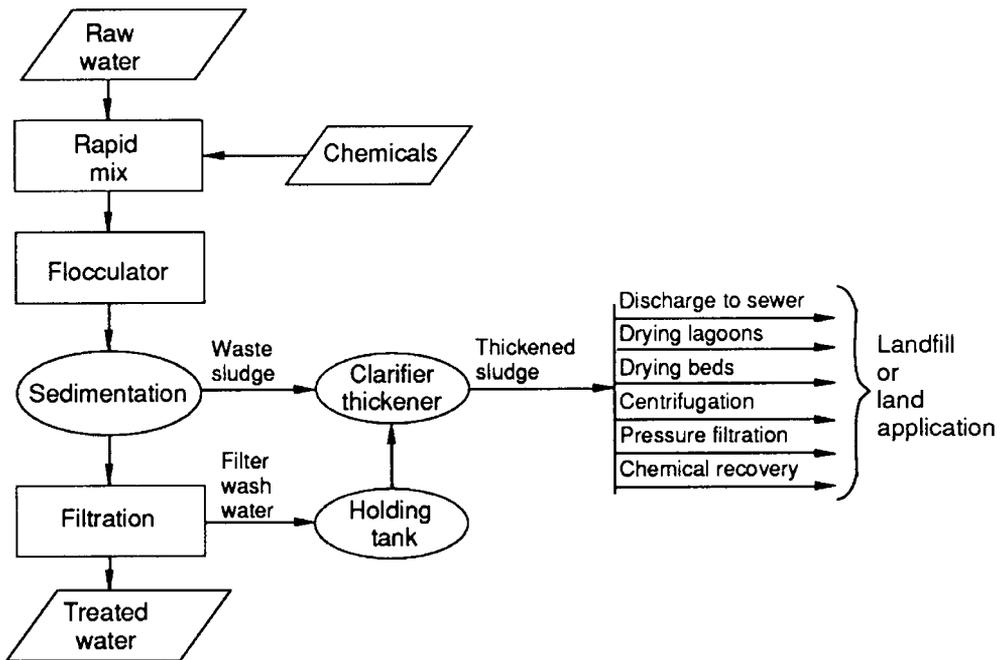


FIGURE 5.49 Sludge source and ultimate disposal.

Coagulation sludges from treatment of turbid waters are difficult to recalcine and recovery of alum is seldom practiced because of low process efficiency, poor quality of the alum recovered, and low cost of filter alum. Gravity thickening is the most straightforward and least expensive method of sludge volume reduction. The design of this process is based upon the settling properties of the sludge; therefore, coagulants or polymers may be added to form larger, heavier floc particles which have high terminal settling velocities and form dense sludge.

Land Application. Land application, either land spreading or in sanitary landfills, is widely practiced as a sludge disposal technique. Typically, 1 acre (0.4 ha) of land can accommodate approximately 10 tons (0.9 t) of sludge solids, depending upon the heavy metal concentration of the sludge. However, availability of land and public acceptance of this practice often eliminates it as an alternative. Sanitary landfills, when located near urban areas, where transportation costs would not be high, are a reasonable solution to the sludge problem if federal, state, and local regulations do not restrict this practice due to the heavy metal concentrations. A program of systematic deposit, compaction, and covering may be accomplished in conjunction with domestic solid wastes.

Vacuum Filters. Rotary drum vacuum filtration may be used for dewatering chemical water treatment plant sludges. The filtration medium consists of natural or synthetic fabric or coil springs. During operation 20 to 40% of the drum surface is submerged in a sludge vat below the drum. A vacuum of 10 to 20 in (25.4 to 50.8 cm) of mercury is maintained inside the submerged part of the drum. Water is drawn into the drum and solids are deposited on media surrounding the drum. The cake grows in thickness and is lifted from the sludge by slow rotation. Solids are removed from the drum by mechanical scraping with a “doctor blade” or by blowback with air pressure from within the drum. Inorganics dewater to 70% solids and biological materials dewater to 20% solids.

Filter Presses. Filter presses are used to dewater chemical sludges to a higher degree than vacuum filters. A series of parallel plates covered on both sides with filter cloths and shaped to permit drainage comprise the filter press. Sludge is introduced under pressure into the spaces separating successive plates and effluent water is forced through the cloth, leaving solids behind. After a period of “pressing time” the press is opened and the accumulated solids are collected. Several hundred kilograms per square centimeter (pounds per square inch) of pressure is required to pass the filtrate through the cake. Time required for the process is dependent on the nature of the sludge; the cake normally contains 50 to 60% solids.

Centrifuges. Centrifugation is another method employed in sludge dewatering. Solid bowl centrifugation is considered the best method, but basket types have been used successfully. The conical horizontal solid bowl type of centrifuge takes in sludge through its hollow shaft. The high-speed rotation of the bowl causes the sludge to occupy the space at the perimeter of the bowl. Angular momentum due to differential density between sludge, solids, and liquid cause the solids to consolidate along the wall. Solids are removed to discharge points by a helical screw that rotates at a speed slightly greater than that of the bowl. The primary problem encountered with a conical centrifuge is solids loss in liquids returned to the slurry pool.

Basket bowl centrifuges are a batch process for sludge thickening or dewatering. Two types of systems may be utilized: solid bowl or perforated bowl. In the solid bowl systems, the raw or digested sludge enters at the center and is spun against the bowl wall. The solids in the solid basket bowl configuration collect at the bottom while the supernatant is collected as overflow from the top of the bowl. The perforated basket bowl centrifuge is lined with a filter medium which during operation collects solids and allows effluent to pass through the perforated bowl into the outer bowl where it is collected. At the end of the cycle when the maximum allowable cake has built up, a knife or plow cuts out the cake for disposal as dewatered sludge.

Basket bowl centrifuges are used when solids recovery and cake dryness are most important. Centrifuges produce dewatered cakes comparable to those produced by vacuum filtration.

Drying Beds. Sludge-drying beds may be applied in dewatering chemical sludges from water treatment plants. A bed is usually composed of a 15-cm (6-in) sand layer underlain with a 12-in (30.58-cm) gravel layer with open-joint tile drains. The bed is encompassed by low walls to contain the wet sludge. Beds are dosed with 6 to 12 in (15.3 to 30.5 cm) of chemical sludge and then allowed to dry. Drying occurs due to two phenomena: (1) drainage of the water through the sand filter medium (yield 13 to 22% solids) and (2) by evaporation (achieving 25 to 44% solids). The rate of drying is a function of temperature, air movement, humidity, and rainfall (if uncovered). Once dried, sludge is removed from the beds with a front-end loader or shovel. The primary disadvantage of drying beds includes labor cost to remove sludge, land area requirements, and climatic conditions. Improved production may be achieved by covering beds or introducing flocculation to sludge prior to application.

ALTERNATIVE TREATMENT PROCESSES

Biologically active filtration, ion exchange, adsorption, reverse osmosis, and aeration are additional water treatment processes that may have special independent applications or may be incorporated into conventional treatment trains to achieve finished water quality goals.

Biologically Active Filtration

Biologically active filtration (BAF) is primarily utilized by plants that disinfect with ozone. The process is used to reduce organic matter measured as assimilable organic carbon (AOC) or biodegradable dissolved organic carbon (BDOC). The level of AOC/BDOC is increased in the ozonation process as a result of the breakdown of high molecular weight natural organic molecules into lower weight molecules that serve as nutrients for the heterotrophic and possibly opportunistic pathogenic bacteria in the water. In a biologically active filter, the AOC/BDOC concentration is reduced by the heterotrophic bacteria that colonize the filter medium (149).

The removal of the biodegradable organic matter is important for water quality in the distribution system because dissolved organic carbon can react with disinfectants to form disinfection by-products (DBPs) and can serve as a nutrient source for heterotrophic bacteria that colonize the distribution system, creating biofilms.

Biologically active filtration is a broad descriptive term for a filtration process that achieves some degree of AOC/BDOC removal through microbial degradation. The process can be a separate unit filtration process following filtration for particle removal; or particle and AOC/BDOC removal can be combined in a single filtration process. In the latter case, the filter often is a multimedia filter that includes activated carbon. A sand filter will support a low level of microbial growth that reduces AOC/BDOC, but not as effectively as an activated carbon filter.

The capability of biologically active filters to remove AOC/BDOC is a function of a variety of factors including filter medium, temperature, biomass, contact time, backwash conditions, and filter ripening stage. Although biologically active filters reduce the food source for bacteria in the distribution system, they also can contribute bacteria to the distribution system through sloughing of biomass in the filter effluent; there are concerns that certain opportunistic pathogens might colonize and proliferate in the filter.

Biologically active filtration is common in Europe, where it has been used (often as a second filtration step) for taste and odor control and for biostability. Given the increasing use of ozone as a disinfectant in the

United States, and concerns about DBPs and microbial regrowth problems in the distribution system, more water supplies are expected to install some form of biologically active filtration.

Ion Exchange

Ion exchange involves the transfer of one ion for another. For example, a cation in solution attaches to the solid exchanger (resin), which in turn releases a different cation into the solution (74). All exchangers exhibit selectivity; they may prefer one ion over another by a factor of 15 or more. This preference is not, however, a fixed number but varies with ionic strength, relative amounts of ions, the kind of exchanger, and to a lesser extent with other factors, such as temperature. Some of these factors influence preference to such an extent that the selectivity for one ion at a given concentration may even reverse if the solution is greatly diluted. The ratios of one ion to others in solution also has a significant effect on selectivity. Some exchangers have a high affinity for a particular ion, while others do not. For example, clinoptilolite (a natural zeolite) has a very strong preference for ammonium, whereas synthetic resin exchangers do not. This preference for one ion over another is the property that makes ion exchange so valuable.

Many common materials are exchangers, including natural zeolites (such as greensand), clay (especially montmorillonite), sulfonated coal (crushed anthracite treated with concentrated sulfuric acid), and peat. But the synthetic resins with their high capacity and controllable properties make large-scale ion exchange practical and feasible. Were the benefits of ion exchange more fully understood, the municipal use of ion exchangers would be more widespread and popular.

Ion exchange has the following advantages:

- It is selective. It can often remove unwanted ions preferentially. Examples of preferential removal include nitrate, iron, manganese, ammonium, and heavy metals.
- There are many manufacturers of ion-exchange equipment, which keeps costs competitive.
- The process and the equipment have been tested over many years. Designs are well developed into “off-the-shelf” units that are rugged and reliable.
- Manual and completely automatic units are available.
- Temperature effects from 0 to 35°C are negligible.
- The process is excellent for softening in small and large installations.

Ion exchange has the following disadvantages:

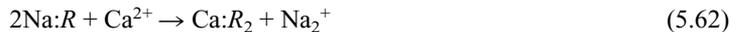
- Chemicals for regeneration may be expensive, corrosive, or even dangerous, and the waste regenerant may present a difficult disposal problem.
- Automatic plants require expert maintenance and nonautomatic plants require knowledgeable operation and frequent attention.
- Capital cost is high (although it can be reduced by optimizing design).
- The expensive ion-exchange resins can be easily ruined by ignorant operation. Examples are fouling of strong base resins by humic acids and oxidation of iron and manganese.

There are many kinds of exchangers, but for the purpose of this discussion the most important ones are shown in Table 5.39. Exchangers are generally divided into four classifications depending upon the kind of functional group, which determines whether cations or anions are exchanged and whether the resin is a strong or weak electrolyte (strong acid, strong base or weak acid, weak base). Strong acid and strong base resins operate at all pH values, but their capacity is limited so they must be regenerated frequently. Because their regeneration is inefficient, the cost of chemicals is high. Weak electrolyte resins have much higher capacity and regenerate almost stoichiometrically (efficiency approaches 90%), but they operate only over a limited pH range.

TABLE 5.39 Common Types of Synthetic Exchangers

Type of resin	Functional group	Drained density		Operating pH range	Maximum exchange capacity				Regeneration
		lb/ft ³	kg/m ³		meq/g	meq/ml	kgf/m ³	kgb	
Strong acid	—SO ₃ —H ⁺ , sulfonic acid	49–53	790–850	0–14	4.8	2.0	1544.2	43.7	Excess strong acid
Weak acid	—COO—H ⁺ , carboxylic acid	45	720	7–14	11	4.0	3473.50	98.3	Weak or strong acid
Strong base type I	—CH ₂ N(CH ₃) ₃ + OH ⁻ , quaternary ammonium	45	720	0–14	4.3	1.3	1003.5	(28.4)	Excess strong base
Strong base type II	—CH ₂ N(CH ₃) ₂ CH ₂ CH ₂ OH ⁺ OH ₂	45	720	0–14	3.4	1.4	1081.3	30.6	Excess strong base
Weak base	—N(CH ₃) ₂ H ⁺ OH ⁻	32	510	0–6	9	2.5	1929.3	54.6	Weak or strong base
Intermediate base	Mix of above two	43	690	0–14	8.8	2.7	2084.8	59.0	Strong base

For water softening and the removal of iron and manganese, a single reactor containing a strong acid resin is needed. The chemical reaction is



Adsorption

Adsorption has been used throughout history as a water treatment process. Carbon-based materials have been especially useful for removing impurities from drinking water (75). The ancient Hindus filtered their water with charcoal and, in the thirteenth century, carbon materials were used in a process to purify sugar solutions. One of these applications was begun in England in the mid-nineteenth century with the treatment of drinking waters for the removal of odors and tastes. From these beginnings, water and wastewater treatment with carbon-based materials have become widespread in municipal and industrial processes, including wineries and breweries, paper and pulp, pharmaceuticals, food, petroleum, and petrochemical applications.

Certain organic compounds in the water supply are resistant to conventional treatment and many others are toxic or nuisances (odor, taste, color forming), at low concentrations. Low concentrations are not readily removed by conventional treatment methods. Activated carbon has an affinity for various organics, and its use for organic contaminant removal from water supplies has been widely implemented.

The purpose of this section is to examine two particular kinds of adsorption processes. One is the use of powdered activated carbon (PAC) and the other is granular activated carbon (GAC) for treating drinking water.

Powdered Activated Carbon (PAC). PAC is finely ground loose carbon. To determine what degree of removal of dissolved organic material can be effected by adsorption, an isotherm test is usually run. The adsorption isotherm is the relationship between the amount of substance adsorbed and its concentration in the surrounding solution at equilibrium.

The adsorption isotherm consists of a curve plotted with residual concentration of the solute in solution on the x axis and the amount adsorbed per unit weight of adsorbent on the y axis, as illustrated in Figure 5.50. Thus, any point on the line gives the adsorption capacity or loading at a particular concentration in solution (76). With activated-carbon adsorption from dilute solutions, such as trihalomethanes in drinking water, we plot logarithmically according to the empirical Freundlich equation:

$$\frac{x}{M} = kC^{1/n} \quad (5.65)$$

where x = amount of substance adsorbed, calculated from original and equilibrium concentrations, C_0 and C_1

M = weight of activated carbon, k and n are constants; k is the intercept at $C_1 = 1$ ($\log C_1 = 0$) and $1/n$ is the slope of the line.

Equation (5.65) usually yields a straight line over the concentration ranges for trihalomethanes usually found in drinking water.

Data points required to construct the adsorption isotherm are usually obtained by applying varying doses of PAC to replicate samples of a solution of the solute in question (a trihalomethane) in water (77). These samples are held in suitable containers to prevent losses. After equilibrium is achieved, no further change in

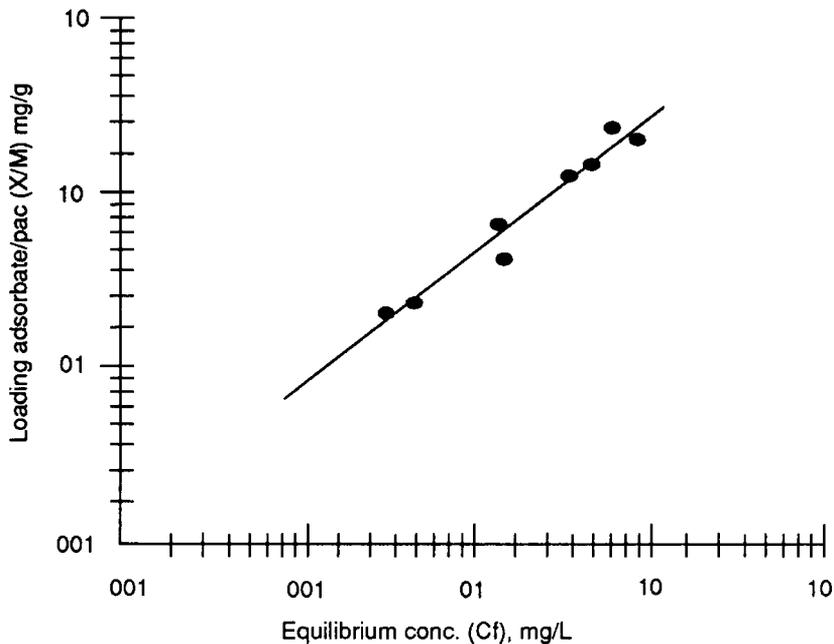


FIGURE 5.50 Adsorption isotherm for chloroform with F-300 (76).

solute concentration occurs. The activated carbon is then separated from the liquid sample, and residual dissolved solute concentrations are measured. The calculated values of X/M are plotted against respective C values on log/log coordinates.

Adsorption isotherms can be used to calculate the required PAC dose needed to reduce a contaminant concentration from some given influent concentration to a given target effluent concentration.

Granular Activated Carbon (GAC). GAC adsorption systems used in drinking water treatment typically use stationary beds with the liquid flowing downward through the adsorbent. Under these conditions adsorbed material accumulates at the top of the bed unit until the amount adsorbed at that point reaches a maximum, as shown in Figure 5.51. The maximum amount of a contaminant that can be adsorbed on activated carbon occurs when the adsorbed material is in equilibrium with the concentration of the contaminant in solution surrounding the adsorbent.

For any given concentration of material in the liquid phase, the “loading on” or “capacity of” the activated carbon at equilibrium can be determined from the appropriate adsorption isotherm, as discussed above. When the adsorbed material is in equilibrium with the influent concentration, the adsorbent is loaded to capacity and that portion of the bed is “exhausted.” In an ideal “plug flow” operation, the exhausted zone moves downward with time in the service unit until the entire adsorbent bed is exhausted. This is shown graphically in Figure 5.51, where the shaded area represents the exhausted activated carbon and the clear area is the remaining fresh activated carbon in the lower portions of the bed.

Figure 5.51 also represents the corresponding breakthrough profile of the contaminant in solution with time. The effluent concentration (C_{out}) remains near zero until the exhausted zone of the GAC reaches the bottom of the column and then abruptly increases to the influent concentration when the entire column is exhausted. Thus, the effluent concentration is equal to the influent concentration when the column is exhaust-

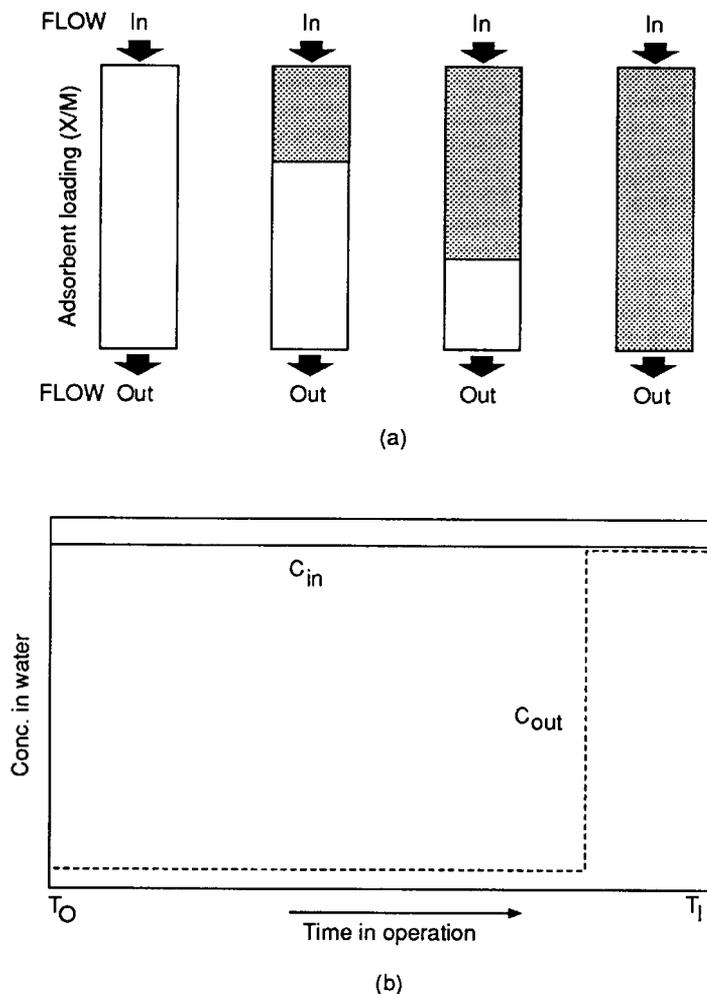


FIGURE 5.51 Ideal adsorption column performance. (a) Adsorbed contaminant profile; (b) effluent contaminant profile (76).

ed or at equilibrium and, ideally, therefore, the time to exhaustion of an adsorber can be predicted from isotherm test information.

For a given situation, the time to reach exhaustion can be estimated by knowing the influent contaminant concentration, the approach velocity (to calculate mass loading), the adsorber bed depth, the density of the adsorbent, and the equilibrium loading from an adsorption isotherm. This estimation assumes a constant influent concentration of contaminant and neglects the influence of competitive adsorption. For example, assume an influent chloroform concentration of $100 \mu\text{g/L}$, a 2 gpm/ft^2 ($5 \text{ m}^3/\text{h/m}^2$) approach velocity, a 3-ft (0.9-in) bed depth, and an adsorbent density of 30 lb/ft^3 (490 kg/m^3). At exhaustion ($C_{out} = C_1$), the loading on or capacity of the adsorbent would be 0.49 mg/g or g/kg .

For a 1-ft^2 (0.09-m^2) cross section of this bed, $2 \text{ gal/min} \times 1440 \text{ min/day} \times 3.78 \text{ L/gal} = 10,886 \text{ L/day}$

passes through. Multiplying by the chloroform concentration, $10,886 \text{ L/day} \times 100 \text{ } \mu\text{g/L}$, yields 1.09 g/day of chloroform applied to the 1-ft^2 cross section. This cross section, 3 ft (0.9 m) deep, contains 3 ft^3 (0.08 m^3) of medium. Converting this to weight yields 90 lb (41 kg) of adsorbent. Therefore, the days required to “load” this adsorbent to equilibrium is

$$\text{Days of operation} = \frac{\text{Total weight of adsorbent} \times \text{equilibrium loading}}{\text{daily load}} \quad (5.66)$$

$$\begin{aligned} \text{Days of operation} &= (41 \text{ kg})(0.49 \text{ g/kg})/1.09 \text{ g/day} \\ &= 18.4 \text{ days} \end{aligned} \quad (5.67)$$

Thus, an estimated 18.4 days would be required to exhaust this bed. The data in Table 5.40 are examples of these calculations for a GAC adsorber with a 10-min empty-bed contact time (EBCT) and an approach velocity of 2 gpm/ft^2 ($5 \text{ m}^3/\text{m}^2\text{h}$). Because, in this example, chloroform is present in the highest concentration and is the compound most poorly adsorbed, the “service time to exhaustion” is the shortest for that compound.

When the other two variables are constant, the time in service to exhaustion is directly proportional to EBCT. For chloroform, a twentyfold increase in influent concentration resulted in a 50% decline in the service time to exhaustion, whereas the same increase in the bromoform influent concentration caused a 75% decline in the service time to exhaustion. Finally, the time in service to exhaustion for bromoform was 24, 16, and 12 times longer than that for chloroform at the influent concentrations of $5 \text{ } \mu\text{g/L}$, $25 \text{ } \mu\text{g/L}$, and $100 \text{ } \mu\text{g/L}$, respectively.

Although this approach may be useful to determine the service life of an ideal adsorber, various parameters affecting the rate of mass transfer within the adsorber (kinetic aspects of adsorption) severely limit its application and make necessary the use of pilot column studies for accurate prediction of performance.

Schematically illustrated in Figure 5.52 is a diagram showing the concentration of adsorbed species on the surface of the adsorbent (X/M) with bed depth. Under operational conditions, adsorbed material accumulates at the top of the bed until the amount adsorbed is in equilibrium with the influent contaminant concentration. At this time the adsorbent is loaded to capacity and that portion of the bed is exhausted. Below that zone is a second zone where dynamic adsorption is taking place, i.e., the contaminant is being transferred from the liquid solute to the adsorbed phase. This zone is called the *mass transfer zone*, and its depth (deviation from ideal plug flow) is controlled by many factors, depending on the contaminant being adsorbed, characteristics of the adsorbent, hydraulic factors, and others (76). The depth of the mass transfer zone is a

TABLE 5.40 Theoretical Time in Service until Exhaustion of Trihalomethane Adsorption

Constituent	Influent concentration, $\mu\text{g/L}$	Loading at equilibrium, mg/g	Time* to exhaustion,† days
CHCl_3	75	0.35	15
CHBrCl_2	50	1.3	87
CHBr_2Cl	25	1.4	189
CHBr_3	10	1.8	606

*10 min EBCT. approach velocity = 2 gpm/ft^2 ($5 \text{ m}^3/\text{h/m}^2$); GAC density = 490 kg/m^3 (30 lb/ft^3).

†Exhaustion for total trihalomethanes will not occur until the species with the best adsorption characteristic reaches exhaustion.

Source: From Ref. 76.

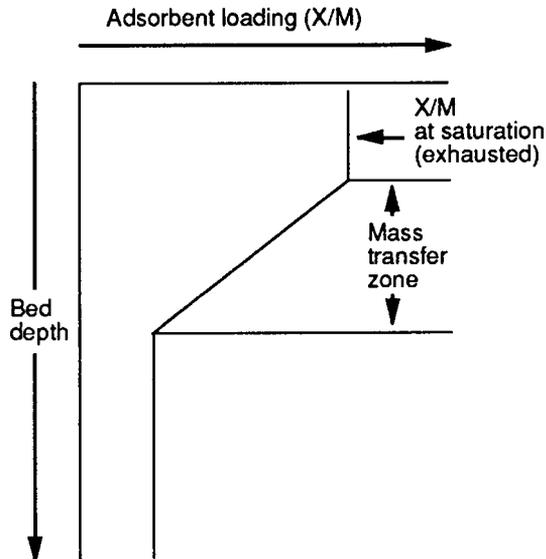


FIGURE 5.52 Mass transfer zone in a GAC adsorber (76).

measure of physical or chemical resistance to mass transfer. Once formed, the mass transfer zone moves down through the adsorbent bed until it reaches the bottom, whereupon the effluent concentration of the contaminant in the aqueous phase begins to rise. Figure 5.53 shows the concentration gradient of adsorbed material (X/M) in an adsorber as the mass transfer zone moves down the column with time. As the mass transfer zone reaches the bottom of the column, “breakthrough” of the contaminant occurs as noted by a detectable increase in effluent concentration. When the adsorber is operated to exhaustion (at equilibrium; $C_{in} = C_{out}$), the breakthrough profile (plot of effluent concentration with time) takes on a classical S shape—a shape controlled by the shape and length of the mass transfer zone.

Thus, any estimate from isotherm data of a dynamic adsorber service time to exhaustion is at least subject to error caused by the differences in mass loading calculations based on ideal plug flow and typical column performance (Figure 5.53) as represented by areas A and B in the effluent concentration profile. Clearly, undesirable concentrations of contaminant may appear long before exhaustion occurs in practice.

For groundwater supplies with excessive levels of nitrates, above 10 mg/L, ion exchange is an effective drinking water treatment process. Other methods, such as reverse osmosis, may be used, but the selective nature of ion exchange makes the process preferable when other constituents are at acceptable levels. Continuous systems are the most common application. In practice, the nitrates may be removed from a sidestream, and the treated water is then blended with the raw water to achieve acceptable levels.

Reverse Osmosis

Osmosis is the process by which water moves across a semipermeable membrane from a low concentration of solute to a high concentration of solute to satisfy the pressure differences caused by the solute. Osmosis and reverse osmosis depend on the presence of a barrier or membrane that is selective so that the solvent of a solution can pass through the membrane while other components of the solution or the solutes cannot.

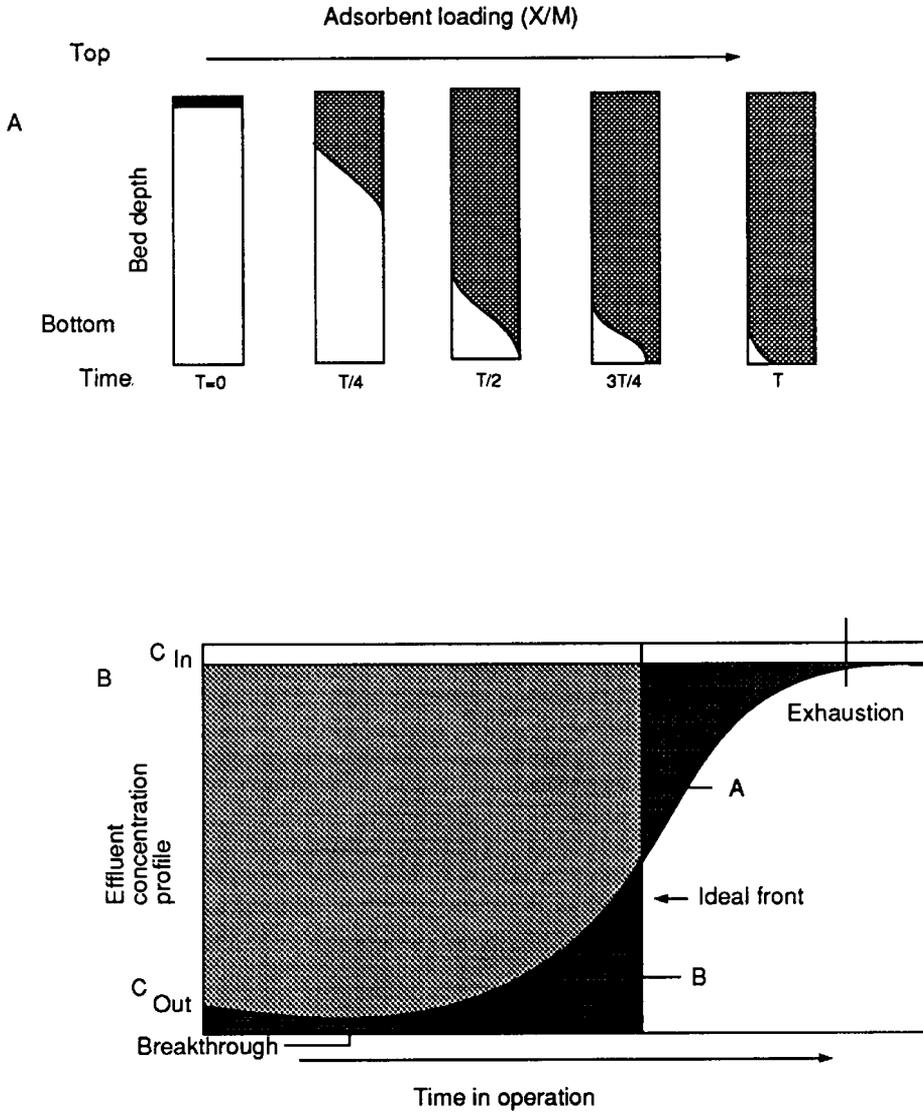


FIGURE 5.53 Typical adsorption column performance (76).

Such a membrane is described as semipermeable. The osmotic pressure is the pressure required to stop the flow of solvent through a semipermeable membrane separating two solutions of different concentrations. To separate water from dissolved solids by reverse osmosis, the applied pressure must be greater than the osmotic pressure.

Two basic simplified equations are used to describe the flow of water and solutes across the membrane in reverse osmosis. The product water or permeate flow through a semipermeable membrane can be expressed as

$$F_w = (A)(\Delta P - \Delta \pi) \quad (5.68)$$

where F_w = water flux, gpd/ft² (cm³/cm² · s)

A = water transport coefficient, gpd/ft²·atm (cm³/cm²·s·atm)

ΔP = pressure differential applied across the membrane, atm

$\Delta \pi$ = osmotic pressure differential across the membrane, atm

The flow of water is thus primarily pressure-dependent. There is some transport of solute across the semi-permeable membrane and this is expressed as

$$F_s = B \Delta C \quad (5.69)$$

where F_s = salt (solute) flux, gpd/ft² (g/cm²·s)

B = salt (solute transport coefficient), ft/2 (cm/s)

$\Delta C = C_1 - C_2$ = concentration gradient across the membrane, lb/ft³ (g/cm³)

The transport of solute is primarily dependent on the pressure differential.

In water treatment, ion exchange has proven effective in removing organics and metal salts. Water feed to a reverse osmosis unit must be preprocessed to remove suspended solids. Once this is accomplished, the feed stream is applied under high pressure to the membrane from which product and concentrate are continuously withdrawn. Two basic types of membranes are used for reverse osmosis: (1) natural (cellulose acetate or nitrate) and (2) synthetic (polyamide (nylon) fibers).

Reverse osmosis with synthetic membranes is used widely for desalination. In this application, saline feedwater is introduced to a module around hollow fibers (outside diameter is approximately 50 mm, inside diameter is approximately 25 mm) looped at one end and open at the other. The open ends are enclosed in a plastic seal to prevent the mixing of concentrate and product water. Water passes through the fiber walls and is collected and the salts are rejected (78).

Cellulose acetate, the most widely used membrane for reverse osmosis, is typically 100 mm thick; however, its rejecting surface is only 0.2 mm thick. The majority of the membrane is spongy and porous, composed primarily of water. Limitations of cellulose acetate membranes include: (1) feedwater pH must be acidic to prevent membrane hydrolysis; (2) microbial populations must be limited to prevent degradation of the membrane; (3) temperature must not exceed 23.33 to 26.67°C (75 to 80°F) or membrane life will markedly deteriorate; (4) projected membrane life is 2 to 3 years in continuous use (79). Cellulose acetate membranes have been incorporated into three different designs: (1) plate and frame, (2) large tube, and (3) spiral wound.

The plate-and-frame design acts as a filter press. The simple rugged equipment which allows single-sheet membrane replacement is one of the main advantages of the plate-and-frame membrane configuration. The disadvantages of the design lie in the difficulty encountered in determining flow patterns, manual labor required for membrane replacement, and high initial equipment cost. Currently, this design is applied in water treatment reverse osmosis operations.

The large-tube design utilizes porous walled tubes inside which the membrane is cast. The tube serves to protect and support the membrane. This configuration yields a well-defined flow regime and minimizes pre-filtration requirements. However, large tubes have several major disadvantages, including the seal between waste and product chamber is tenacious, a large number of tubes are required for adequate surface area, and membranes cannot be replaced within a tube.

The spiral-wound reverse osmosis unit consists of two membranes separated by porous mesh, sealed on three sides, with the fourth connected to perforations in a tube and glued in place (Figure 5.54). The membrane blanket is wrapped around the tube with a screen separating its layers (Figure 5.55). The entire module is inserted into a larger tube into which the raw water is injected (Figure 5.56). Spiral winding of the membrane blanket allows high packing density, controlled spacing, and ease of replacement. The spiral-wound

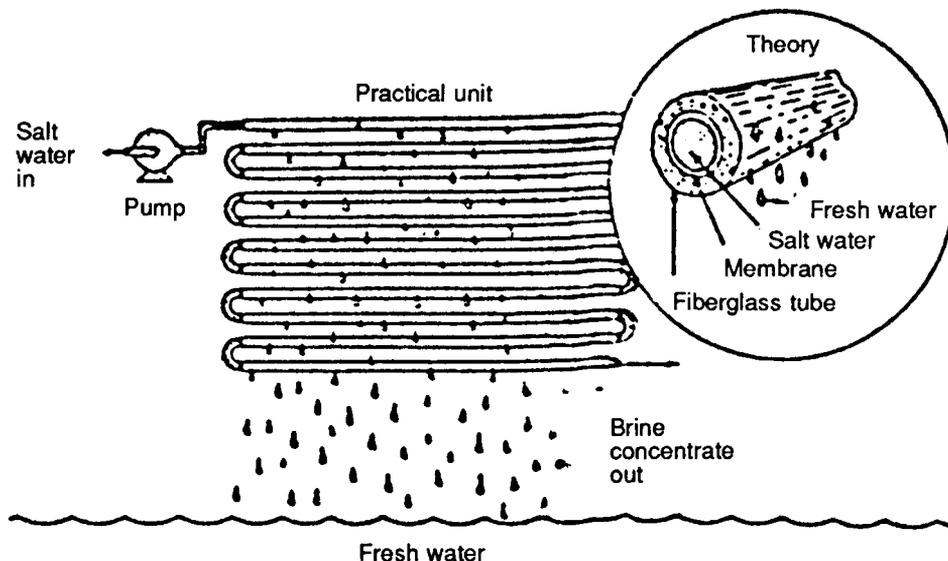


FIGURE 5.54 Simplified illustration of tubular reverse osmosis (80).

tubes require total replacement of the module when failure occurs. Figure 5.57 is a schematic illustration of a reverse osmosis system.

Conversion rates range up to approximately 75% of feedwater volume in reverse osmosis operations. Typically, conversions of greater than 75% will require additional preconditioning of the raw water. Normal pretreatment of water for reverse osmosis should include removal of excess turbidity and suspended solids, adjustment and control of temperature, inhibition or control of the formation of compounds which when precipitated will inhibit water passages or coat membrane, disinfection to prevent slime growths or contamination of the equipment, and removal of all oil and similar organics (79).

Reverse osmosis, when used for water treatment, has been found to improve tastes, color, cooking and washing uses, and consumer attitudes. Reverse osmosis acts as a molecular filter to remove 90 to 95% of total dissolved inorganic solids, 95 to 97% of most dissolved organics, and 98% of biological and colloidal contaminants.

Aeration

Aeration has a long tradition in water treatment (82–87), including such applications as the addition of oxygen or carbon dioxide gases to water and the removal of dissolved hydrogen sulfide gas. During the past decade, additional applications of aeration have been developed, including air stripping for removal of volatile organic contaminants from drinking water supplies (88). Preliminary cost analyses of the organic contaminants suggest that this process may be considerably cheaper than alternative processes, including granular activated carbon (GAC) or adsorbent resins (89). Thus, this process merits a detailed analysis of its technical and economic feasibility for control of volatile organic contaminants in drinking water supplies.

The design principles for gas absorption and air stripping have been extensively developed in the field of chemical engineering (89–92). In general, chemical engineers design systems to remove dissolved gases less than 100 mg/L. Consequently, the procedures developed for the design of absorption or stripping systems can be simplified and applied to water treatment applications.

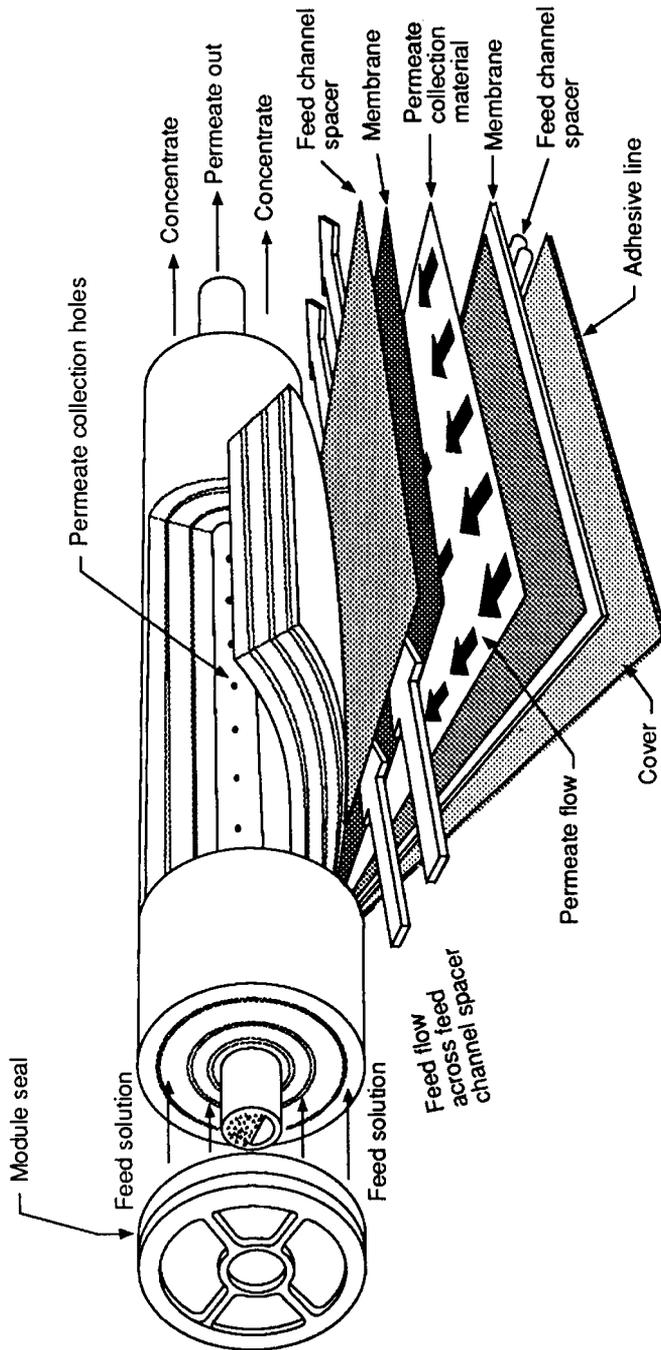


FIGURE 5.55 Spiral wound membrane models (87).

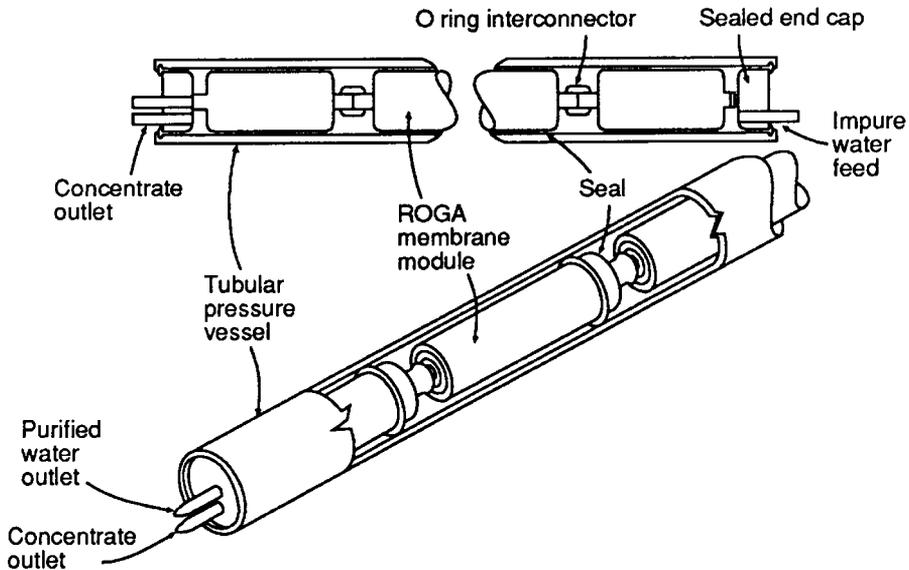


FIGURE 5.56 Permeate collecting channel of the spiral wound membrane module (81).

This section presents a discussion of the basic principles of the design of airstripping systems as applied to the removal of volatile trace organics from dilute solutions. Although air stripping can be conducted in a number of alternative equipment designs, this discussion will focus on countercurrent packed towers, which offer several process advantages.

The relative amounts of the volatile substances contained in air and water and their concentrations in water with respect to their saturation value are factors that control the rate at which the interchange takes place. Substances that occur in water in amounts less than or in excess of their saturation values are changed in concentration by aeration, with the saturation value being the limit of change for both conditions.

The substances involved in the reaction must be volatile. Oxygen, carbon dioxide, and hydrogen sulfide are volatile, and their concentrations in water are readily affected by the process. Many of the taste- and

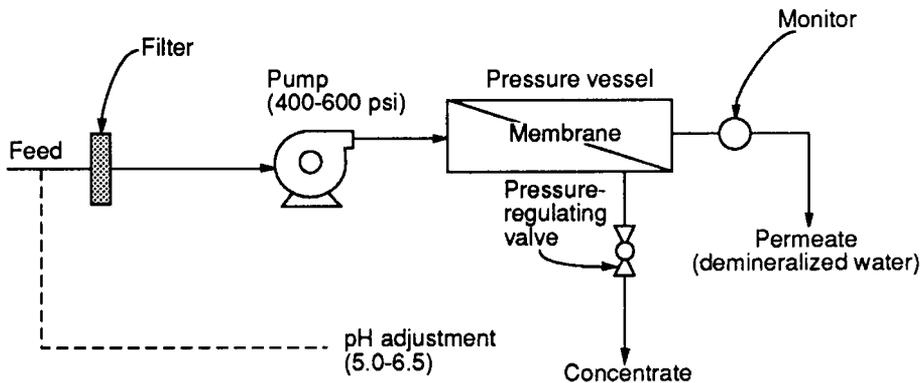


FIGURE 5.57 Schematic of reverse osmosis system (81).

odor-producing compounds occurring in water as the result of algal growths or industrial wastes are not volatile at the temperatures encountered in natural waters and cannot be removed by aeration. Since higher temperatures increase the volatility of the compounds and decrease their saturation values, aeration for the removal of volatile materials is more effective in warm than in cold waters. Similarly, the removal by aeration of some gases, such as H_2S , CO_2 , and NH_3 , is strongly dependent upon the pH of the water. Low pH values favor the removal of these compounds by aeration.

Design Considerations. The interchange of the substances from water to air or from air to water takes place at the air–water surface. The rate at which the interchange occurs is the result of the relative concentration of the substance at this surface and the rapidity with which new surfaces are formed and exposed.

Since molecular diffusion of gases through liquids occurs at low rates which exert little effect on the efficiency of the process, all types of aerators, to be efficient, must develop and continually change large surface areas where fresh contacts are made, and through which the interchange may take place.

The exchange can be described by these formulas (92, 93):

Gas absorption:

$$C_t = S - (S - C_0)10^{-k(A/V)t} \quad (5.70)$$

Gas release:

$$C_t = S + (C_0 - S)10^{-k(A/V)t} \quad (5.71)$$

where C_t = concentration of gas in water
 S = saturation concentration
 C_0 = initial concentration of gas in water
 A/V = air-to-water ratio, vol/vol
 k = rate constant
 t = time period

These formulas and the differential equations from which they are derived indicate that:

1. At any instant, the rate of gas transfer is directly proportional to the difference between the gas saturation concentration S and the actual concentration C_t in the water.
2. The rate of gas transfer is directly proportional to the ratio of the exposed area to the volume of water, A/V .
3. The rate of gas transfer is directly proportional to the gas transfer coefficient k , which in turn is dependent on the diffusivity of the gas in question and the film resistance.
4. The total amount of gas transfer is greater as the time of aeration increases.
5. The percentage change in gas saturation deficit $S - C_0$ or surplus $C_t - S$ for any given time period t is constant based on the deficit or surplus at the beginning of the time period.
6. Temperature and pressure are important factors because they influence gas solubility S . Temperature also influences diffusivity and film resistance and hence the value of k .

There are a number of papers reporting on the general subject of aeration that contain additional information that may be of interest (93–97).

The time factor and the ratio of surface area to the volume of water, A/V , are important factors that must be considered in the design of an aerator. The partial pressure or the concentration in the air of the gas to be removed must also be considered and provision made for ventilation in installations where the unit is to be housed.

As corrosion of the structure and equipment represents a major consideration in the design of aerators,

materials that will resist the corrosive action of the water must be used. Concrete, aluminum, asbestos cement, copper-bearing steel, stainless steel, and creosoted lumber have been used in the construction of aerators. Bronze and cast iron are used in nozzles and a number of patented devices. Ordinary steel should be used only in those locations that are readily accessible for painting at frequent intervals.

Because slime and algal growths may be troublesome in waterfall aerators of the cascade and tray types, especially if the units are located outdoors and subject to sunlight, treatment by chlorine or copper sulfate may be necessary.

Physical Chemistry of Aeration. According to Dalton's law, the total pressure of an ideal gas mixture is the sum of the partial pressures of all of its components. Thus the partial pressure of a given contaminant in air is the product of the total pressure and the gas phase mole fraction of the contaminant in the air:

$$P_a = Y_a P_t \quad (5.72)$$

where Y_a = mole fraction of contaminant a in gas phase

P_t = total pressure

P_a = partial pressure of contaminant a in air

When solutions are very dilute, it is often observed that the equilibrium partial pressure of the contaminant in the air is proportional to the solution concentration of the contaminant. This phenomenon is known as Henry's law and can be mathematically represented as follows:

$$P_a = H_a X_a^* \quad (5.73)$$

where H_a = Henry's constant of contaminant a , atm

X_a^* = solution concentration of contaminant a

Thus, at equilibrium, the concentration and mole fraction of contaminant a in the liquid and gases phases are related by Eq. 5.72 and 5.73 to give

$$Y_a = X_a^* \left(\frac{H_a}{P_a} \right) \quad (5.74)$$

The larger the Henry's constant, the greater will be the equilibrium of contaminant a in the air. Thus, contaminants having larger Henry's constants are more easily removed by air stripping.

Most gases and vapors of interest follow Henry's law quite satisfactorily in the range of concentrations experienced in domestic water treatment. Table 5.41 shows Henry's constants for 25 selected compounds at 20°C. Henry's constants are strongly influenced by temperature. The relationship of the Henry's constant to temperature can be modeled by a van't Hoff-type relation. If the enthalpy change due to dissolution of the contaminant in water is considered independent of temperature, the relation takes the following form:

$$\log_{10} H = \left(\frac{\Delta H^\circ}{RT} \right) + K \quad (5.75)$$

where R = universal gas constant, 1.987 kcal/kmol·K

T = absolute temperature, K

ΔH° = change in enthalpy due to dissolution of component a in water, kcal/kmol

K = constant

TABLE 5.41 Henry's Constant for Selected Compounds

Compound	Henry's constant (at 20°C), atm
Vinyl chloride*	3.55×10^5
Oxygen	4.3×10^4
Nitrogen	8.6×10^4
Methane	3.8×10^4
Ozone	3.9×10^3
Toxaphene*	3.5×10^3
Carbon dioxide	1.51×10^3
Carbon tetrachloride*	1.29×10^3
Tetrachlorethylene*	1.1×10^3
Trichloroethylene*	5.5×10^2
Hydrogen sulfide	5.15×10^2
Chloromethane	4.8×10^2
1,1,1-Trichloroethane*	4.0×10^2
Toluene*	$3.4 \times 10^2(25^\circ\text{C})$
1,2,4-Trimethylbenzene*	$3.3 \times 10^2(25^\circ\text{C})$
Benzene*	2.4×10^2
1,4-Dichlorobenzene*	1.9×10^2
Chloroform*	1.7×10^2
1,2-Dichloroethane*	61
1,1,1-Trichloroethane*	43
Sulfide dioxide	38
Bromoform*	35
Ammonia	0.76
Pentachlorophenol*	0.12
Dieldrin*	0.0094

*Computed from water solubility data and partial pressure of pure liquid at specified temperature.

Table 5.42 shows values of H° and K for 20 compounds of interest in water treatment. The values show that Henry's constant for most volatile hydrocarbons increases about threefold for every 10°C temperature rise.

Air stripping can successfully lower the concentration of volatile organics in drinking water. Among the several factors that influence aeration effectiveness, perhaps the three most important are

- The molecular properties of the contaminant(s)
- The ratio of air to water applied to the aerator
- The ability to contact a large amount of air with a thin film of water to facilitate transfer of the contaminant into the gaseous phase

The first factor is best represented by Henry's law constant, which describes the concentration of the contaminant in the gas and water phases at equilibrium. Experience has shown the efficiency of aeration varies directly with this parameter (98). For example, 1,2-dichloroethane is more difficult to strip from water than 1,1,1-trichloroethane, which in turn is more difficult to strip than 1,1-dichloroethylene. The other factors listed above are dependent on the type of aerator.

Aeration Systems. In 1984, an estimated 10 to 12 United States utilities were aerating to strip volatile organics from their drinking water (Figure 5.58). The three principal types of aerators are redwood slat aera-

TABLE 5.42 Temperature Dependence of Henry's Constant

Compound	Formula	H° , kcal/kmol $\times 10^{-3}$	K
Benzene	C_6H_6	3.68	8.68
Chloroform	$CHCl_3$	4.00	9.10
Carbon tetrachloride	CCl_4	4.05	10.06
Methane	CH_4	1.54	7.22
Ammonia	NH_3	3.754	6.31
Chloromethane	CH_3Cl	2.48	6.93
1,2-Dichloromethane	$C_2H_4Cl_2$	3.62	7.92
1,1,1-Trichloroethane	$C_2H_3Cl_3$	3.96	9.39
1,1-Dichloroethane	$C_2H_4Cl_2$	3.78	8.87
Trichloroethylene	C_2HCl_3	3.41	8.59
Tetrachloroethylene	C_2Cl_4	4.29	0.38
Carbon dioxide	CO_2	2.07	6.73
Hydrogen sulfide	H_2S	0.020	5.84
Chlorine	Cl_2	1.74	5.75
Chlorine dioxide	ClO_2	2.93	6.76
Sulfur dioxide	SO_2	2.40	5.68
Difluorochloromethane	CHF_2Cl	2.92	8.18
Oxygen	O_2	1.45	7.11
Nitrogen	N_2	1.12	6.85
Ozone	O_3	2.52	8.05

tors, packed towers, and spray towers. Two additional methods, diffused-air sparging and air-lift pumping, are being evaluated for potential full-scale application. Schematic drawings of these aeration devices are shown in Figures 5.59 and 5.60.

Diffused Air Aerators. Aeration equipment used in present practice employs one of two methods to control time and A/V . One method exposes water films to the air and the other introduces air in the form of small bubbles in the water. The first is generally known as the *waterfall type* and the second as the *diffused-air type*.

Because waterfall aerators, as the name implies, require that the water be dropped in the process, their use results in the loss of a considerable amount of head. On the other hand, although little or no loss of head is required for the diffused-air units, they require the expenditure of energy in compressing the air and forcing it through small orifices into the water at some distance below the surface.

Difficulties in the operation of waterfall aerators can be expected from freezing during the winter in extreme climates unless they are housed. If an aerator is enclosed, careful consideration should be given to ventilation in the interest of efficiency and safety.

It has been demonstrated that good ventilation is necessary for the effective removal of gases from solution (100). Based on data from pilot and full-scale operations, the media in multiple-aerators should be so arranged that fresh air continuously passes through the falling water and over the media. It is impossible to secure consistent results with an aerator depending on natural ventilation alone. Better results were obtained when a breeze was blowing than when a calm prevailed.

Ventilation for an enclosed aerator is also of importance from a safety standpoint in the case of removal of carbon dioxide, methane, and hydrogen sulfide. The first named is an asphyxiant, the second creates an explosion hazard, and the third is highly poisonous.

Packed Tower Aerators. This device consists of a column, 3.3 to 9.8 ft (1 to 3 m) in diameter and 16.4 to 32.8 ft (5 to 10 m) in height, that is filled with packing material. This packing can be glass, ceramic, or plas-

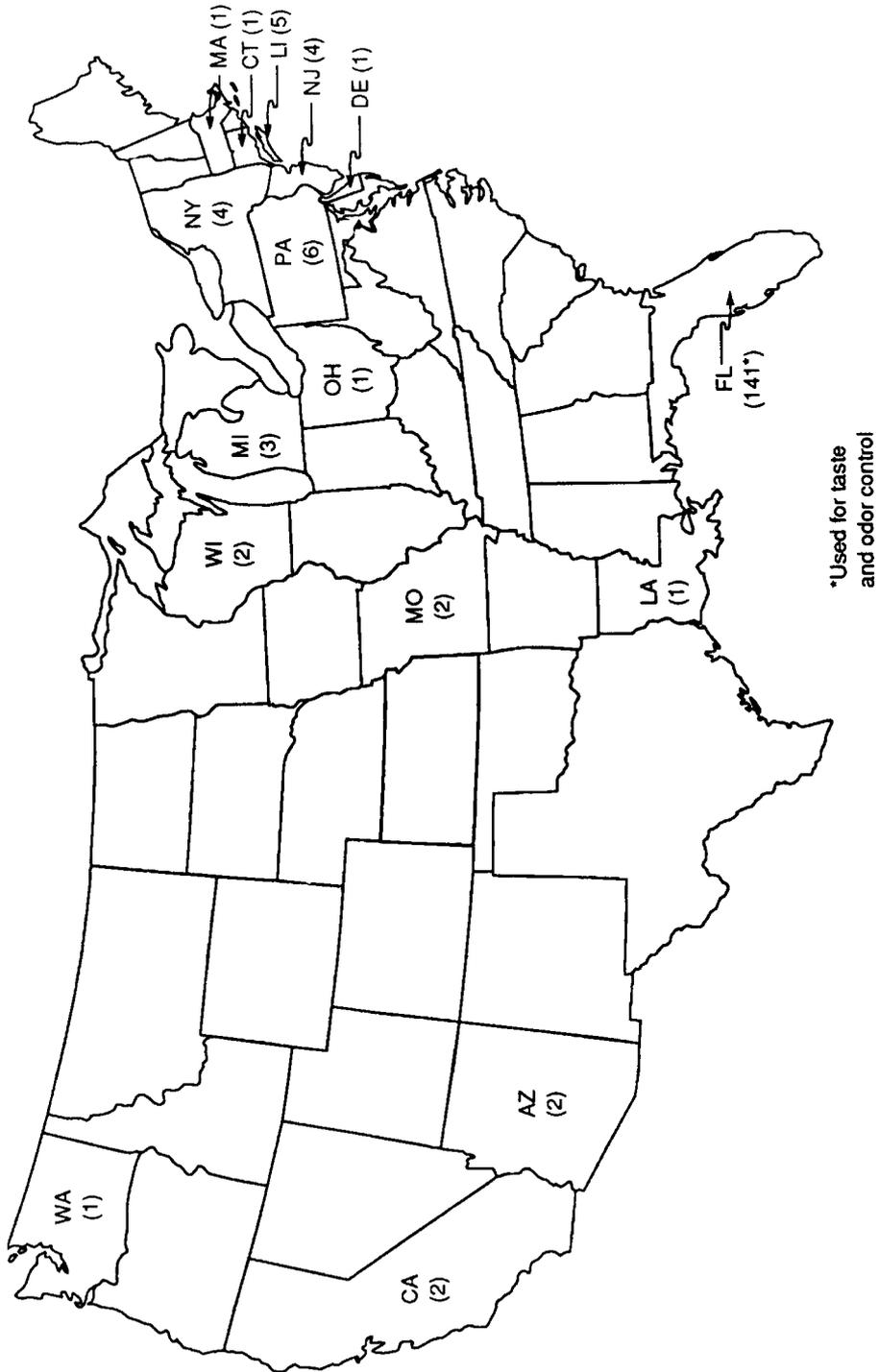


FIGURE 5.58 Aeration systems for the removal of volatile organic contaminants from groundwater (99).
*Used for taste and odor control

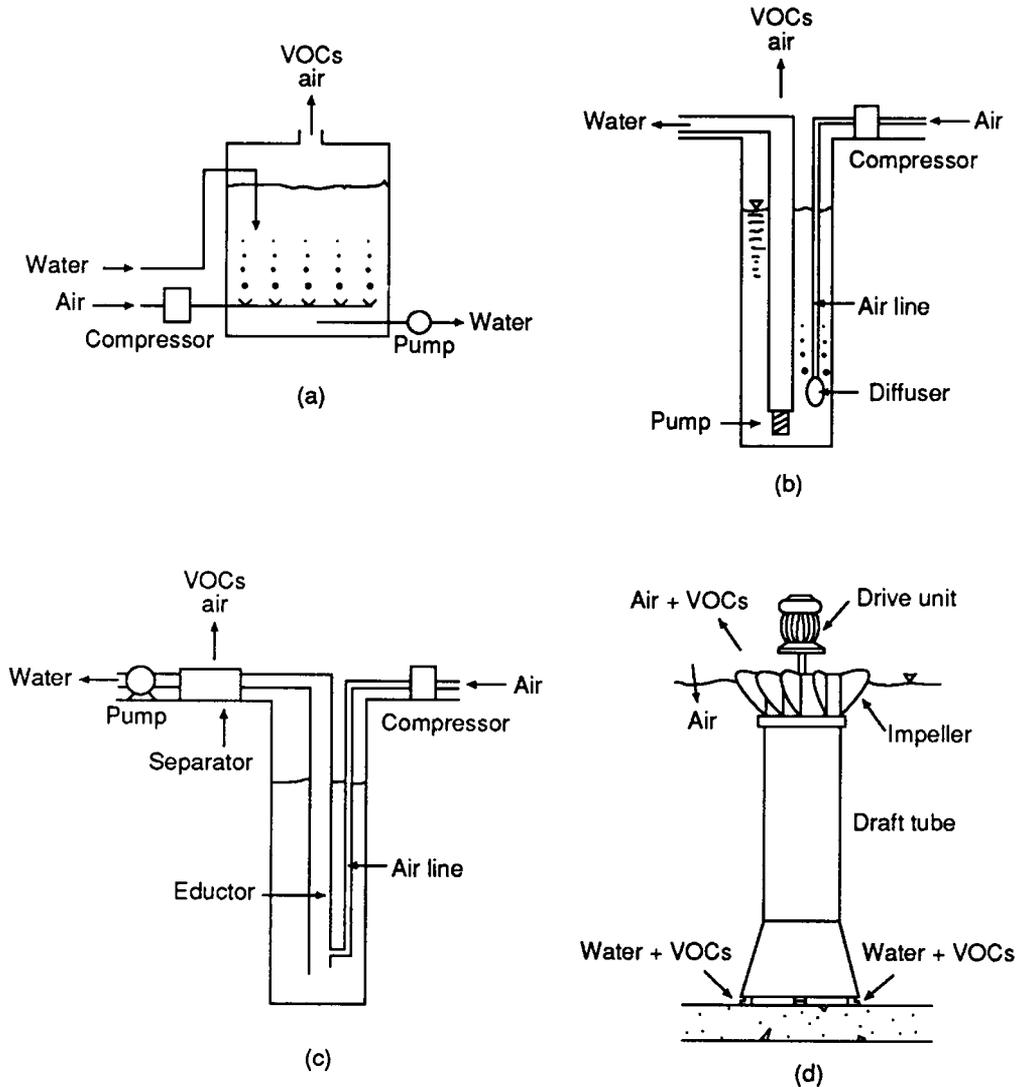


FIGURE 5.59 Aeration devices that put air through water. (a) Diffused-air aerator; (b) in-ground aeration; (c) airlift pump; (d) mechanical surface aerator (99).

tic and is available in numerous geometrical shapes to facilitate the transfer of mass from the aqueous phase to the gas phase. The inside wall of the aeration column has several “redistributors” that force the water over the packing and thus prevent the water from simply running down the walls. The diagram in Figure 5.60 shows a fan at the top. This is called an *induced-draft* packed tower. Perhaps more common is the *forced-draft* packed tower, where a blower positioned at the bottom of the tower forces air up through the packing. Two excellent references are available that describe these devices and show how they can be designed to remove 98% or higher of the volatile organic compounds found in groundwater (101, 102).

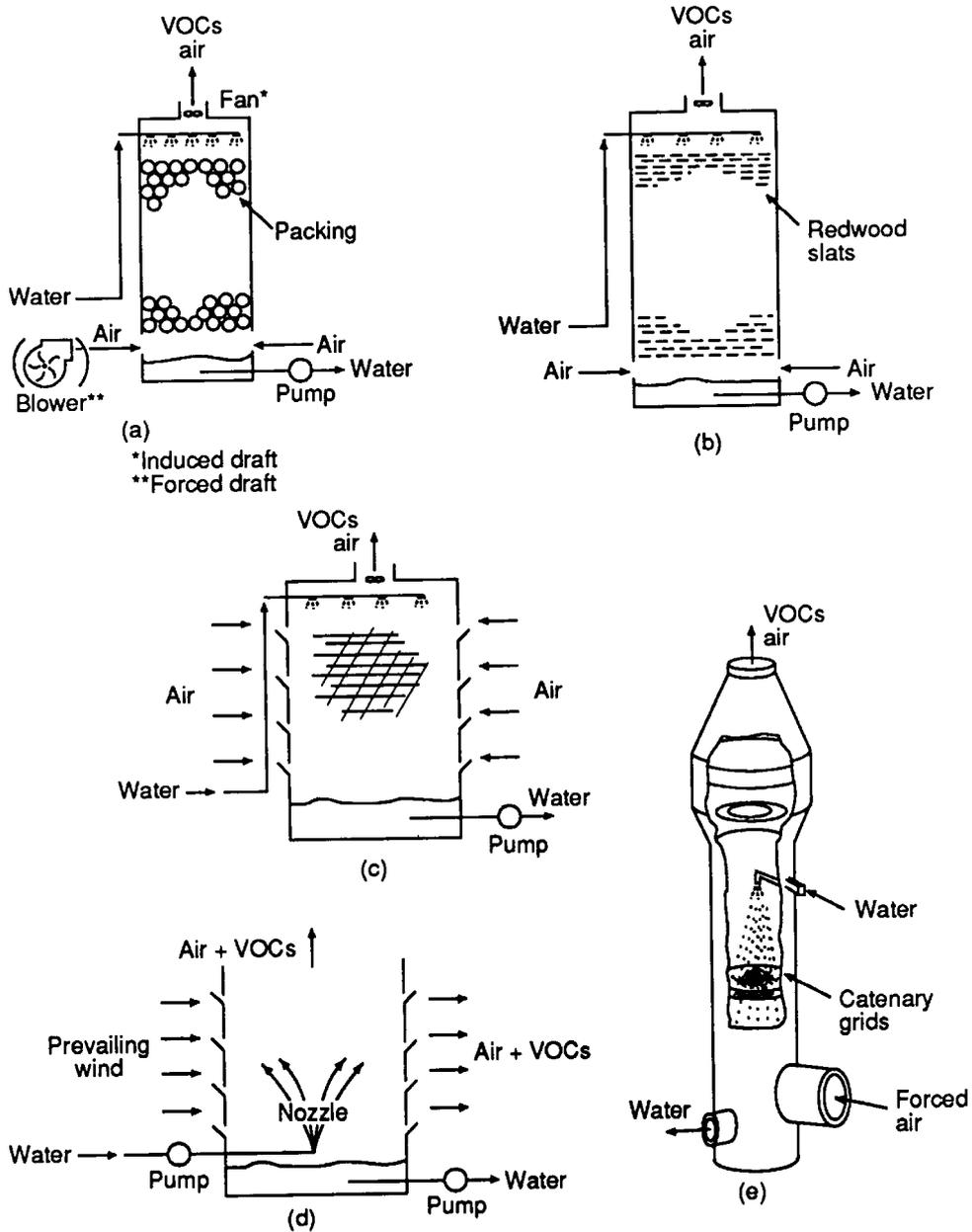


FIGURE 5.60 Aeration devices that put water through air. (a) Packed-tower aerator; (b) redwood slat aerator; (c) cooling tower; (d) spray tower; (e) catenary grid scrubber (99).

Redwood Slat Aerators. These devices are fundamentally cooling towers, but they have been used to remove iron and manganese or hydrogen sulfide from drinking water. They also can remove a high percentage (20 to 90%) of volatile organics (103).

Spray Aerators. Spray aerators are being used in the state of Florida at a drinking water source containing trichloroethylene and related industrial solvents. In that installation, the water is pumped through four aerators in series. The removal efficiency for this operation is reported to be greater than 99% (104).

Early in the twentieth century, air-lift pumps were commonly used in the United States. These are simple devices with only two pipes in the well. One pipe introduces compressed air into the open bottom of the other pipe, called an eductor. In the eductor, air mixes with water. The mixture, being of lighter density than the surrounding water, will rise to the surface. The air is separated before the water is pumped into the distribution system. Because of poor pumping efficiency, typically 35%, they fell into disuse with the introduction of submersible pumps.

Several observations can be made about the aeration process.

- Because mixtures of solvents exist in contaminated water and even though the effectiveness of the process varies for each solvent, aerating to remove one specific contaminant will also reduce concentrations of the others.
- The percent removal of a contaminant is very constant for a given set of conditions and does not vary with the influent concentration.
- The temperature of groundwater is consistently around 55.4°F (13°C).
- Even when subjected to extremely cold air during the winter, the temperature of the aerated water drops but a fraction of a degree, thus these units can be operated in cold weather climates. Some housing, however, is probably prudent to protect the device from freezing in the event of a pump failure.

Questions have been raised concerning the secondary effects of aeration. These include potential air quality problems created by exhaust gases from the aerator, and the potential for water quality deterioration from airborne particulates, oxidized inorganics, instability resulting in corrosion, and biological growth in the aeration device.

Package Treatment Plants

Package treatment technology offers an alternative to in-ground conventional treatment technology (see Table 5.43). Package plants are treatment units that are assembled in a factory, skid mounted, and transported to the site. The treatment processes utilized in package plants are essentially variations of coagulation and filtration treatment trains that treat up to 6 mgd (23,000 m³/day). The difference between these and custom-designed plants is that the package plants arrive on-site virtually ready to operate and built to minimize the day-to-day attention required to operate the equipment.

Several hundred filtration package plants have been installed nationwide, mostly to remove turbidity and bacteria from water with low to moderate levels of turbidity (150). The maximum contaminant levels and treatment technique requirements promulgated under the SDWA are changing the focus of treating water from control of turbidity to more sophisticated goals, such as the control of disinfection by-products (total trihalomethanes).

New designs for package plants will no doubt be influenced by the desire to remove total or dissolved organic carbon and will incorporate the use of alternative disinfectants, such as ozone. Highly variable influent water quality requires operator attention and tends to negate the package plant advantages of low cost and automation. In the United States, many of the state environmental agencies are reluctant to endorse them completely (151).

Although other treatment technologies are amenable to a “package plant” type of operation, this section

TABLE 5.43 Small System Treatment Technology Overview

Technology	Advantages	Disadvantages
<i>Filtration</i>		
Slow sand	Operational simplicity and reliability Low cost Ability to achieve greater than 99.9% <i>Giardia</i> cyst removal	Not suitable for water with high turbidity Requires large land areas
Diatomaceous earth	Compact size Simplicity of operation Excellent cyst and turbidity removal	Most suitable for raw water with low bacterial counts and low turbidity (less than 10 NTU) Requires coagulant and filter aids for effective virus removal Potential difficulty in maintaining complete and uniform thickness of diatomaceous earth on filter septum
Reverse osmosis Membranes	Extremely compact Automated	Little information available to establish design criteria or operating parameters Most suitable for raw water with less than 1 NTU; usually must be preceded by high levels of pretreatment Easily clogged with colloids and algae Short filter runs Concerns about membranes failure Complex repairs of automated controls High percent of water lost in backflushing
<i>Disinfectant</i>		
Chlorine	Very effective; has a proven history of protection against waterborne disease. Widely used. Variety of possible application points. Inexpensive. Appropriate as both primary and secondary disinfectant.	Potential for harmful halogenated by-products under certain conditions
Ozone	Very effective. No THMs formed.	Relatively high cost. More complex operations because it must be generated on-site. Requires a secondary disinfectant Other by-products such as bromates, which may be regulated at very low levels.
Ultraviolet radiation	Very effective for viruses and bacteria. Readily available. No known harmful disinfectant residuals.	Requires a secondary disinfectant Simple operation and maintenance for high quality waters.
<i>Organic contaminant removal</i>		
Granular activated carbon	Effective for a broad spectrum of organics	Spent carbon disposal
Packed tower aeration	Effective for volatile compounds	Potential for air emissions issues
Diffusional aeration	Effective for volatile compounds/ radionuclides	Clogging air missions, variable removal efficiencies
Advanced oxidation	Very effective	By-products
Reverse osmosis	Broad spectrum removed	Variable removal efficiencies, wastewater disposal

(continues)

TABLE 5.43 Small System Treatment Technology Overview (*continued*)

Technology	Advantages	Disadvantages
<i>Inorganic contaminant removal</i>		
Reverse Osmosis	Highly effective	Expensive waste removal
Ion exchange	Highly effective	Expensive waste removal
Activated alumina	Highly effective	Expensive waste removal
GAC	Highly effective	Expensive waste removal

will concentrate on filtration. There are about 650 to 700 package plants operating in the United States with capacities ranging from 7,200 gpd to 4 mgd (27.3 m³/day to 0.18 m³/sec) (152). Many are built to conventional design standards. Others, using tube settlers, have reduced size and larger capacities. More recently, package plants have been built with efficient and compact adsorptive clarifiers. Package plants with adsorptive clarifiers have capacities ranging from 14,400 gpd to 6 mgd (54.5 m³/day to 0.26 m³/sec).

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 emergency supplies, recreational areas, state parks, construction sites, ski areas, military installations, and other areas not served by public supplies.

Package plants can differ widely with regard to design criteria, and operating and maintenance considerations. Influent water quality is the most important consideration in determining the suitability of a package plant application. Complete influent water quality records should be examined to establish turbidity levels, seasonal temperature fluctuations, and color level expectations. Both high turbidity and color may require coagulant dosages beyond many package plant design specifications. In cases of consistently high levels of turbidity and color, the package plant capacity should be down-rated or a larger model selected. Where turbidity exceeds 100 to 200 NTU, presedimentation may be required as a pretreatment step. Pilot tests may be necessary to select a package plant for more innovative designs using high flow rates and shorter detention time unit processes.

System Description and Design Considerations. The three basic types of package plant filtration systems are conventional package plants, tube-type clarification package plants, and adsorption clarifier package plants.

Conventional Package Plants. Conventional 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:

- 20- to 30-minute flocculation detention time
- 2-hour sedimentation detention time
- Rapid sand filters rated at 1.34 L/sec/m² (2 gpm/ft²)

Tube-Type Clarifier Package Plants. This type of plant has two versions with different capacity ranges. One version can treat from 10 to 100 gpm (0.63 to 6.3 L/sec) and the other, equipped with dual units, can treat from 200 to 1,400 gpm (12.6 to 88.3 L/sec).

In these 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 to 20 minutes, depending on the flow.

The flocculated water then enters the tube settlers, which consist of many 1-in (2.5-cm) diameter, 39-in

(99-cm) long split-hexagonal-shaped passageways. The large surface area of the many 1-in (2.5-cm) diameter tube settlers achieves an effective clarification overflow rate of less than 150 gpd/ft² (6.1 m³/day/m²). Adequate clarification is attained with less than 1 5-minute detention time.

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 headloss is reached, backwashing of the filter is initiated automatically. A manual backwash cycle can be initiated at any time, if desired. Settled sludge from the tube settlers is flushed during the backwashing cycle. Combining backwashing and tube settler flushing simplifies operation and reduces operator skill requirements.

Adsorption Clarifier Package Plant. The adsorption clarifier 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.

While passing through the adsorption media, the coagulant and water are mixed, contact flocculated, and clarified. The mixing intensity, as measured by the velocity gradient, ranges from 150–300 sec⁻¹. Flocculation is accomplished through turbulence as the water passes through the adsorption media. In addition, flocculation is enhanced by 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 clarifier can achieve 95% or greater removal at 10 gpm/ft² (6.8 L/sec/m²) (153). 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 headloss 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.

Operation and Maintenance. Package plant operation is simplified by automated features, and maintenance requirements are well documented in manuals (see Table 5.44). 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 operation 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.

Cost Estimation Models for Drinking Water Treatment Unit Processes

Cost estimation plays an integral role in planning, implementation, and administration of any water treatment project. Over the last 20 years, U.S. Environmental Protection Agency has developed a substantial

TABLE 5.44 Operational Conditions For Treatment Technologies (154, 155)

Technology	Level of operational skill required	Level of maintenance required	Energy requirements
Granular activated carbon	Medium	Low	Low
Packed column aeration	Low	Low	Varies
Slow sand filtration	Low	Low	Low
Diatomaceous earth	Low	Medium	Medium
Reverse osmosis	Low	Medium	High
Chlorine	Low	Low	Low
Ozone	High	Medium	Varies
UV	Low	Low	Low

amount of cost data and related cost curves that can be used to estimate the cost of various unit processes. These unit processes can in turn be aggregated into water treatment systems. These cost curves have been used extensively for determining the economic impact of proposed federal drinking water regulations. The initial cost curves were presented in the late 1970s and early 1980s (156, 157). Various efforts have been made to upgrade and modify these cost curves since these first reports were published (158, 159). A complete list of unit processes for which cost curves have been developed are listed elsewhere (160). Description of applications of costing are reported in literature (161–163).

Treatment systems are usually divided into two main classes—large systems (1 to 200 mgd (3785 to 757,000 m³/d)) and small systems (2500 gpd to 1 mgd (9.5 to 3785 m³/d)). This separation is made because many processes applicable to one range are not applicable to the other, and even when a process is applicable, the conceptual design of the components varies significantly.

Costs are estimated under two main categories—construction costs and operation and maintenance costs. Construction costs are presented in terms of eight components: excavation and site work, manufactured equipment, concrete, steel, labor, pipes and valves, electrical equipment and instrumentation, and housing. Each component cost can be updated using an appropriate cost index to reflect variations in escalation, or a composite index such as the Engineering News Record Construction Cost Index can be used instead.

In addition to the construction costs, the following special costs need to be added to obtain the total capital cost: general contractors overhead and profit, engineering costs, land costs, legal fiscal and administrative costs, and interest during construction. Since these costs vary significantly from region to region, they are not included in the cost curves developed but can be added at the appropriate point in the calculations (160).

The operations and maintenance (O&M) costs are developed based on five main components: energy requirements (process-related energy and building-related energy), natural gas, diesel fuel, maintenance materials, and labor. Energy requirements are separated for process and buildings to account for the large geographic variations in the building energy requirements. Local variations in energy and labor rates can be incorporated conveniently, since the costs are obtained from the cost curves in terms of energy units and labor hours. The maintenance materials requirements, which are for all repair and maintenance items, are calculated based on national U.S. averages. Chemical costs are not included in these material costs, due to large regional variations in chemical costs. However, these costs need to be included in any final cost calculations.

OTHER WATER SOURCES

In addition to ground and surface water processes, other possible sources of water supply include wastewater reuse and desalinization. Each of these possibilities is discussed in separate sections below.

Wastewater Reuse

It is important to distinguish carefully between reuse for nonpotable and potable purposes (105). The potential for nonpotable reuse is far greater than for potable reuse if for no other reason than that the demands for nonpotable uses far exceed potable requirements. Also, the public health hazards are far less significant for nonpotable uses. It is important to note that direct nonpotable reuse is already widely practiced as, for example, Bethlehem Steel's use of Baltimore's wastewater. Wastewater can of course also be reused for such purposes as industrial, agricultural, and recreational facilities. As interest in renovated wastewater grows, more and more communities and organizations are investigating its potential. The uses to which reused water should be put will of course vary with local conditions. Any community that is considering reuse will have to evaluate fully the problems and possibilities of its own site-specific situation (106).

Potable reuse can be accomplished in any one of three ways. (1) Direct potable reuse by the reintroduction of highly treated sewage effluent from the treatment plant directly back into the existing water distribution system. This is the classical "pipe-to-pipe" definition of reuse. (2) Planned indirect reuse involving the purposeful discharge of highly treated wastewater upstream from a water supply intake. (3) Groundwater recharge involving either the injection of effluent into an aquifer that is the source of potable supply, or the spreading of effluent on the ground to allow it to filter down to the aquifer. In each of these methods, renovated wastewater eventually reaches the home water tap.

One of the few locations in the United States that is contemplating direct water reuse is in Denver, Colorado, the site of one of the largest and most carefully conceived projects for wastewater reuse in the United States. Denver is also one of the few places that is actively planning a direct pipe-to-pipe reuse system. Denver is currently conducting extensive pilot and field-scale research into the proper combination of unit processes to provide treated wastewater into the drinking water system. Figure 5.61 shows the process flow diagram for the proposed Denver reuse plant.

To properly understand the challenge associated with treating domestic wastewater to the quality associated with potable supply, one has to examine the typical domestic wastewater characterizations and their concentrations. Table 5.45 contains typical wastewater concentrations against the drinking water standards (107). Column 2 contains the 14-plant data point average of these concentrations for various contaminants associated with the discharge. Column 3 contains the drinking water standards for easy comparison. The difference between the two columns provides some indication of the kind of treatment and the difficulty that will be encountered in achieving a water supply of potable quality.

Table 5.46 summarizes expectant constituent removal by unit processes. Blanks in the table indicate no data or inconclusive results. The unit processes considered are primary treatment, trickling filters, softening, coagulation and flocculation, sedimentation, filtration, selective ion exchange, reverse osmosis, chlorination, and ozone (107). The unit process trains and their associated removal capabilities as shown in Table 5.45 with drinking water standards will provide some indication of how difficult it is to treat wastewater to drinking water standards.

Desalinization

Sources of saline water include: (1) undiluted seawater, containing about 35,000 mg/L total dissolved solids (TDS), and (2) brackish water, containing from 1000 to about 10,000 to 15,000 mg/L TDS and are widely represented by highly mineralized groundwaters and diluted seawater (80).

The major saline water conversion processes are distillation, electrodialysis, freezing, and reverse osmosis. Ion exchange is also a process used in desalinization. Each of these technologies is discussed briefly in the following sections.

Distillation. Distillation, or evaporation of saline water to produce fresh water, goes back to antiquity, and is the only process that was seriously used or considered prior to World War II. Today, distillation is the most

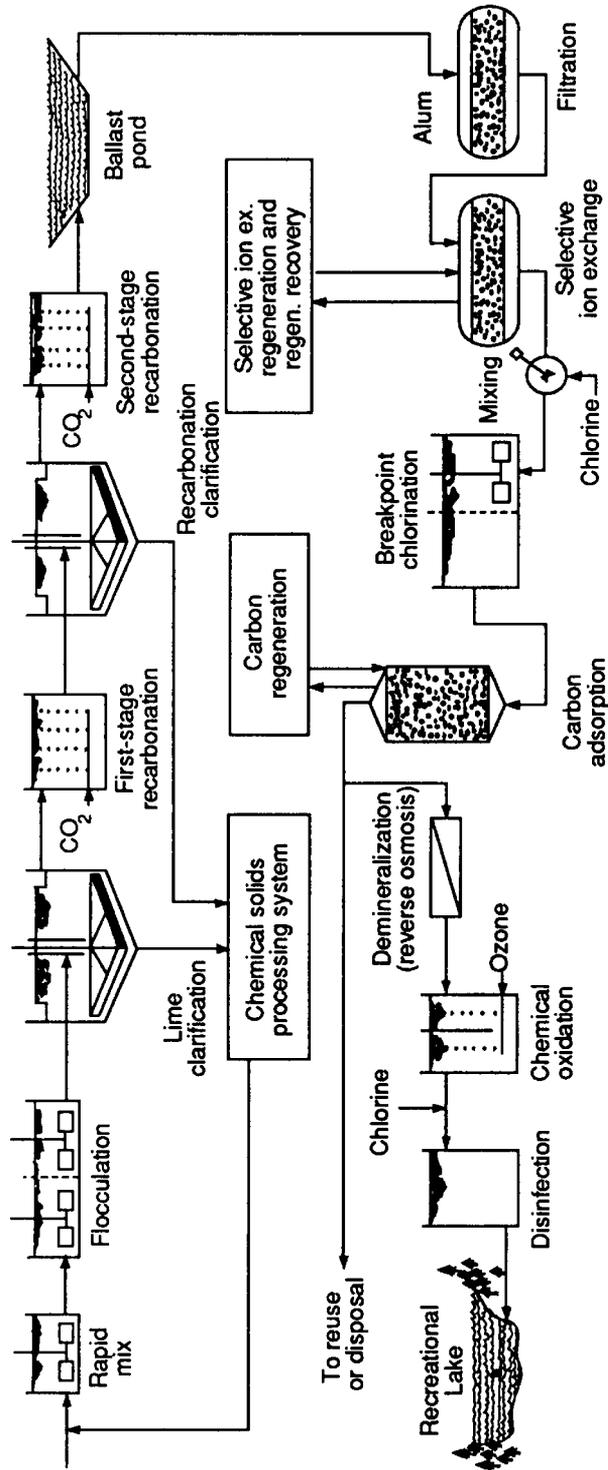


FIGURE 5.61 Process flow diagram—proposed Denver reuse plant (106).

TABLE 5.45 Comparison of Domestic Wastewater and Drinking Water Constituent Concentrations* (107, 129)

Constituent	Domestic wastewater, mg/L	Drinking water maximum contaminant level, mg/L	
		WHO	EPA
BOD	181		
COD	417		
TSS	192		
NH ₃ -N	20		10
NO ₃ -N	0.61	50	—
Org-N	13		
Ortho-P	6.8		
Total-P	9.4		
Alkalinity	211		
Oil and grease	61		
pH	7.2		
Arsenic	0.007	0.05	0.05
Barium	0.235	1.0	2
Cadmium	0.008	0.01	0.015
Chromium	0.167	0.05	0.1
Copper	0.117	—	—
Fluoride	0.864	0.7–1.7	1.4–2.4
Iron	2.25	—	—
Lead	0.148	0–1	**
Manganese	0.117	—	—
Mercury	0.112	—	0.002
Selenium	0.006	0.001	0.05
Silver	0.022	—	0.005
Zinc	0.419	—	—
Endrin [‡]	0.005	—	—
Lindane [‡]	0.260	—	—
Methoxychlor [‡]	0.102	—	—
Toxaphene [‡]	0.001	—	—
2,4-D [‡]	0.152	—	—
2,4,5-IP Silvex [‡]	0.110	—	—
TOC	102		
Hardness	362		
Color P-C units	67		
Turbidity	2.16		
Foaming agents	1.07		
TDS	2577		

*All values in mg/L except as noted.

[‡]Average from 14 data sources.[‡]ppb.

**Treatment technique requirement, action level 0.015 mg/L.

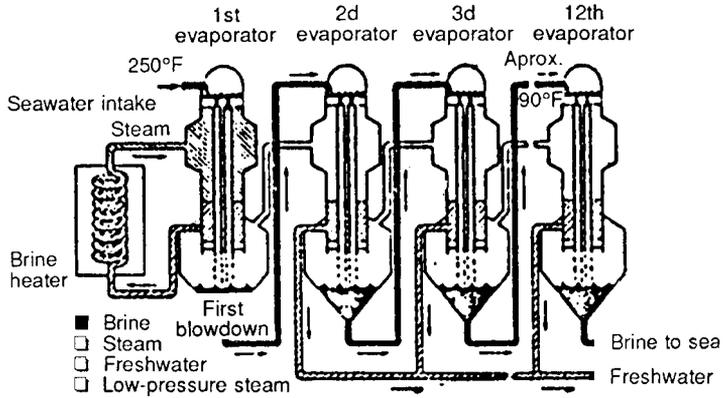
TABLE 5.46 Percent Removals by Various Treatment Processes (107)

Constituent	Wastewater treatment		Water treatment		
	Primary treatment	Trickling filters	Softening	Coagulation–sedimentation	RO
BOD	42	69	53	76	
COD	38	58	52	69	89
TOC	34		73	51	
TSS	33	44	65	83	
Alkalinity	—	16			
Oil and grease	65		40	89	
NH ₃ N	18				
Color	15		46	72	
Foaming agents	27		39	55	
Turbidity	31		67	86	
Phosphorus	27				
Arsenic	34		6	83	
Cadmium	38	0	30	72	
Chromium	44	0	56	86	
Copper	49	56	55	86	
Fluoride			50	44	
Iron	43		87	83	
Lead	52	44	44	90	
Manganese	20		93	40	
Mercury	11	0	0	24	
Selenium	0		0	0	
Silver	55		49	89	
Zinc	36		78	80	

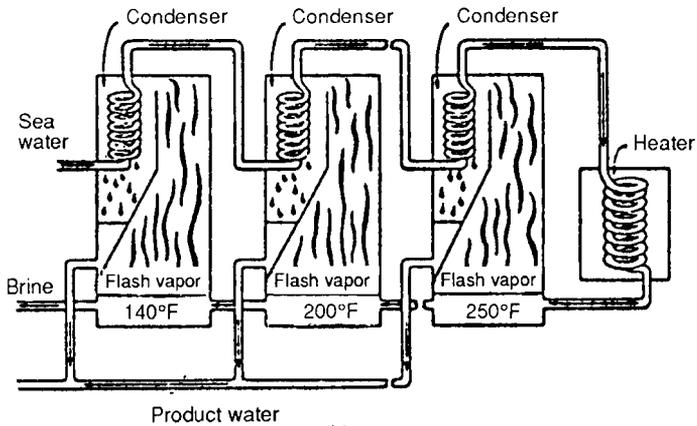
widely used saline water process, and is the most highly developed and economical process for use on seawater. There are three types of distillation processes currently being used. These are: (1) long tube vertical (LTV), (2) multistage flash (MSF), and (3) vapor compression (VC). In LTV distillation, the water can be vaporized and flows by gravity down the inside of a long vertical tube, while steam or hot vapor supplies heat on the outside. In MSF distillation, the water is heated under pressure in tubes and then allowed to expand suddenly or flash into a chamber. As some of the water evaporates or flashes, the remaining water cools slightly and then flows into another chamber at lower pressure where it flashes again. The flashed vapor condenses on the outside of tubes in each chamber, through which cold water is flowing and picking up heat. The condensed pure water then drips into collecting pans and is pumped to service. In VC distillation, pure water vapor, which has been evaporated at a tube surface or in a flash chamber, is mechanically compressed using flash centrifugal or actual flow gas compressor to raise its temperature and pressure for use in vaporizing more water. VC cycles must utilize mechanical or electrical energy or work rather than heat as the primary energy input for distillation. Figure 5.62 contains schematics of these three distillation processes.

Electrodialysis. Electrodialysis (ED) was developed subsequent to World War II and was the first process to demonstrate that there was a practical method other than distillation for salt water. In ED, salts and minerals are removed from a stream of saline water through special plastic membranes by the action of a direct electrical current. These salts and minerals pass through the membrane in the form of positively and negatively charged ions. The water from which these ions have been removed flows between the membranes and

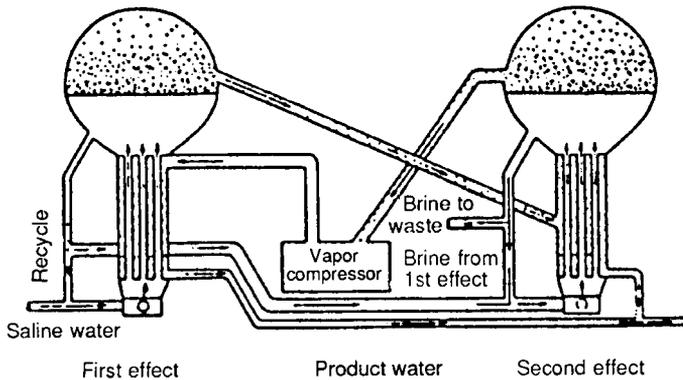
WATER SUPPLY



(a)



(b)



(c)

FIGURE 5.62 Schematics of various distillation procedures. (a) Simplified diagram of long-tube vertical distillation process. (b) Simplified diagram of multistage flash distillation process. (U.S. Department of the Interior, Office of Saline Water.)

is collected as a partially demineralized product by manifolds cut through the membranes. The salts and minerals removed from the product stream pass through the membranes into another stream of water which continuously washes the other side of each membrane and emerges through manifolds as a more concentrated waste stream. The amount of electrical current and membrane surface required to desalt a given quantity of water by ED depends on the amount and type of mineral to be removed; hence the cost of ED treatment is also dependent upon the amount and type of mineral to be removed. Figure 5.63 shows a schematic of the basic ion and water flow in ED.

Freezing. If saline water is frozen in such a way that salt crystals or brine pockets are not trapped in the ice crystals, the resulting ice will be essentially free of minerals. The mineral-free ice can be carefully separated from the brine and melted to produce potable water from saline water. This process and experience have attracted much attention for the past decade or more.

Ion Exchange. Ion exchange (IE) processes have been used for 30 or more years for the so-called zeolite softening of water and, more recently, for demineralization by the use of both cation and anion beds, or mixed beds regenerated by acid and alkali. Increased interest in saline water conversion and particularly brackish water conversion has led to new developments in the IE field, with the objective of broadening the range of TDS over which IE is useful. Up until recently, IE was not much used for demineralization on waters containing more than a few hundred parts per million TDS, since both the size of the required IE and the

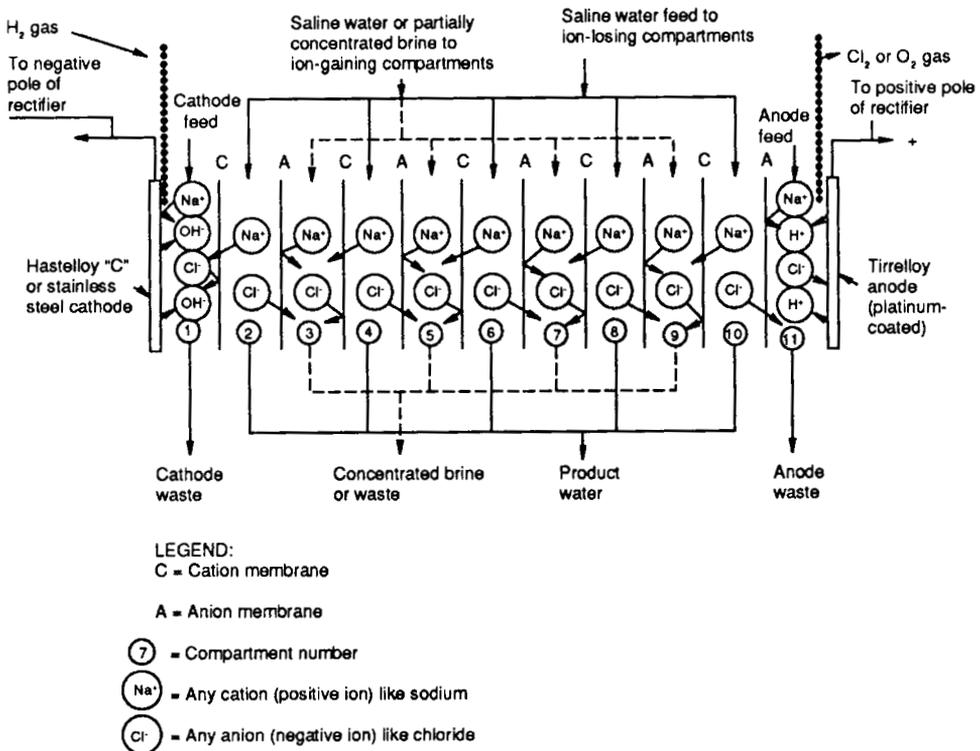


FIGURE 5.63 Basic ion and water flow in an electrodiolysis stack (80).

amounts of chemical regenerate required became prohibitive. Theoretically, at least, one equivalent of acid and one equivalent of alkali is required in an IE process for the removal of one equivalent of salt or mineral from water. In processes available up until recently, 2 to 3 equivalents of acid and 1.2 to 2.0 equivalents of alkali per equivalent of salt removed were required.

DISTRIBUTION

Transmission and distribution systems are needed to deliver water to individual customers in the required quantity and at satisfactory pressure. The distribution system is generally the major investment of municipal waterworks. Distribution systems can be considered to consist of both storage facilities and service lines.

Distribution System Water Quality

Drinking water treatment in the United States has played a major role in protecting public health through the reduction of waterborne disease. For example, in the 1880s, for one year, the typhoid death rate was 158 deaths per 100,000 in Pittsburgh, Pennsylvania but by 1935 the typhoid death rate had declined to 5 per 100,000. These reductions in waterborne disease outbreaks were brought about by the use of sand filtration, disinfection, and the application of drinking water standards (164).

Concern over waterborne disease and uncontrolled water pollution resulted in a dramatic increase in Federal water quality legislation between 1890 and 1970. Even though significant advances were made in elimination of waterborne disease outbreaks during that time period, drinking water concerns began to change. By the 1970s more than 12,000 chemical compounds were known to be in commercial use, with many more being added each year. Many of these chemicals cause contamination of ground and surface water and are known to be carcinogenic and/or toxic. The passage of the Safe Drinking Water Act of 1974 was a reflection of this concern.

The Safe Drinking Water Act of 1974 and its Amendments of 1986 (SDWAA) requires that the U.S. Environmental Protection Agency establish maximum contaminant level goals (MCLGs) for each contaminant that may have an adverse effect on the health of persons. Each goal is required to be set at a level at which no known or anticipated adverse effects on health occur, allowing for an adequate margin of safety (165). Maximum Contaminant Levels (MCLs) must be set as close to MCLGs as feasible.

The SDWAA has posed a massive challenge to the United States drinking water industry. A large number of regulations are being implemented over a short time frame and will cause the water utility industry difficulty in meeting the various rules and regulations established under the SDWAA. Initially, the emphasis in understanding, interpreting, and implementing the SDWAA and its regulations focused on treated water. There is substantial evidence, however, that many factors can cause water quality to deteriorate between the treatment plant and the point of consumption. Some of these factors include: chemical and biological quality of source water; effectiveness and efficiency of treatment processes; adequacy of the treatment facility, storage facilities and distribution system; age, type, design, and maintenance of the distribution network; and quality of treated water (166).

Most of the regulations established under the SDWAA have been promulgated with little understanding of the effect that the distribution system can have on water quality. However, the SDWAA has been interpreted as meaning that some MCLs shall be met at the consumer's tap, which, in turn, has forced the inclusion of the entire distribution system when considering compliance with a number of the SDWAA MCLs, rules, and regulations.

Distribution systems are frequently designed to insure hydraulic reliability, which includes adequate water quantity and pressure for fire flow as well as domestic and industrial demand. In order to meet these goals large amounts of storage are usually incorporated into system design, resulting in long residence

times, which in turn may contribute to water quality deterioration. Many water distribution systems in this country are approaching 100 years old and an estimated 26% of distribution system pipe in this country is unlined cast iron and steel and is in poor condition. At current replacement rates for distribution system components, a utility will replace a pipe every 200 years (167).

Conservative design philosophies, an aging water supply infrastructure, and increasingly stringent drinking water standards are resulting in concerns over the viability of drinking water systems in the United States. Questions have been raised over the structural integrity of drinking water systems in the United States as well as their ability to maintain water quality from the treatment plant to the consumer.

SDWAA regulations, emphasizing system monitoring include the Surface Water Treatment Rule (SWTR), the Total Coliform Rule (TCR), the Lead and Copper Rule, and the Trihalomethane Regulation. Both the SWTR and the TCR specify treatment and monitoring requirements that must be met by all public water suppliers.

The SWTR requires that a detectable disinfectant residual be maintained at representative locations in the distribution system to provide protection from microbial contamination. The TCR regulates coliform bacteria, which are used as “surrogate” organisms to indicate whether a system contamination is occurring. Monitoring for compliance with the Lead and Copper based entirely on samples taken at the consumer’s tap. The current standard for trihalomethanes (THMs) is 0.1 mg/L for systems serving more than 10,000 people but the anticipated Disinfectant and Disinfection By-Products (D-DBP) rule may impose the current (or a reduced) THM level on all systems. This regulation also requires monitoring and compliance at selected monitoring points in the distribution system.

Some of these regulations may, however, provide contradictory guidance. For example, the SWTR and TCR recommend the use of chlorine to minimize risk from microbiological contamination. However, chlorine or other disinfectants interact with natural organic matter in treated water to form disinfection by-products. Raising the pH of treated water will assist controlling corrosion but will increase the formation of trihalomethanes.

A recent development has been the application of water quality models to study the propagation of contaminants in distribution systems. Several simulation models are currently in use that predict changes in disinfectant concentrations based on hydraulics and chemical kinetics. For example, EPANET is a public sector water quality model distributed by the U.S. Environmental Protection Agency (168). Currently available water quality modeling programs are listed Table 5.47.

WATER STORAGE TANKS

Water storage tanks may come in different types of configurations, but all are used to provide storage to meet fluctuations in water use demand, to furnish storage for fire fighting, and to stabilize pressures in the water distribution system.

Types of Storage Tanks

The four prominent types of water storage tanks, as shown in Figure 5.64, are standpipe, reservoir, hydropneumatic, and elevated. A standpipe is characterized as a ground tank with a height greater than its diameter, while a reservoir’s height is equal to or less than its diameter. Hydropneumatic tanks may or may not have a flexible diaphragm separating the air from the water. As illustrated in Figure 5.65, elevated storage tanks are furnished in a variety of shapes.

Standpipes and reservoirs may be constructed with concrete or steel. Elevated tanks usually are steel construction. In recent years, a steel tank on a concrete tower has become popular in some areas.

TABLE 5.47 Available Water Quality Modeling Programs

Model	Distributor	Notes
AQUA	Computer Modeling, Inc. 2121 Front St. Cuyahoga Falls, OH 44221 (216) 929-7886	
Cybernet	Haestad Methods, Inc. 37 Brookside Rd. Waterbury, CT 06708 (800) 727-6555	Works within and requires AutoCAD. Steady-state water quality/time-of-travel only. Uses KYPIPE2 engine.
EPANET	Dr. Lewis Rossman USEPA 26 W. Martin Luther King Dr. Cincinnati, OH 45268 (513) 569-7603	Executable code available at no charge from EPA or via Internet. Source code available from EPA.
FAAST-3	Faast Software 3062 East Avenue Livermore, CA 94550 (510) 455-8086	
H2ONET	MW Soft, Inc. 300 N. Lake Ave., Suite 1200 Pasadena, CA 91101 (800) 442-0638	Works within and requires AutoCAD. Uses EPANET engine as basis for calculations.
KYPIPE3	Civil Engineering Software Center University of Kentucky Lexington, KY 40506 (606) 257-3436	
PICCOLO	Safège Consulting Engineers P.O. Box 727, Parc d'Ile 15-27 rue du Port Nanterre Cedex, France Phone: 01133146147181	
Stoner Work Station	Stoner Associates, Inc. P.O. Box 86 Carlisle, PA 17013 (717) 243-1900	
TDHNET	TDH Engineering 607 Ninth St. Laurel, MD 20707 (301) 490-4515	Steady-state water quality/time-of-travel only. Compatible with KYPIPE input format.
WATER	Municipal Hydraulics, Ltd. 2474 Pylades Drive RR#3 Ladysmith, B.C., Canada (604) 722-3810	
WaterCAD	Haestad Methods, inc. 37 Brookside Rd. Waterbury, CT 06708 (800) 727-6555	Contains a standalone CAD package. Uses EPANET engine as basis for calculations.
WATERMAX	The Pitometer Associates 2 N. Riverside Plaza Chicago, IL 60606 (312) 236-5655	

(continues)

TABLE 5.47 Available Water Quality Modeling Programs (*continued*)

Model	Distributor	Notes
Waterworks	Aquateq Research 50-2450 Hawthorne Ave. Port Coquitlam, B.C. Canada V3C 6B3 (604) 944-6831	
WATNET	WRC 2655 Philmont Ave. Huntington Valley, PA 19006 (215) 938-8444	

Tank Capacity

The selection and sizing of a water storage tank involve many considerations, including peak water demand, fire flow, distribution system, water quality, and economics. These factors also contribute to site selection, which considers elevation and site-specific features.

Peak Demand. Peak demand is usually the first capacity consideration. Most water treatment plants are operated best on a 24-hour production basis and produce a quantity of water equal to the average day’s demand. Clearwells offer a cushion between production and demand and are considered a production reserve rather than a distribution system reserve.

If the distribution system operates with a relatively constant pumping rate equal to the 24-hour demand–production rate, the variations in hourly demand must be stored in a tank(s). Demands are usually lowest in the early morning hours, when the storage tanks are filled. As the day progresses, demands tend to increase and to reach a peak rate in the late afternoon, when the tanks are feeding back into the distribution system. A system demand curve relationship is about the same for an average as well as a peak demand day.

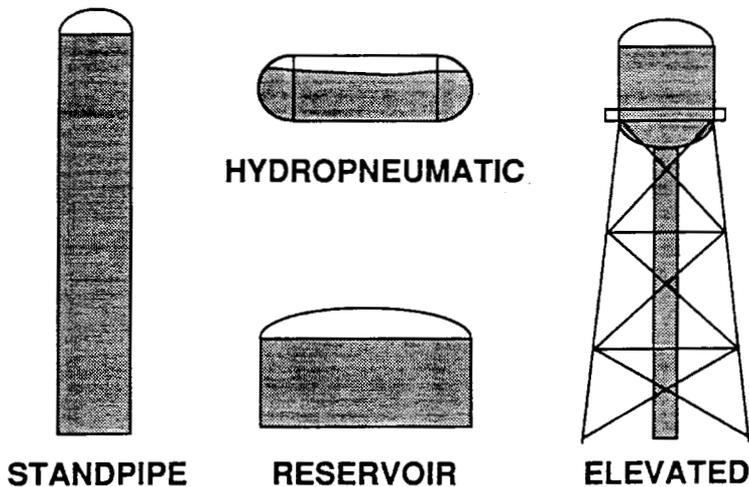


FIGURE 5.64 Types of Water Storage Tanks.

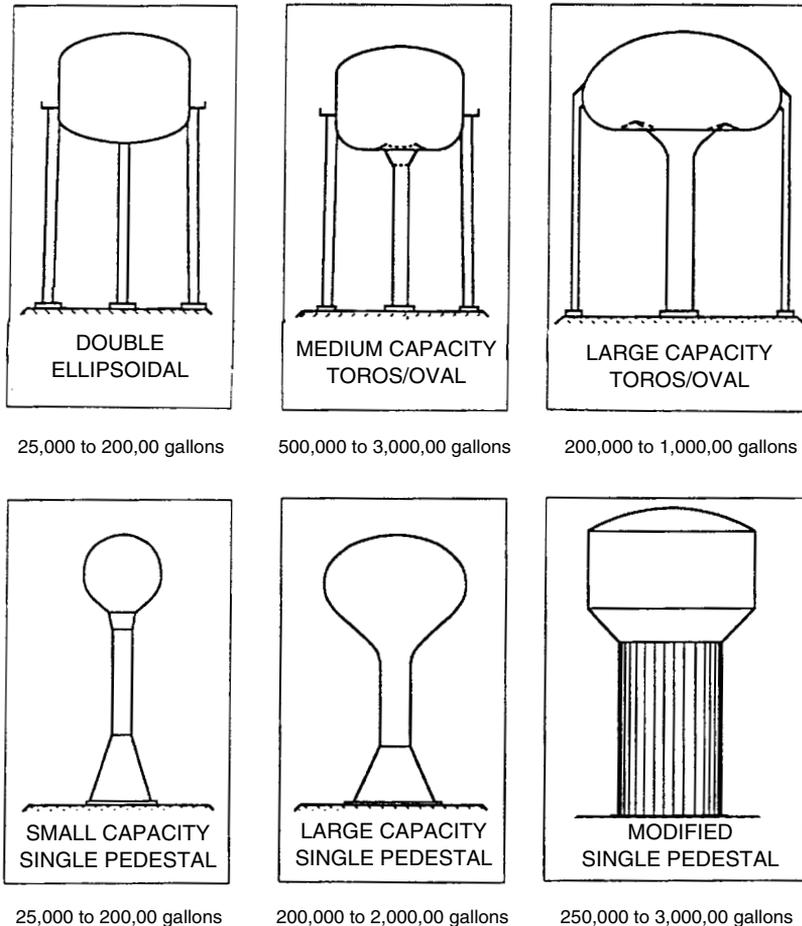


FIGURE 5.65 Types of Elevated Tanks. *Note:* One gallon = 3.785×10^{-3} cubic meters.

The filling and emptying of the tanks also help to maintain a relatively constant pressure in the system. To illustrate variations in flow during the day, a representative distribution system demand curve is shown in Figure 5.66.

Fire Flow. Fire flow is another important design factor. Insurance underwriters provide formulas to determine desirable quantities, pressures, and fire flow duration. These formulas involve consideration of all classes and uses of all buildings within the service area. When fire flow or a line break makes more of a demand on the distribution system than the stored water, additional pumping from the water treatment plant is required.

Distribution System. Head loss in the distribution system must be considered so that the water storage tanks may be filled daily. Otherwise, the tanks will only partially fill during periods of low demand, such as weekends. The stored water is useful only when the distribution system is adequate to transport it to the demand.

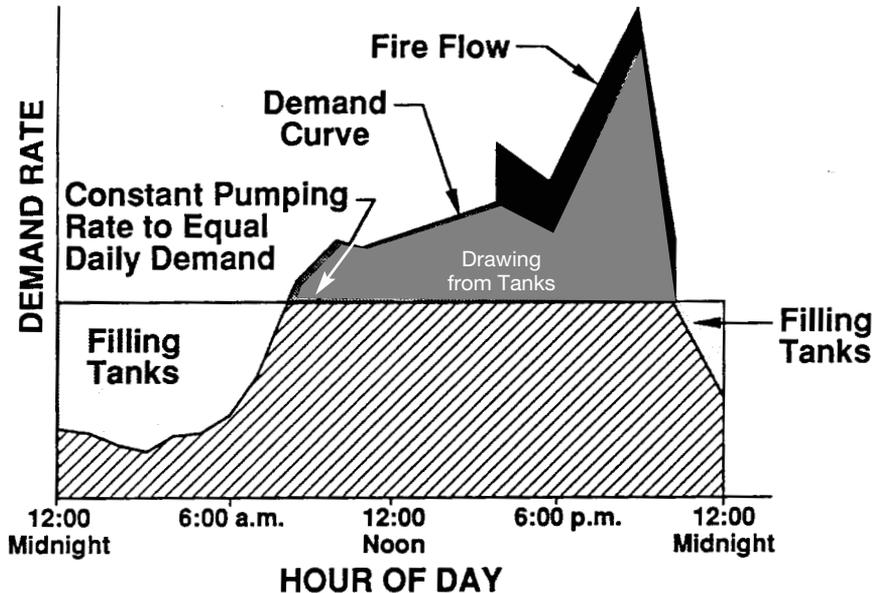


FIGURE 5.66 System Demand Curve for Typical Daily Flow.

Future needs are an important consideration, and the tank capacity must be examined with future expansion and demands on the distribution system in mind. When a new industry or other source of increased demand is contemplated, an important location factor may be sufficient quantities and pressures of water in the distribution system. Future demands also include distribution system changes, including acquisition of private and/or other public systems.

Water Quality. Water quality is an important issue in the design of water storage tanks. When there is considerable volumetric change, also known as turnover, during the day, the resulting mixing action minimizes holding time for water quality degradation. Such changes are common for elevated tanks. On the other hand, standpipes tend to operate on a last in, first out basis and, thus, provide minimal mixing. For large-diameter reservoirs, mixing also may be a potential problem.

The best options for maintaining water quality are specific to each site. Situations and water characteristics vary, but a minimum turnover of 15% is reasonable. Conditions are better when the water level in the tank fluctuates in the lower part of the tank rather than having the tank remain full all the time.

For new tanks in developing areas with initial low flow demands, mixing may be needed for only an interim period. When mixing is required, the storage tank size and shape are key factors. In some cases, separate inlet and outlet piping will provide sufficient mixing. In more critical cases, recirculation pumping or internal mixers may be necessary.

Economics. Capital and operating costs are influenced by tank elevation and size and by associated distribution system improvements. For example, the cost of a reservoir and standpipe are, respectively, a little less or some more than 50% that of an elevated tank. Unit prices are highest for small tanks and tend to level off for larger tanks holding more than 0.5 to 1 million gallons (1400 to 3800 cubic meters). Thus, for large installations, the use of two tanks may be considered to improve reliability and ease of maintenance operations.

Design and operation of water storage tanks is an important power management tool. Normally, water

and electric power demands fluctuate similarly during the day, and savings are achieved by off-peak electrical use. Rather than constant pumping for the water treatment plant, the part-time use of more or larger pumps and increased storage volume may be cost effective. More water is pumped to the storage tanks during low-consumption hours, resulting in less pumping during high water and electric demand hours.

Off-peak pumping is illustrated in Figure 5.67. The water use curve is the same as for Figure 5.66, but to take advantage of better power rates, the pumping varies. Alternative construction and operating life-cycle cost evaluations should be developed in cooperation with the local power provider.

Tank Site Selection

New water storage tanks are installed to meet a specific need of the water distribution system. To perform this function, the tank is normally located close to the area of need. However, other factors, namely elevation and site suitability, may make it more desirable and cost effective to locate the tank further away.

Also, the cost of repainting a tank located near an automobile parking lot, a building, or other public use area is escalated due to the risk of damage from drifting paint. This is especially critical with an elevated tank.

Site Size. Water storage tanks only require sites as modest as 100 ft by 100 ft (30 m by 30 m) for small tanks and 150 ft by 150 ft (45 m by 45 m) for large tanks. During the construction (erection) phase, an area of similar size is needed for material storage, equipment, and ground assemblies. Larger areas should be obtained where possible to lessen initial or future aesthetic or nuisance issues and to provide space for maintenance.

Elevation. For elevated tanks, the higher the site, the less tower height is needed to elevate the water to the proper elevation to provide the proper head. With 20 feet (6 m) or more of extra elevation, considerable cost

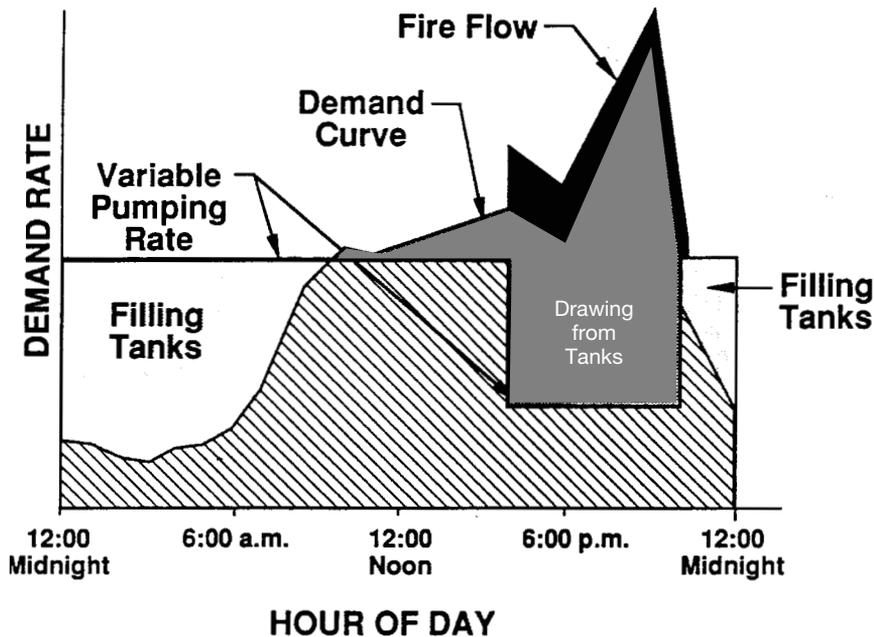


FIGURE 5.67 System Demand Curve with Off-peak Pumping.

savings may be achieved by reducing tower height by a tower section. Also, a standpipe or reservoir becomes more competitive at higher site elevations.

Typical design practice is to design the storage tank with the overflow at the water distribution datum plane. Since the tank should fill completely, a hydraulic analysis should be performed to ensure that system pressure losses will not be a problem. Booster pumps may be used on an interim basis until distribution improvements are completed.

Soil Conditions. Water storage tanks are heavy when full of water, and foundations are an expensive part of tank installation. Since tanks cannot tolerate differential settlement, a site with good foundation soils will save foundation costs and avoid potential settlement problems or structural tank failure.

Aesthetics. The not-in-my backyard syndrome (NIMBY) should be expected for potential sites in developed areas. Care should be taken to make the new water storage tank nonoffensive to passersby and adjacent development. Modern and special tank designs and screening techniques often are used when remote sites are not available or practical locations.

Regardless of the quality of material and craftsmanship used in the original coating applied to a new steel tank, cleaning and recoating will be required in the future. Public relations problems and extra expenses occur if the tank is located in an area sensitive to the drifting of dust from cleaning and droplets from painting. Problems may arise if the tank is near homes, roadways, or commercial establishments, such as automobile dealerships. Better locations are parks, industrial areas, or other locations where the surrounding area is less developed or where a temporary close-off around the tank is practical.

Tank Protection

Steel water storage tanks are very susceptible to corrosion on both the outside and inside surfaces. Normally, outside surfaces can be protected by coatings. However, a more aggressive environment is created for the submerged surfaces inside the tank, where both coating and cathodic protection may be necessary.

Painting is critical to the useful life of steel tanks. The requirements for quality paint work are a good painting specification, a coating system designed for the exposure conditions, appropriate surface preparation, correct application, and good inspection. Surface preparation, selection of the coating type, application method, and film thickness are important to the painting specification for new tanks and for refurbishing existing tanks.

Surface preparation removes contaminants and provides a sound substrate for coatings. Considerations in choosing surface preparation are the type of coating system applied, exposure considerations, economics, and physical and/or site limitations.

Protective coating systems include acrylics, waterborne epoxies, mastic and high-solids epoxies, and high-solids polyurethanes. Since the high-solids epoxies satisfy toxicological and volatile organic carbon emissions criteria, they are one of the few coatings suitable for interior applications. In cases of repainting a storage tank, compatibility of old and new coatings must be carefully evaluated.

In theory, cathodic protection mitigates corrosion at pinholes or other locations of exposed metals. This allows the system to operate at lower voltage levels. However, the compatibility of coating and cathodic protection systems must be established. Once installed, the cathodic protection system must be monitored and adjusted between recoatings.

Service Lines

Service lines are pipes, including accessories, that carry water from the main to the building or property being served. Service lines can be any size, depending on how much water is required to serve a particular cus-

tother. Most service lines are tapped directly to the main by means of special machines, which for 2-in (5-cm) and smaller services drill, thread, and insert special valves called corporation stops. For 3-in (7.5-cm) and larger taps, a tapping machine drills a hole in the main through a tapping valve attached to the main by means of a tapping saddle or sleeve. The service pipe is then attached to the corporation stop or tapping valve and laid as desired to the point of service, which is normally a meter setting or a curb stop located at the property line. Table 5.48 lists the maximum size taps that can be tapped directly to ductile iron mains for various thread engagements. Service saddles for services up to 2 in (5 cm) in size may be used on pipe material, such as PVC, because direct tapping is not recommended. Figures 5.68 and 5.69 show typical installations for a 2-in (5-cm) or smaller service in warmer climates and typical installation for a 3-in (7.5-cm) or larger service, respectively. In cold climates, meters are installed in buildings not subject to freezing.

Service pipes should be sized so that the utility's design pressure is maintained at the customer's property for the desired flows and excessive flow velocities are minimized. Pressure loss due to pipe friction is usually the controlling factor in selecting a proper pipe size. Velocities should be limited to control cavitation, water hammer, and excessive pipe noise. Table 5.49 shows friction losses in popular sizes of the most commonly used pipe materials, and Table 5.50 shows computed pressure losses for service components.

Currently, the most commonly used materials for 2-in (5-cm) and smaller service piping are flexible copper, polyethylene, and polybutylene. Type K copper is the most commonly used material for copper services. The American Water Works Association (AWWA) has established standards for both polyethylene and polybutylene tubing. AWWA Standard C-902 covers polybutylene pipe, tubing, and fittings from 0.5 in (1.3 cm) through 3 in (7.5 cm) for water. The most common materials for 3-in and larger services are ductile iron, PVC, asbestos cement, and steel. The AWWA has also published standards for these pipe materials. Good construction practices must be used when installing service lines to avoid costly repairs in the future. This must include burying the pipe below frost lines, maintaining proper ditch conditions, proper backfill, trench compaction, and protection from underground structures which may cause damage to the pipe.

Water Mains

The most commonly used pipes for water mains are ductile iron, prestressed concrete cylinder, polyvinyl chloride (PVC), reinforced plastic, steel, and asbestos cement.

Ductile Iron Pipe

Ductile iron is a form of cast iron in which the graphite in flake form has been converted to nodules or spheres. The result is a material that has high strength, ductility, and the corrosion resistance of gray cast iron. Ductile iron is the only form of cast iron pipe manufactured in the United States today. AWWA Standard C-150 and C-151 cover the design and manufacture of this pipe. Ductile iron pipe is available in sizes from 3 in (7.5 cm) through 60 in (152.4 cm), and in 18 and 20 ft (5.5 and 6.1 m) nominal lengths for working pressure ratings up to 350 lb/in² (24.6 kgf/cm²). Table 5.51 illustrates laying conditions and design thickness classes for different types of service. Cement mortar lining should be specified for corrosion protection when obtaining ductile iron pipe for water or sewage service. This is covered by AWWA Standard C-10S. A variety of joints are available for use with ductile iron pipe, which includes push-on (the most common), mechanical, flanged, ball-and-socket, and numerous joint designs (108).

Prestressed Concrete Cylinder Pipe. Prestressed concrete cylinder pipe is a composite pipe made up of a concrete core inside a steel cylinder with high-tensile-strength wires spirally wrapped around the cylinder and covered with a rich cement mortar. This pipe is available in diameters from 16 to 144 in (40.6 to 365.8 cm), and in nominal 20-ft (6.1-in) lengths for pipes through 108 in (274.3 cm) in diameter. For larger pipes, the nominal laying length is 16 ft (4.9 in). Since this is a composite pipe, each length must be designed for

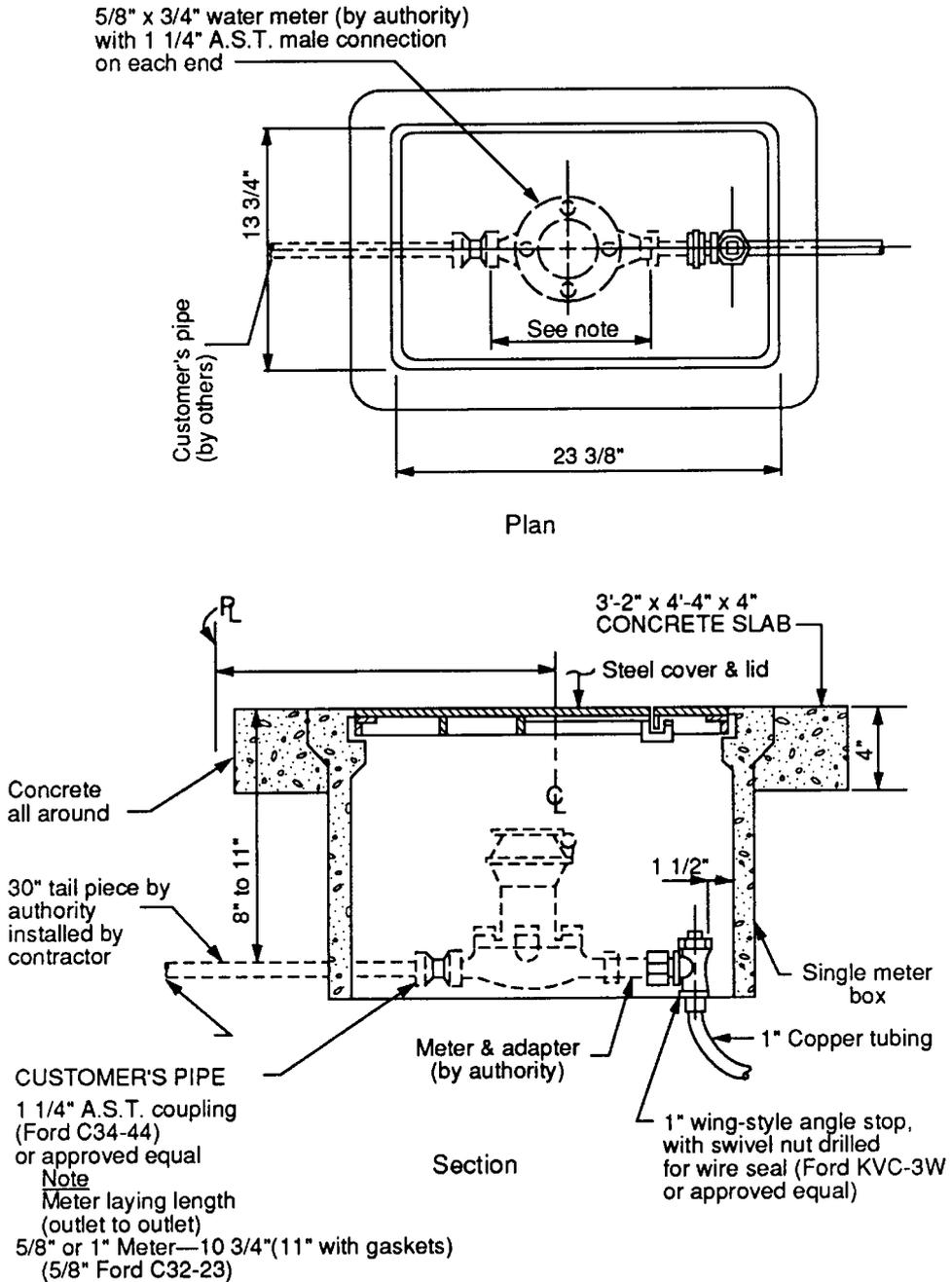


FIGURE 5.68 Miami-Dade Water & Sewer Authority single meter box.

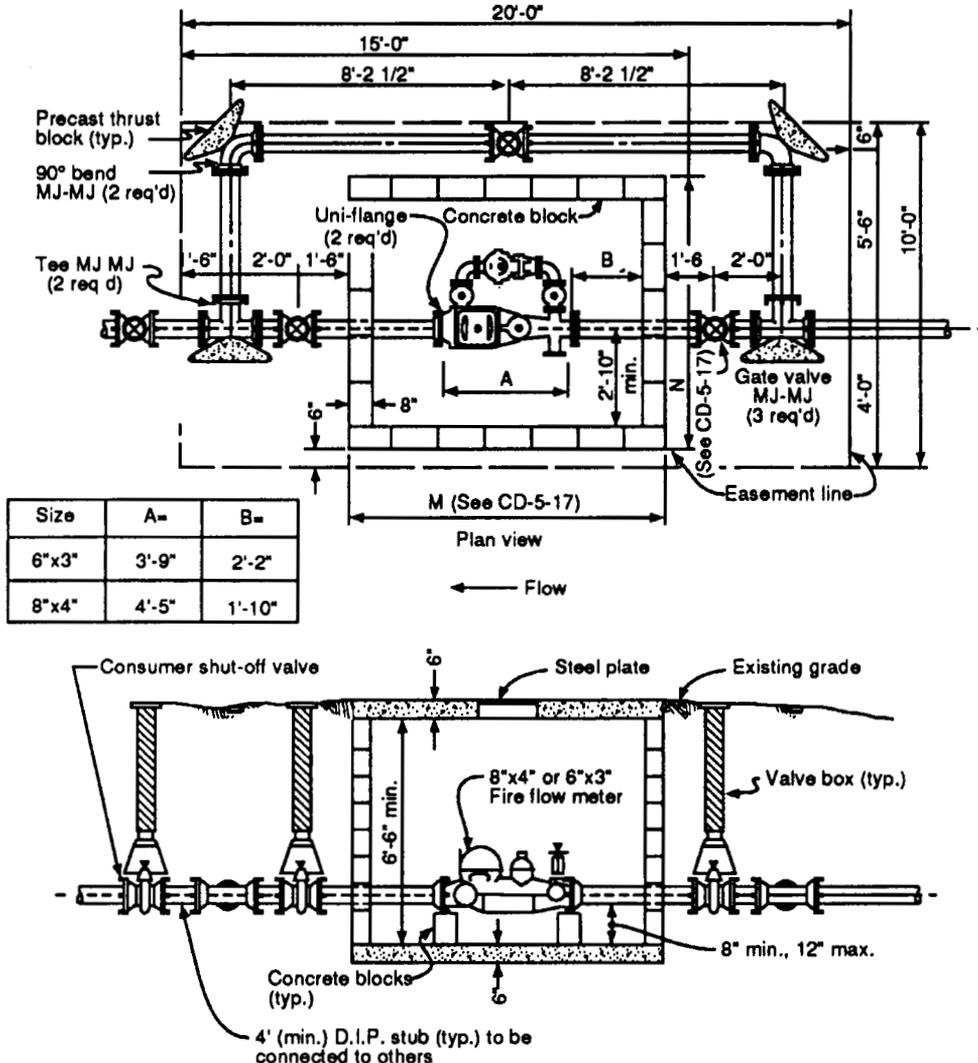


FIGURE 5.69 Tampa Water Department meter service installation.

the specific pressures and external loadings encountered in service. A variety of joints and adapters are available to connect two standard appurtenances. Figures 5.70 and 5.71 show a cross section of a lined and embedded prestressed cylinder pipe. AWWA Standard C-301 covers the design and manufacture of prestressed concrete cylinder pipe for water service (109).

Polyvinyl Chloride Pipe. Polyvinyl chloride (PVC) pipe is manufactured from thermoplastic compounds prepared by combining PVC resin with modifiers, stabilizers, lubricants, and pigments. AWWA Standard C-900 covers pipe 4 to 12 in (10.2 to 30.5 cm), which is available in nominal 20-ft (6.1 m) lengths. Pipe under this standard is available in three pressure classes: 100, 150, and 200 lb/in² (7.0, 10.5, and 14.1 kgf/cm²). In-

TABLE 5.49 Comparison of Computed Water Flow Friction Losses for Service Line, lb/in²/ft

Flow rate, gpm	¾ in			1 in			1¼ in	
	Copper Type K	Plastic or iron pipe ID	Plastic or Copper Tube OD	Copper Type "K"	Plastic or Iron Pipe ID	Plastic or Copper Tube OD	Copper Type "K"	Plastic or Iron Pipe ID
1	0.003	0.002	0.005					
2	0.008	0.005	0.013					
3	0.016	0.010	0.026					
4	0.025	0.015	0.041					
5	0.037	0.024	0.065	0.009	0.007	0.022		
6	0.048	0.031	0.081	0.011	0.008	0.029		
7	0.063	0.041	0.108	0.018	0.011	0.038		
8	0.075	0.049	0.128	0.021	0.013	0.045		
9	0.095	0.062	0.160	0.026	0.017	0.057		
10	0.121	0.078	0.203	0.035	0.021	0.072		
11	0.166	0.108	0.274	0.057	0.040	0.104		
12	0.192	0.128	0.317	0.069	0.048	0.126		
13	0.206	0.134	0.338	0.071	0.050	0.130		
14	0.228	0.149	0.377	0.086	0.059	0.156		
15	0.263	0.169	0.428	0.107	0.076	0.200		
20	0.433	0.286	0.734	0.120	0.085	0.225	0.042	0.030
25	0.656	0.429	1.118	0.226	0.136	0.343	0.059	0.046
30	0.920	0.599		0.242	0.188	0.450	0.083	0.060
35		0.809		0.370	0.254	0.611	0.108	0.076
40						0.763	0.135	0.091
45						0.924	0.156	0.109
50						1.100	0.200	0.131
55							0.234	0.161
60							0.278	0.185

tegral bell and compiling joints are available with gaskets and solvent cement used for sealing. This pipe is lightweight and corrosion-resistant. PVC pipe under the AWWA standard is available in either steel pipe or cast iron pipe outside diameters. ASTM Standard D2241 in addition to AWWA C-900 covers PVC pipe for water service (110).

Reinforced Plastic Pipe. Reinforced plastic pipe is available in two forms: reinforced thermosetting-resin pipe and reinforced plastic mortar pipe. Reinforced thermosetting-resin pipe is a composite pipe made of glass fiber reinforcement embedded in a cured thermosetting resin. Reinforced plastic mortar pipe is a composite pipe made of glass fiber reinforcement and aggregate embedded in or surrounded by cured thermosetting resin. This type of pipe is available in 1 to 144 in (2.5 to 365.8 cm) diameter and in nominal 10, 20, 40, or 60 ft (3.0, 6.1, 12.2, or 18.3 m) lengths. Standard pressure classes available are 50, 100, 150, 200, and 250 lb/in² (3.5, 7.0, 10.5, 14.1, and 17.6 kgf/cm²). Bell-and-spigot joints with gaskets and adhesive sealers are available along with reinforced overlays and various mechanical types of joints. This pipe is covered by AWWA Standard C-950.

Steel Pipe. Steel pipe is divided into two categories according to method of manufacture: mill pipe and fabricated pipe. Mill pipe is defined as "steel pipe of any size produced to meet finished pipe specifica-

TABLE 5.50 Computed Pressure Losses for Service Components, lb/in²

Flow rate, gpm	Corporation stop, in		Curb stop, in		Globe valve, in		Gate or ball valve, in		Meter, in			
	¾ (1)	1 (2)	¾ (3)	1 (4)	¾ (5)	1 (6)	¾ (7)	1 (8)	⅝	¾	1	
1	0.015	0.004	0.011	0.003	0.060	0.017						
2	0.050	0.014	0.034	0.008	0.195	0.058	0.001					
3	0.103	0.029	0.071	0.017	0.301	0.09	0.002					
4	0.170	0.03	0.117	0.028	0.664	0.20	0.003		0.39			
5	0.246	0.07	0.170	0.04	0.962	0.29	0.005	0.002	0.62			
6	0.340	0.11	0.234	0.05	1.33	0.43	0.007	0.002	0.89	0.39		
7	0.457	0.14	0.315	0.07	1.79	0.57	0.00	0.004	1.22	0.53		
8	0.557	0.17	0.384	0.08	2.18	0.71	0.011	0.004	1.60	0.71	0.25	
9	0.703	0.22	0.485	0.10	2.75	0.85	0.013	0.005	2.02	0.89	0.32	
10	0.879	0.23	0.606	0.13	3.44	1.0	0.016	0.007	2.50	1.10	0.39	
11	1.20	0.28	0.72	0.15	4.08	1.2	0.020	0.009	3.00	.35	0.48	
12	1.31	0.32	0.83	0.18	4.72	1.4	0.024	0.011	3.60	1.60	0.57	
13	1.40	0.37	0.94	0.21	5.36	1.6	0.028	0.013	4.20	1.88	0.67	
14	1.54	0.42	1.05	0.24	6.00	1.8	0.032	0.015	4.90	2.18	0.77	
15	1.0	0.47	1.17	0.27	6.64	2.0	0.036	0.017	5.60	2.50	0.89	
16	2.11	0.53	1.31	0.30	7.46	2.3	0.040	0.019	6.40	2.85	1.02	
17	2.25	0.59	1.45	0.34	8.28	2.6	0.044	0.021	7.20	3.22	1.16	
18	2.36	0.65	1.60	0.38	9.10	2.9	0.049	0.023	8.10	3.60	1.30	
19	2.54	0.72	1.75	0.42	9.93	3.2	0.054	0.025	9.00	4.00	1.45	
20	2.75	0.80	1.90	0.46	10.76	3.5	0.059	0.028	10.00	4.45	1.60	
25		1.2		0.67		5.1		0.047		7.10	2.50	
30		1.6		0.92		7.0		0.062		10.00	3.60	
35		2.1		1.2		9.0		0.077			4.80	
40		2.7		1.5		11.6		0.086			6.40	
45		3.3		1.9		14.6		0.093			8.10	
50		4.0		2.3		17.6		0.130			10.00	

Note: (1) Based on equivalent loss of 5.86 ft of ¾ in type K copper tubing.

(2) Based on equivalent loss of 6.67 ft of 1 in type K copper tubing.

(3) Based on equivalent loss of 4.04 ft of ¾ in type K copper tubing.

(4) Based on equivalent loss of 3.85 ft of 1 in type K copper tubing.

(5) Based on equivalent loss of 22.90 ft of ¾ in type K copper tubing.

(6) Based on equivalent loss of 29.1 ft of 1 in type K copper tubing.

(7) Based on equivalent loss of 0.14 ft of ¾ in type K copper tubing.

(8) Based on equivalent loss of 0.21 ft of 1 in type K copper tubing.

To convert lb/in² to kgf/cm², multiply by 0.0703; to convert gpm to m³/min, multiply by 0.003785; to convert feet to meters, multiply by 0.3048.

tions.” This includes seamless, furnace-butt-welded, electric-resistance-welded, and fusion-welded pipe. Mill pipe is generally available in sizes from 0.125 to 36 in (0.3 to 91.4 cm). Fabricated pipe is manufactured from sections of steel plates with spiral or straight butt welded seams. AWWA Standard C-200 covers 6-in (15.2-cm) and larger steel pipe for water service. Various lengths are available or may be specified, but nominal random lengths may be furnished averaging 29 ft (8.8 in). Pipe can be furnished with ends for field butt welding or field-assembled mechanically coupled joints. Rubber-gasketed ball-and-spigot joints are also available. Protecting steel pipe from corrosion both internally and externally is most important to maintain the life of the pipe. To accomplish this, a variety of coatings are available. Cathodic protection is usually applied to buried steel pipe for additional protection (111).

TABLE 5.51 Pressures and Depths of Cover for Ductile Iron Pipe

Size, in	Thickness class	Thickness, in	Working Pressure, † lb/in ²	Maximum Depth of Cover in Feet‡				
				Laying Conditions				
				Type 1	Type 2	Type 3	Type 4	Type 5
	51	0.25	350	98	100*	100*	100*	100*
	52	0.28	350	100*	100*	100*	100*	100*
	53	0.31	350	100*	100*	100*	100*	100*
	54	0.34	350	100*	100*	100*	100*	100*
	55	0.37	350	100*	100*	100*	100*	100*
	56	0.40	350	100*	100*	100*	100*	100*
4	51	0.26	350	76	86	96	100*	100*
	52	0.29	350	100*	100*	100*	100*	100*
	53	0.32	350	100*	100*	100*	100*	100*
	54	0.35	350	100*	100*	100*	100*	100*
	55	0.38	350	100*	100*	100*	100*	100*
	56	0.41	350	100*	100*	100*	100*	100*
6	50	0.25	350	32	38	44	56	75
	51	0.28	350	49	57	64	80	100*
	52	0.31	350	67	77	86	100*	100*
	53	0.34	350	91	100*	100*	100*	100*
	54	0.37	350	100*	100*	100*	100*	100*
	55	0.40	350	100*	100*	100*	100*	100*
8	50	0.27	350	25	30	36	46	64
	51	0.30	350	36	42	49	61	81
	52	0.33	350	47	54	62	77	99
	53	0.36	350	64	73	82	100*	100*
	54	0.39	350	100*	100*	100*	100*	
	55	0.42	350	100*	100*	100*	100*	
10	50	0.29	350	19	24	29	38	55
	51	0.32	350	27	32	38	49	66
	52	0.35	350	35	41	47	59	79
	53	0.38	350	45	52	59	74	95
	54	0.41	350	57	65	74	91	100*
	55	0.44	350	67	77	86	100*	100*
12	50	0.31	350	17	22	27	36	52
	51	0.34	350	23	28	33	43	60
	52	0.37	350	30	35	41	53	71
	53	0.40	350	36	42	49	61	81
	54	0.43	350	45	52	59	74	95
	55	0.46	350	54	62	71	87	100*
	56	0.49	350	64	73	83	100*	100*

(continues)

TABLE 5.51 Pressures and Depths of Cover for Ductile Iron Pipe (*continued*)

Size, in	Thickness class	Thickness, in	Working Pressure,† lb/in ²	Maximum Depth of Cover in Feet‡				
				Laying Conditions				
				Type 1	Type 2	Type 3	Type 4	Type 5
14	50	0.33	350	15	19	24	33	49
	51	0.36	350	19	23	28	38	55
	52	0.39	350	24	29	34	44	62
	53	0.42	350	30	35	41	53	71
	54	0.45	350	36	42	49	61	81
	55	0.48	350	43	50	57	71	92
	56	0.51	350	52	59	67	83	100*
16	50	0.34	350	13	17	21	30	47
	51	0.37	350	16	21	25	34	51
	52	0.40	350	20	25	30	40	57
	53	0.43	350	25	30	36	46	64
	54	0.46	350	30	35	41	53	71
	55	0.49	350	35	41	47	59	79
	56	0.52	350	41	48	55	68	89
18	50	0.35	350	11	15	20	29	42
	51	0.38	350	14	19	23	32	49
	52	0.41	350	18	22	27	36	53
	53	0.44	350	22	26	31	41	58
	54	0.47	350	25	30	35	47	64
	55	0.50	350	30	35	41	53	71
	56	0.53	350	35	41	47	59	79
20	50	0.36	300	10	14	18	27	38
	51	0.39	350	13	17	21	30	44
	52	0.42	350	16	20	25	34	50
	53	0.45	350	19	23	28	38	54
	54	0.48	350	22	27	32	42	59
	55	0.51	350	26	31	37	47	65
	56	0.54	350	30	35	41	53	71
24	50	0.38	250	8	12	17	23	31
	51	0.41	300	10	15	19	27	36
	52	0.44	350	13	17	21	30	41
	53	0.47	350	15	19	24	33	47
	54	0.50	350	18	22	27	36	53
	55	0.53	350	20	25	30	40	57
	56	0.56	350	24	29	34	44	61
30	50	0.39	200		10	14	18	25
	51	0.43	250		12	16	21	29
	52	0.47	300		14	19	24	33
	53	0.51	350	§	17	21	29	38
	54	0.55	350		19	24	33	44
	55	0.59	350		22	27	36	51
	56	0.63	350		26	31	41	57

TABLE 5.51 Pressures and Depths of Cover for Ductile Iron Pipe (*continued*)

Size, in	Thickness class	Thickness, in	Working Pressure [†] , lb/in ²	Maximum Depth of Cover in Feet [‡]				
				Laying Conditions				
				Type 1	Type 2	Type 3	Type 4	Type 5
36	50	0.43	200		10	13	17	25
	51	0.48	250		12	16	20	28
	52	0.53	300		15	19	24	32
	53	0.58	350	§	17	21	28	37
	54	0.63	350		20	25	33	43
	55	0.68	350		23	28	37	50
	56	0.73	350		26	31	41	50
42	50	0.47	200		9	13	16	24
	51	0.53	250		12	15	19	27
	52	0.59	300		14	18	22	30
	53	0.65	350	§	17	22	27	35
	54	0.71	350		20	24	32	42
	55	0.77	350		23	28	38	48
	56	0.83	350		26	31	41	57
48	50	0.51	200		9	12	15	23
	51	0.58	250		12	14	18	26
	52	0.65	300		14	18	21	30
	53	0.72	350	§	17	21	25	34
	54	0.79	350		20	24	30	40
	55	0.86	350		23	28	37	47
	56	0.93	350		26	31	41	55
54	50	0.57	200		9	12	15	23
	51	0.65	250		12	14	18	25
	52	0.73	300		14	17	21	29
	53	0.81	350	§	17	21	25	34
	54	0.89	350		20	25	30	40
	55	0.97	350		23	28	37	47
	56	1.05	350		27	32	42	55

*Calculated maximum depth of cover exceeds 100 ft.

[†]These pipes are adequate for the rated working pressure plus a surge allowance of 100 lb/in². Ductile iron pipe for working pressure higher than 350 lb/in² is available.

[‡]An allowance for single H-20 truck with 1.5 impact factor is included for all depths of cover.

§Laying condition Type 1 is limited to 24-in and smaller pipe. For 30-in and larger pipe, laying condition Type 1 should not be used.

Asbestos-Cement Pipe. Asbestos-cement pipe is manufactured by mixing portland cement, with or without silica, and asbestos fiber. It is available in diameters from 4 to 42 in (10.2 to 106.7 cm) with 10 or 13 ft (3.0 or 4.0 m) lengths for larger pipe. Three classes are available, 100, 150, and 200, for pipe manufactured up through 16 in (40.6 cm) in diameter. Larger pipe is classified according to total designated pressure. A rubber gasket asbestos-cement coupling is normally used to join pipe lengths together. AWWA Standards C-400, C-401, C-402, and C-403 cover asbestos-cement pipe for distribution and transmission service.

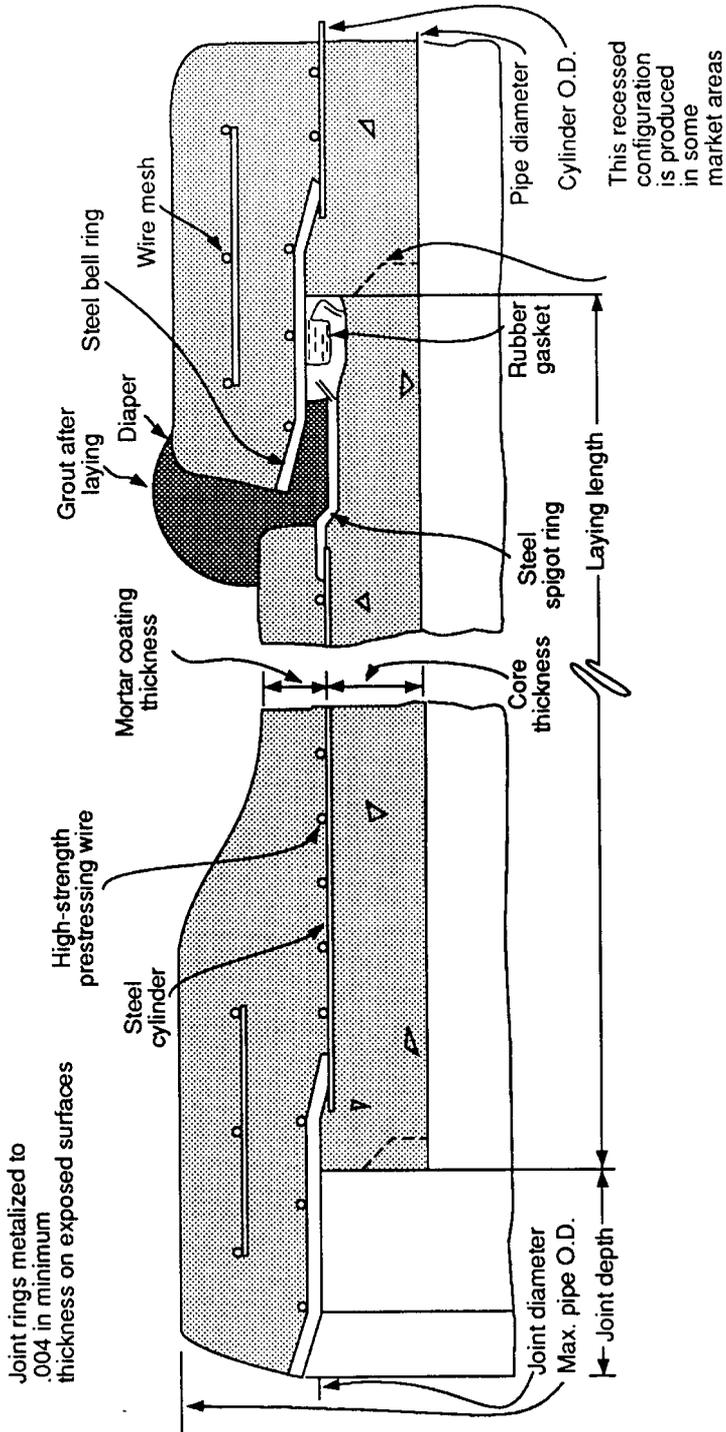


FIGURE 5.70 Lined cylinder pipe (109).

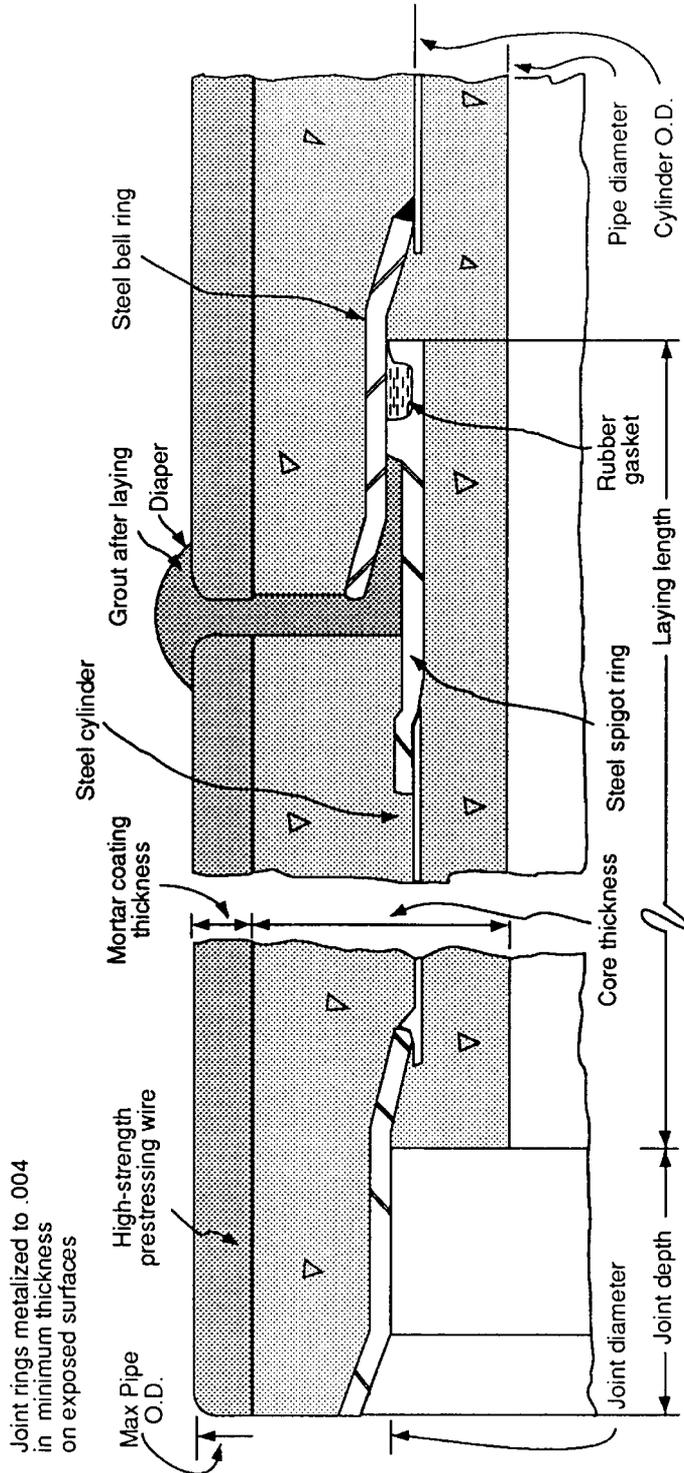


FIGURE 5.71 Embedded cylinder pipe (109).

Fittings. Gray and ductile cast iron fittings are available for iron, PVC, and asbestos-cement pipe. Fittings fabricated of steel are available for use with steel and prestressed concrete cylinder pipe and fiberglass reinforced plastic fittings are available for use with these types of pipe. PVC fittings are frequently used with iron pipe. Lightweight ductile iron fittings are becoming more popular in many areas. These fittings are available in 4 to 12 in (10.2 to 30.5 cm) diameter with mechanical joints, and are rated at 350 lb/in² (24.6 kgf/cm²) working pressures. AWWA C-10 covers gray and ductile iron fittings from 4 to 48 in (10.2 to 121.9 cm). Most fittings manufacturers supply catalogs containing dimensions and weights. Fittings are available for flanged, mechanical, and push-on joints in pressures ranging up to 350 lb/in² (24.6 kgf/cm²).

Installation. The manufacturer of the particular pipe material to be installed should be contacted for precise installation instructions. This section contains recommended installation practices which can be used for all types of pipe.

All pipe should be unloaded or moved with care and not dropped. Equipment should not be allowed to drive over or otherwise strike the pipe. When moving pipe, slings, forklifts, or special pipe tongs should be used. No pipe should be pushed along the ground or over pavement.

Trenches for water mains should be straight and at the proper depth. Pipe should be buried to a depth below the frost line in northern climates, and in warmer climates at a depth sufficient to cushion the pipe against traffic loads. Generally, 4 to 5 ft (1.2 to 1.5 m) of cover is used for large mains and 2.5 to 3 ft (0.76 to 0.91 m) for smaller mains. In some northern states, the mains may have as much as 8 ft (2.4 m) of cover over them due to frost penetration. Figure 5.72 shows maximum depth of frost penetration in the United States (112).

The trench should be as narrow as permissible and still be wide enough to provide for proper joining and compaction around the pipe. The suggested trench width is the nominal pipe diameter plus 2 ft (0.6 in). For deep trenches sharing a bank, sloping may be necessary to keep the bank from caving in. Trench bottoms should be undercut 6 to 9 in (15.2 to 22.9 cm) and sand, clean fill, or crushed stone installed to provide a cushion against the rock.

The pipe should be lowered into the trench, not rolled or dropped. Joining of the pipe should be in accordance with the pipe manufacturer's instructions. For all pipe, the bell should be clean and gaskets should be checked for cuts and correct positioning inside the bell. If mechanical joints are used, the follower glands should be advanced toward the bell evenly and the proper torque applied to the bolts. The bottom bolts especially should be checked, as workers tend not to properly tighten these nuts. When push-on joints are used, the joint should be made with the pipe being straight and, if deflection is necessary, the pipe can be moved after joining. This avoids possible damage to and unseating of the gasket.

Clean backfill should be used to fill the trench and should not contain large rocks, organic material, frozen material, refuse, or cinders. The backfill should be free of rocks larger than 8 in (20.3 cm) in their greatest dimension around the pipe and at least 1 ft (0.3 m) above the top of the pipe. It is important to ensure that the backfill is compacted under the pipe and preferably to the spring line of the pipe, or as instructed by the engineer. Any compaction above the pipe is done solely for the benefit of the surface above. Construction equipment should not be driven in the trench over the pipe for compaction purposes. This causes unnecessarily high stress in the empty pipe wall, which may result in damage.

Whenever the water main changes direction, ends, or changes size, thrust forces are created. These forces can be substantial. Thrust blocks, restrained joints, and tie rods are used to restrain thrusts. Table 5.52 shows the thrust that must be restrained by the thrust blocks. Thrust blocks are used to reduce the thrust pressure against the soil and to transmit this pressure from the fitting to the soil. Figure 5.73 illustrates thrust block location for various situations (113). Thrust blocks are usually made of at least 2000-lb (907-kg) concrete and stress should not be put on the block for at least 5 days after pouring. They are normally poured against undisturbed earth. Frequently, precast thrust blocks are used for pipe up through 8 in (20.3 cm) in diameter. When these blocks are used, the soil behind the blocks should be compacted to density. Soil cement may be used to assist in obtaining the required density. Steel tie rods and patented restrained joints are also used to

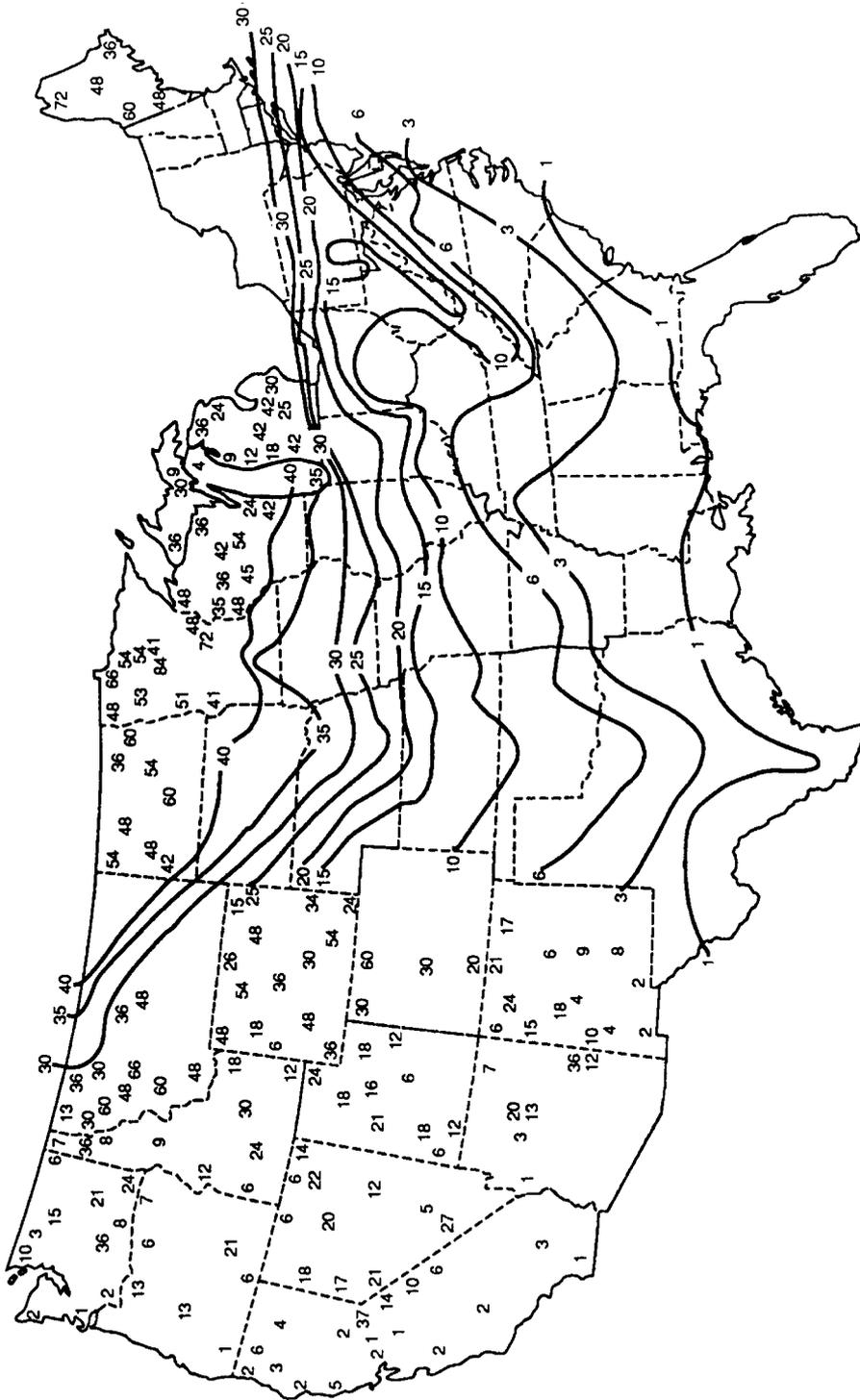


FIGURE 5.72 Frost penetration chart. Contours represent average depth of frost penetration in inches where data permit generalization. Elsewhere depths at individual stations are shown (based on 40 years of record) (1/12).

TABLE 5.52 Resultant Thrust at Fittings at 100-lb/in² Water Pressure

Nom. pipe dia., in	Total pounds				
	Dead end	90° bend	45° bend	22½° bend	11¼° bend
4	1,810	2,559	1,385	706	355
6	3,739	5,288	2,862	1,459	733
8	6,433	9,097	4,923	2,510	1,261
10	9,677	13,685	7,406	3,776	1,897
12	13,685	19,353	10,474	5,340	2,683
14	18,385	26,001	14,072	7,174	3,604
16	23,779	33,628	18,199	9,278	4,661
18	29,865	42,235	22,858	11,653	5,855
20	36,644	51,822	28,046	14,298	7,183
24	52,279	73,934	40,013	20,398	10,249
30	80,425	113,738	61,554	31,380	15,766
36	115,209	162,931	88,177	44,952	22,585
42	155,528	219,950	119,036	60,684	30,489
48	202,683	286,637	155,127	79,083	39,733
54	256,072	362,140	195,989	99,914	50,199

Note: To determine thrust at pressures other than 100 lb/in², multiply the thrust obtained in the table by the ratio of the pressure to 100. For example, the thrust on a 12-in, 90° bend at 125 lb/in² is

$$19,353 \times \frac{125}{100} = 24,191 \text{ lb}$$

To convert from lb/in² to kgf/cm², multiply by 0.0703; to convert from pounds to kilograms, multiply by 0.4535; to convert from inches to centimeters, multiply by 2.54.

restrain thrust. The length of pipe to be restrained on each side of the fitting depends on the thrust force, size, soil compaction along the pipe, depth of pipe, and type of soil. Figure 5.74 shows some patented restrained joints available (114).

Occasionally, it is necessary to install water mains above ground. When this is done, special attention should be given to the design of the pipe at the support. High localized stress can be reestablished which may cause the pipe to fail. Sad dies are normally provided for the pipe to rest on to reduce these high stresses. Some pipe, such as steel, will normally require a protective coating to prevent atmospheric corrosion.

Water mains may be installed underwater. When this occurs, ball-and-socket joint pipe is generally used. The pipe can be joined on the bank, and if the waterway is not too wide, it can be pulled into place. For larger crossings, the pipe can be assembled on a barge or floated into place and then sunk. Figure 5.75 shows a typical ball-and-socket joint (114).

All water mains should be tested for leakage and disinfected before being put in service. The water main is usually flushed after installation to remove any dirt or debris. Flushing velocities should be at least 2.5 ft/s (0.8 m/s) to remove debris. Test pressures are normally 1.5 times the system average pressure for a 2-h duration. Table 5.53 shows the allowable leakage suggested by DIIRA at various test pressures. This is one-half the leakage allowed in AWWA Standard C-600, the standard for the installation of cast iron pipe (113).

Mains are usually disinfected by the tablet method or slug method of chlorination. The tablet method is normally used for short extensions up to 2500 ft (762 m) of pipe, 12 in (30.5 cm) in diameter and smaller. For this method, the required number of hypochlorinate tablets is placed at the top of each joint of pipe. The pipe is then filled slowly, left for at least 24 h, and then flushed. Table 5.54 shows the number of tablets required for water mains up to 12 in (30.5 cm) in diameter.

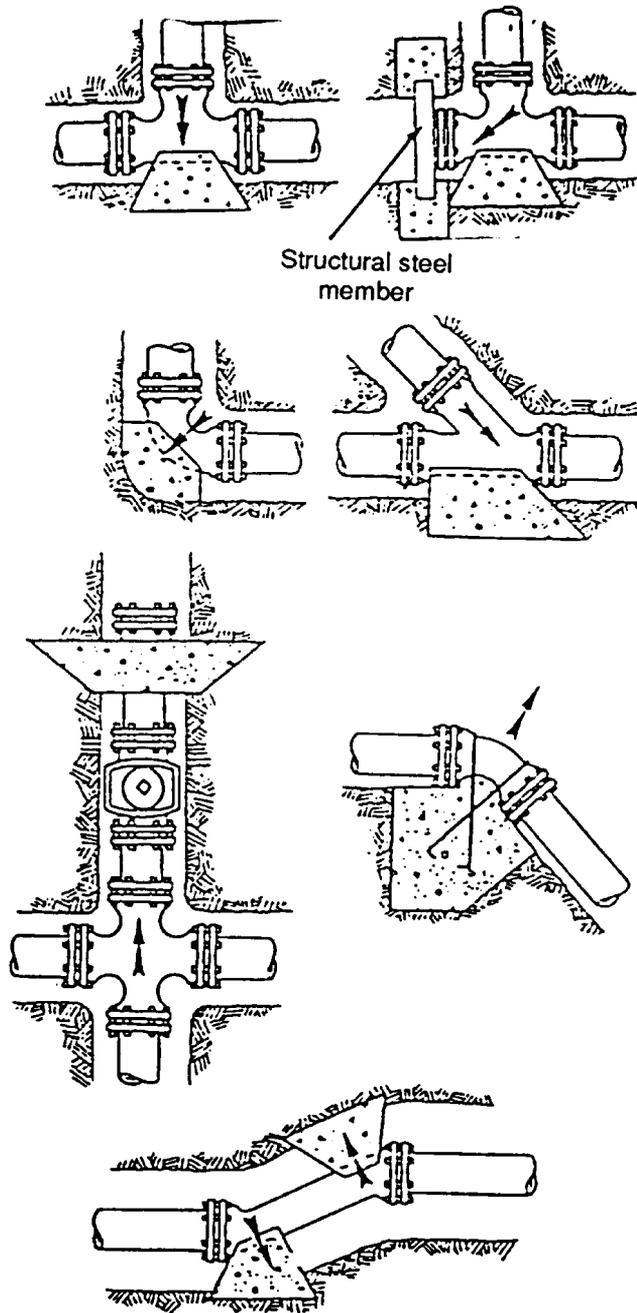


FIGURE 5.73 Thrust block location (113).

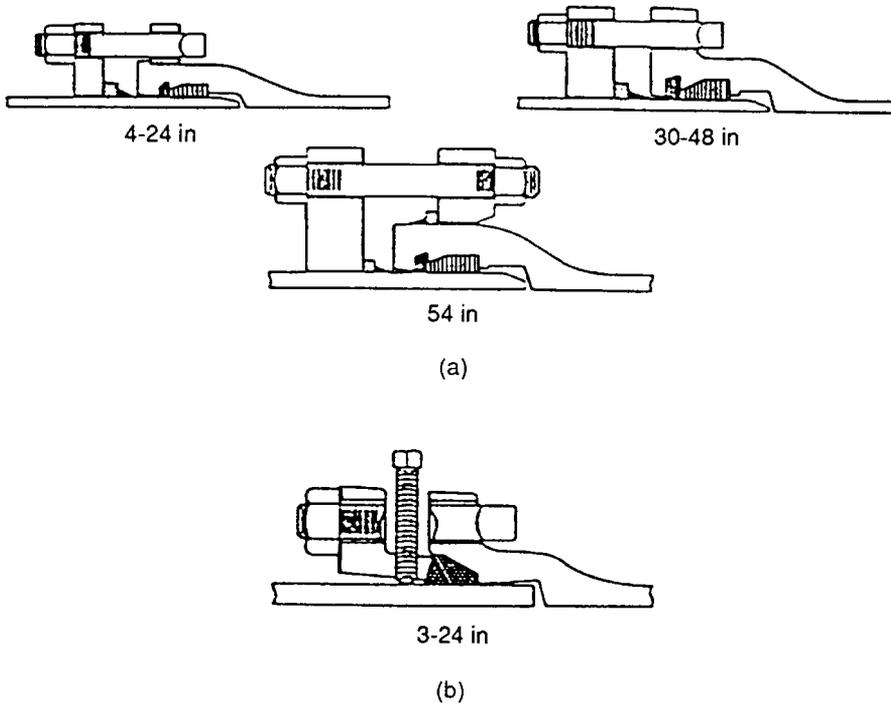


FIGURE 5.74 Restrainted joint pipe. (a) Lok-Fast joint; (b) mechanical joint with retainer gland (114).

The slug method is used for larger-diameter pipe and in areas where it is not desirable to waste large quantities of water. This method consists of moving a slug of highly concentrated chlorine (at least 300 mg/L) along the interior of the pipe. The contact time of the slug with the pipe wall should be at least 3 h.

Bacteriological tests should always be made of the disinfected main, and it should not be put into service until proper test results are obtained. AWWA Standard C-601 covers disinfection of water mains.

Repair and Maintenance. Broken or leaking water mains should be repaired as soon as possible to minimize property damage and loss of water. Circular breaks are repaired with pipe clamps. Split pipe usually requires that the split section be removed and a new section installed by sleeves in its place. Leaking joints and bells can be repaired by using special clamps which can fit over the entire pipe bell. It is important that when

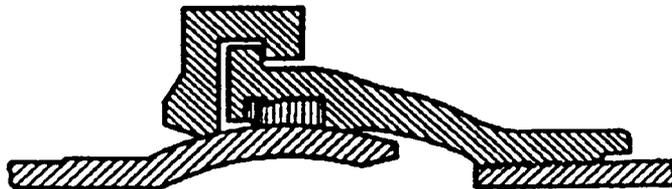


FIGURE 5.75 Flex-Lok boltless ball-joint pipe (114).

TABLE 5.53 Allowable Leakage (gal/h) per 1000 ft of Pipeline*

Avg. test pressure, lb/in ²	Nominal pipe diameter, in																
	2	3	4	6	8	10	12	14	16	18	20	24	30	36	42	48	54
450	0.32	0.48	0.64	0.95	1.27	1.59	1.91	2.23	2.55	2.87	3.18	3.82	4.78	5.73	6.69	7.64	8.60
400	0.30	0.45	0.60	0.90	1.20	1.50	1.80	2.10	2.40	2.70	3.00	3.60	4.50	5.41	6.31	7.21	8.11
350	0.28	0.42	0.56	0.84	1.12	1.40	1.69	1.97	2.25	2.53	2.81	3.37	4.21	5.06	5.90	6.74	7.58
300	0.26	0.39	0.52	0.78	1.04	1.30	1.56	1.82	2.08	2.34	2.60	3.12	3.90	4.68	5.46	6.24	7.02
275	0.25	0.37	0.50	0.75	1.00	1.24	1.49	1.74	1.99	2.24	2.49	2.99	3.73	4.48	5.23	5.98	6.72
250	0.24	0.36	0.47	0.71	0.95	1.19	1.42	1.66	1.90	2.14	2.37	2.85	3.56	4.27	4.99	5.70	6.41
225	0.23	0.34	0.45	0.68	0.90	1.13	1.35	1.58	1.80	2.03	2.25	2.70	3.38	4.05	4.73	5.41	6.03
200	0.21	0.32	0.43	0.64	0.85	1.06	1.28	1.48	1.70	1.91	2.12	2.55	3.19	3.82	4.46	5.09	5.73
175	0.20	0.30	0.40	0.59	0.80	0.99	1.19	1.39	1.59	1.79	1.98	2.38	2.98	3.58	4.17	4.77	5.36
150	0.19	0.28	0.37	0.55	0.74	0.92	1.10	1.29	1.47	1.66	1.84	2.21	2.76	3.31	3.86	4.41	4.97
125	0.17	0.25	0.34	0.50	0.67	0.84	1.01	1.18	1.34	1.51	1.68	2.01	2.52	3.02	3.53	4.03	4.53
100	0.15	0.23	0.30	0.45	0.60	0.75	0.90	1.05	1.20	1.35	1.50	1.80	2.25	2.70	3.15	3.60	4.05

*For pipe with 18-ft nominal lengths. To obtain the recommended allowable leakage for pipe with 20-ft nominal lengths, multiply the leakage calculated from the table by 0.9. If the pipeline under test contains sections of various diameters, the allowable leakage will be the sum of the computed leakages for each size.

Note: To convert from feet to meters, multiply by 0.3048; to convert from lb/in² to kgf/cm² multiply by 0.0703; to convert from inches to centimeters, multiply by 2.54

TABLE 5.54 Number of 5-g Hypochlorite Tablets Required for Dose of 50 mg/L[†]

Length of section, ft	Diameter of pipe, in					
	2	4	6	8	10	12
13 or less	1	1	2	2	3	5
18	1	1	2	3	5	6
20	1	1	2	3	5	7
30	1	2	3	5	7	10
40	1	2	4	6	9	14

[†]Based on 3½-g available chlorine per tablet.

Note: To convert from feet to meters, multiply by 0.3048; to convert from inches to centimeters, multiply by 2.54

a section of pipe is sleeved in the line that it be disinfected and that the entire section be thoroughly flushed to prevent contamination. Care should be given to ensure that all of the air is removed from the main when refilling. This may require opening several hydrants in the vicinity of the break.

Pipe Rehabilitation

It is desirable to maintain the carrying capacity of the pipe in the distribution system as high as possible to provide the designed flow and keep pumping costs as low as possible. Capacity in the pipeline is lost, sometimes severely, due to precipitation of aluminum hydroxide, scaling, and tuberculation. It has been documented that some unlined cast iron pipes have lost 50% of their original capacity in less than 30 years. Figure 5.76 illustrates the loss in Hazen–Williams C coefficient for several systems in the United States (115).

Unlined cast iron and steel water pipes can be rehabilitated to near their original carrying capacity by cleaning and the application of a cement mortar lining. Unlined cast iron and steel pipe should be lined after cleaning, otherwise the carrying capacity will soon return to that prior to cleaning. Normally this will occur in less than two years. Pipes from nominal diameter of 4 in (10.2 cm) up through 6 in (15.2 cm) have been successfully cleaned and cement-lined. Of course, there are many factors other than economics that may dictate cleaning and lining smaller sizes. Lined pipe which has lost carrying capacity because of scaling and precipitation of aluminum hydroxide can be cleaned with relative ease (113).

Pipeline Cleaning. The two methods of cleaning commonly used are the plastic pig, which is inserted in the pipe, and dragged or hydraulically propelled scraper mechanisms. Figure 5.77 illustrates a typical main-cleaning operation (116). Figure 5.78 shows a pipe before and immediately after cleaning.

Plastic Pigs. Plastic pigs have been used in the United States for cleaning water mains since the late 1960s. These pigs have covers of polyurethane sponge covered partially with special tough plastic strips, which may be impregnated with special abrasive coatings as shown in Figure 5.79. The double-spiral or criss-cross pattern is the best for general water main cleaning use. Pigs are usually bullet-shaped for smaller sizes and the length is approximately twice the diameter. Larger pigs have filet ends with a means of attachment on the ends to facilitate insertion into the pipe. Pigs are available in sizes from 2 to 72 in (5.1 to 182.9 cm) in diameter.

The method of cleaning is simple. The pig is inserted into a slot segment of the pipe to be cleaned. This segment is then reinstalled in the pipeline using normal procedure. The pig should be 0.25 to 0.5 in (0.6 to 1.3 cm) larger in diameter than the pipe to be cleaned. Since the pig is flexible, it can be compressed for

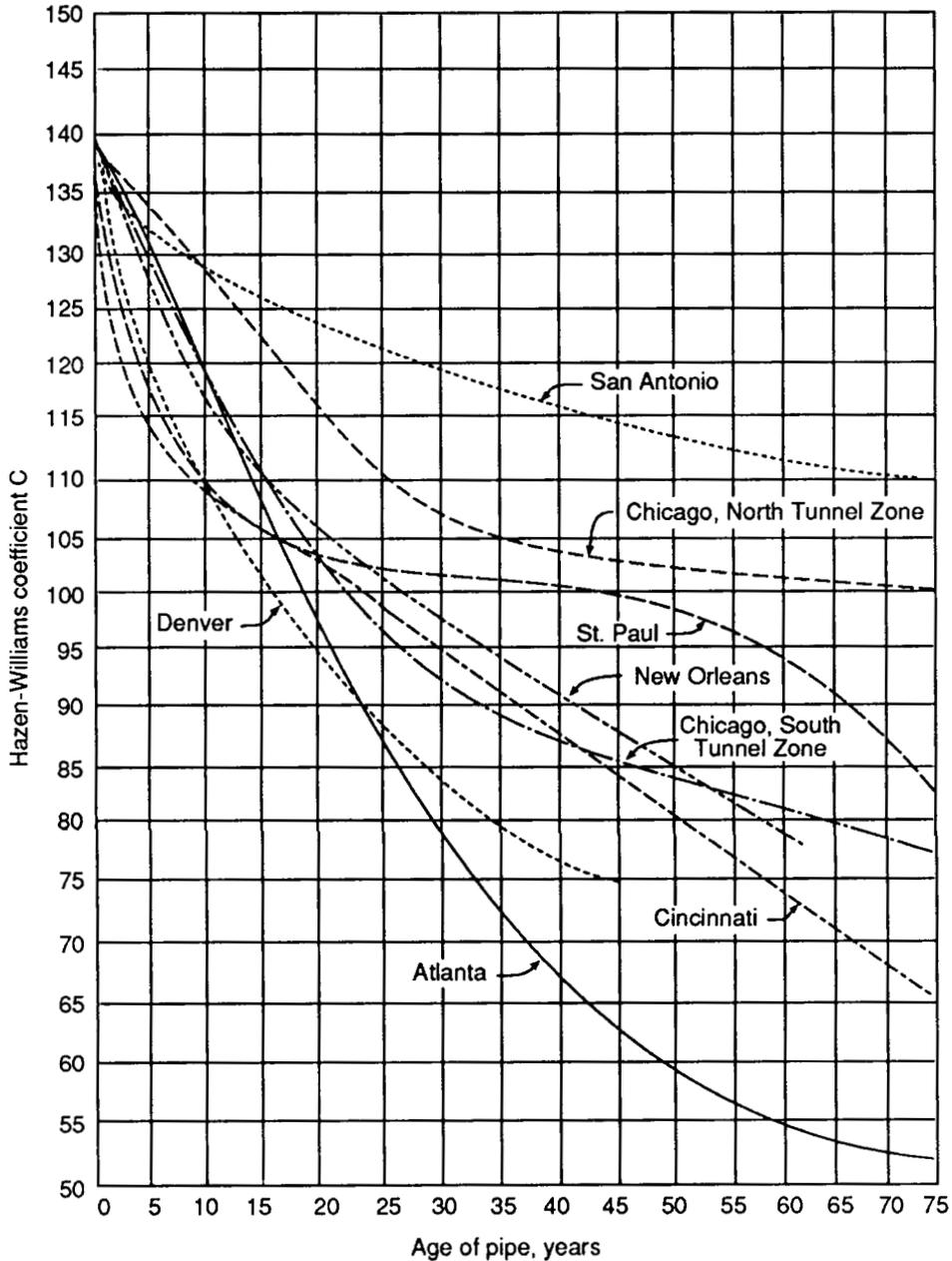


FIGURE 5.76 Trend curves of head loss tests. (Reprinted from *Journal of AWWA*, Vol. 56, No. 2 (February 1964) by permission. Copyright © 1964, The American Water Works Association.) (115)

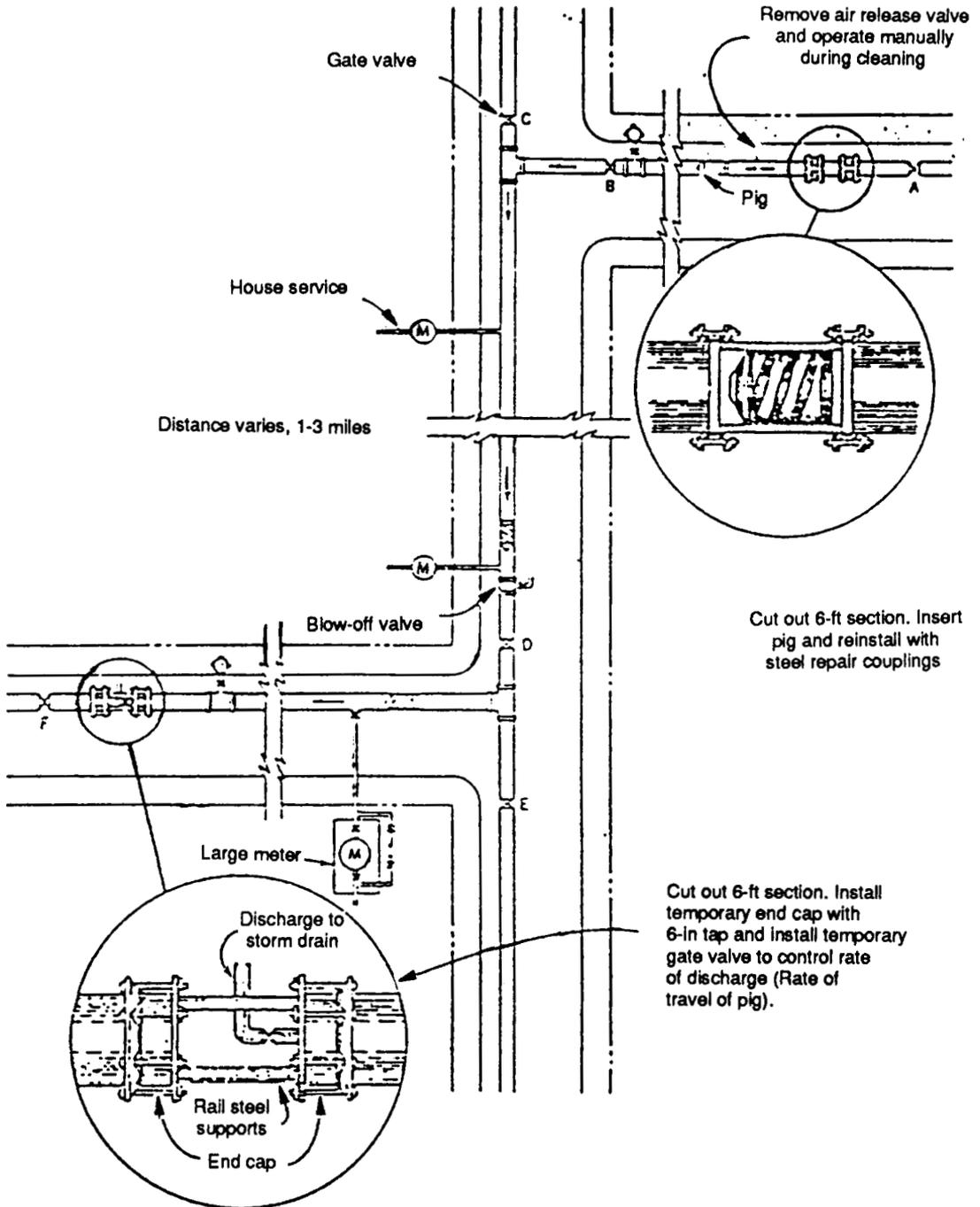
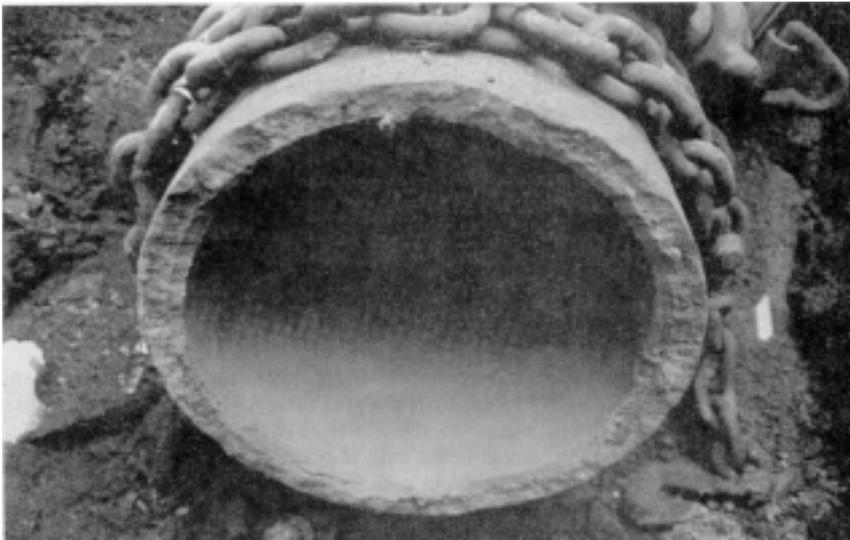


FIGURE 5.77 Typical main cleaning operation. (Reprinted from *Journal of AWWA*, Vol. 60, No. 8 (August 1968) by permission. Copyright © 1968, The American Water Works Association.) (116)



(a)



(b)

FIGURE 5.78 Results of effective pipe cleaning. (a) Before, (b) after.

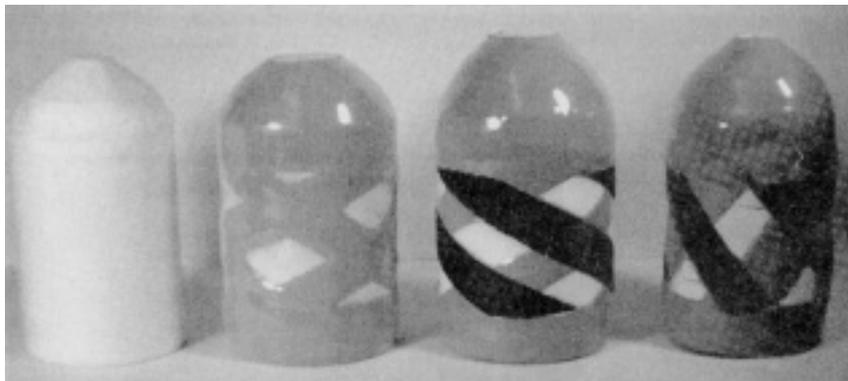


FIGURE 5.79 Plastic pig designs.

insertion into the pipe by winching. Pigs may be used singularly or in tandem in the pipeline. Optimum speed of travel through the pipeline is from 40 to 60 ft/min (12.2 to 18.3 in/min) and is accomplished by regulating discharge through hydrants. Often two passes of the pig through the pipe is sufficient to return the carrying capacity back to normal. Since pigs are flexible, they can travel through leaks and reduced section of pipe.

Pipe Scrapers. Pipe scraping mechanisms or pipe “scrapers” are used when there is serious scaling and tuberculation on the pipe and by specialized cleaning and lining contractors. Scrapers are covered with small springlike sweeper arms protruding from a series of disks (117). Hydraulically propelled scrapers are used in the same manner as plastic pigs for cleaning pipe. The drag scraper is pulled back and forth through sections of pipe to be cleaned by cables. Access is provided at approximately 500 to 800 ft (152.4 to 243.8 m) intervals to facilitate insertion or removal of the scrapers.

Sufficient passes are made by the scraper to insure that the pipe is sufficiently clean. Generally squeegees are attached to scrapers to remove debris dislodged by the drag scraper. For pipelines 30 in (72.6 cm) in diameter and larger, electrically driven rotating scrapers are used to remove scale and tuberculation. It is recommended that the pipe not be returned to service until it has been lined.

Cement Mortar Lining. Cleaned pipe can be lined in place with cement mortar using special equipment. AWWA Standard C602 covers the quality and thickness of the lining to be replaced in the pipe. Table 5.55 shows recommended thicknesses for lining pipeline in place. For pipe 24 in (61 cm) in diameter and smaller, lining is applied to the pipe wall by a spinning action of a comblike device that breaks the mortar and slings it against the pipe wall. The mortar is then finished.

Distribution System Flushing

Distribution systems are flushed periodically to improve water quality; this is often triggered by customer complaints. Some systems include complete drainage of water storage tanks annually as a part of flushing and cleaning of the distribution system. Flushing is achieved by high-velocity flow by venting from fire hydrants, and is planned to flow in the direction of normal flow. High-velocity drainage through sections that have dead ends and low flow rates under normal operation results in entrainment and expulsion of sediments in the pipes and improves the physical, biological, and chemical quality of water (169).

TABLE 5.55 Recommended Thickness of Cement Mortar Lining for Pipelines in Place

Type of pipe							
Old cast iron and ductile iron		Old steel		New cast iron and ductile iron		New steel	
Diameter, in	Nominal thickness of lining,* in	Diameter, in	Nominal thickness of lining,* in	Diameter, in	Nominal thickness of lining,* in	Diameter, in	Nominal thickness of lining,* in.
4–10	1/8	4–12	1/4	4–10	1/8	4–12	3/16
13–36	3/16	14–22	5/16	12–36	3/16	14–36	1/4
36+	1/4	24–60	3/8	36+	1/4	42–60	3/8

*For badly deteriorated pipe or abnormal use, the purchaser should specify such greater thickness of lining as engineering judgment indicates. In all instances, tolerance for lining thickness shall be plus 1/8 in with no minus tolerance.

Note: To convert from inches to centimeters, multiply by 2.54.

Valves

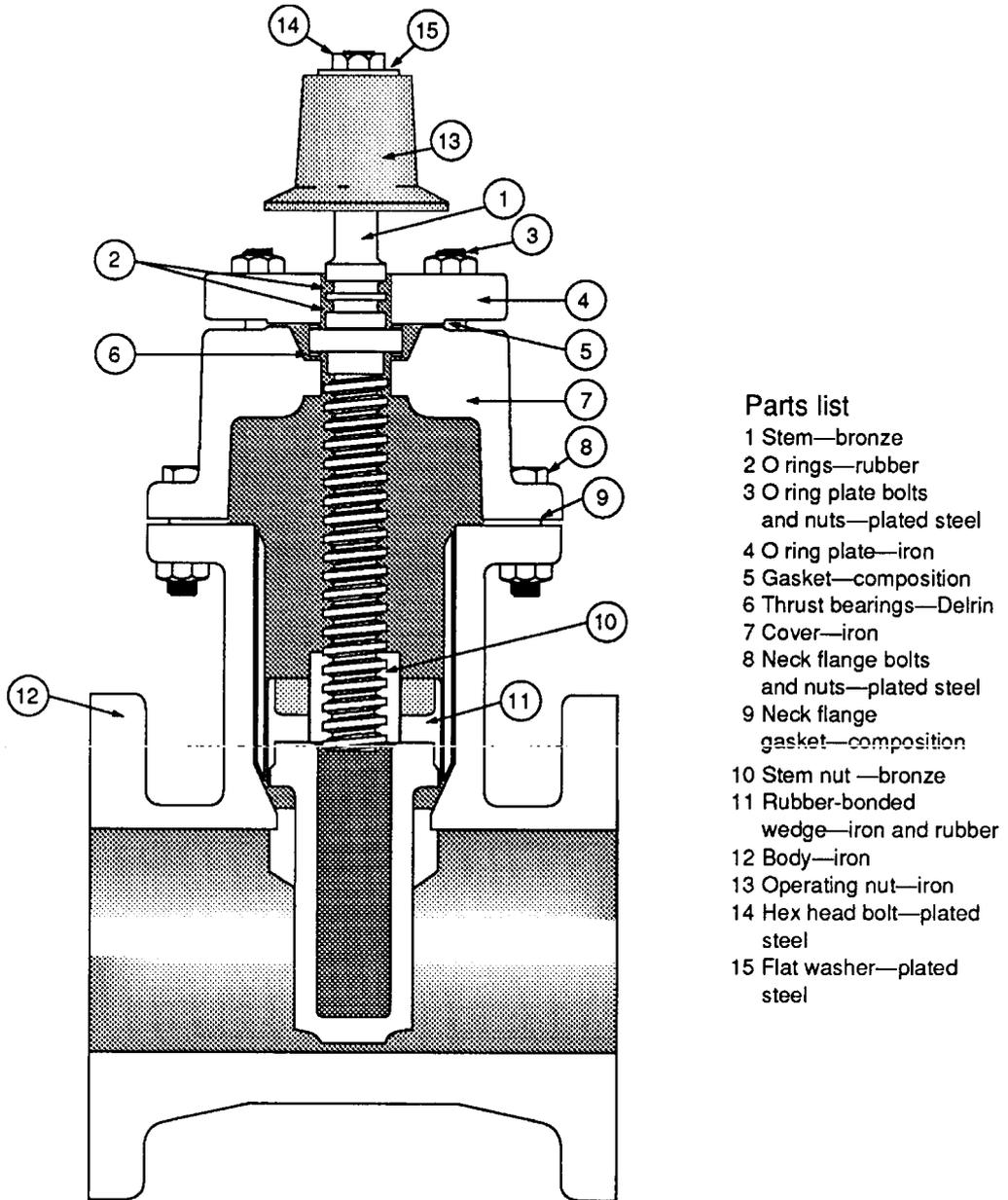
Valves are used in the distribution system to isolate sections for maintenance and repair. They should be located in the system so that the areas isolated will cause a minimum of inconvenience to the customer, and care should be used to ensure that only the number of valves necessary are installed. In the looped distribution grid, valves should be located so that the loop can be supplied from adjoining loops when one side of the loop is out of service. Large mains should be valved at least every mile to facilitate maintenance and to keep dewatering to a practical minimum. Valves should be installed on each side of obstacles such as subaqueous river crossing, bridges, limited-access highways, and at other areas where access is restricted.

Gate Valves. Gate valves are the most widely used valves for 30.5-cm (12-in) and smaller mains. They operate by blocking the water flow with a disk or wedge and operate in a perpendicular direction to the water flow by means of a threaded stem. When it is in the open position, the gate is positioned out of the waterway into another section of the valve housing called the bonnet.

Resilient seat gate valves are the most widely used today because of their simplicity and tight closure. The seating portion of the gate is covered with a resilient material, which, when the valve is in the closed position, presses firmly against the valve body causing a leak-free closure. Figure 5.80 shows a resilient-seat gate valve (112).

Use of the double-disk gate valve has decreased substantially since the resilient-seat gate valve was introduced. However, some are still supplied. This valve consists of two disks containing a bronze seat ring. One side of each disk, when in the closed position, makes contact with the bronze seat rings around the waterway in the body. When in the open position, the disks are collapsed and are located out of the waterway in the valve bonnet. When closing, the disks are moved into the waterway by means of a threaded stem and the last turn of the stem forces the disks apart to make contact with the valve body seat rings by means of wedging mechanisms.

Butterfly Valves. Butterfly valves are the most commonly used valve for 16-in (40.6-cm) and larger mains. This type of valve consists of a disk connected to a shaft which runs through the valve body to an operator. For closure, the disk is rotated perpendicular to the flow. Seating is achieved by resilient material either on the disk or on the valve body. The disk is rotated parallel with the water flow to fully open the valve. Because of the balanced forces on the disk, it can be operated with a large pressure differentials across the valve. This is particularly helpful when initially filling large mains or when mains must be emptied for pipe maintenance. Figure 5.81 illustrates a typical butterfly valve (112).



Parts list

- 1 Stem—bronze
- 2 O rings—rubber
- 3 O ring plate bolts and nuts—plated steel
- 4 O ring plate—iron
- 5 Gasket—composition
- 6 Thrust bearings—Delrin
- 7 Cover—iron
- 8 Neck flange bolts and nuts—plated steel
- 9 Neck flange gasket—composition
- 10 Stem nut —bronze
- 11 Rubber-bonded wedge—iron and rubber
- 12 Body—iron
- 13 Operating nut—iron
- 14 Hex head bolt—plated steel
- 15 Flat washer—plated steel

FIG 5.80 Resilient seat gate valves (112).

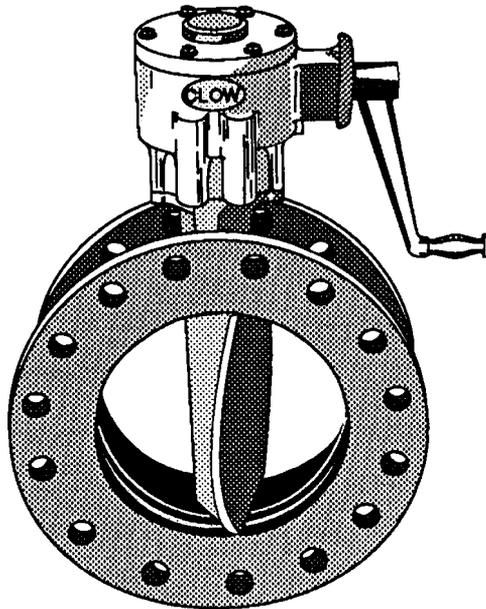


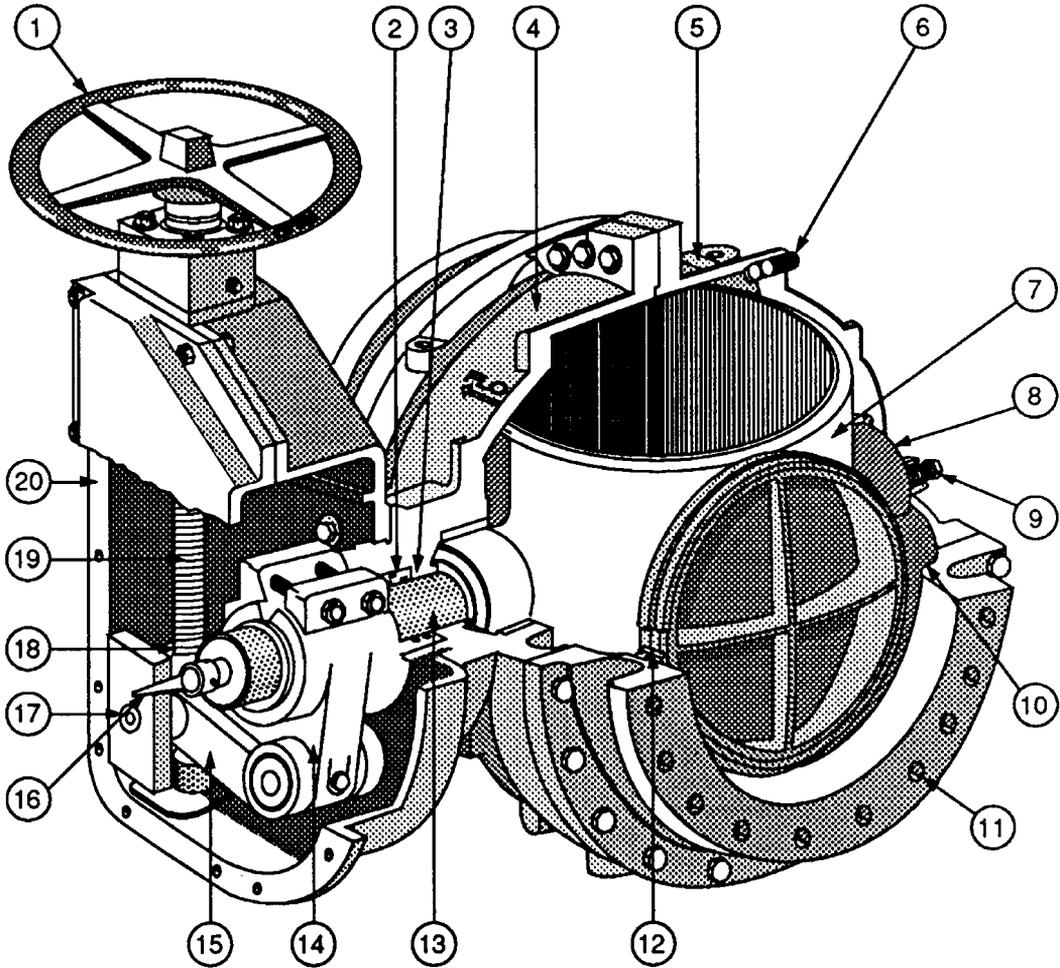
FIG 5.81 Butterfly valve (112).

Ball Valves and Cone Valves. Ball valves and cone valves are primarily used as pump discharge valves however they are occasionally used in the distribution system. They consist of a rotating element with a hole through it which locks into the valve body. When the hole in the element is aligned with parts in the valve body, the valve is in the open position. When the rotating element is turned 90°, the flow is stopped and allows for tight closure. Because of the low pressure losses across this valve, it can usually be smaller than the main to which it is attached, resulting in some monetary savings. Figure 5.82 shows a typical ball valve (118).

Altitude Valves. Altitude valves are special valves used to control the flow of water into ground and elevated storage tanks and standpipes. Double-acting altitude valves open when the pressure on the pumped side is lower than that on the tank side and closes automatically when the tank is full. Single-acting altitude valves open to permit flow into the tank and automatically close when the tank is full. Outflow is through a check valve in another pipe. The most popular designs use diaphragm and globe valve principles. Generally, these valves are actuated by water pressure. However, there are some designs that utilize solenoid valves. Variations of this basic design can be used for pressure-reducing or -sustaining valve applications. Figure 5.83 shows a typical altitude-valve installation from an elevated tank (119).

Air Release Valves. The development of air pockets in pipelines and appurtenances can severely inhibit the efficient flow of water through them. These pockets normally occur at summits in pipelines and appurtenances.

Air release valves are usually the most effective means of correcting air accumulation problems. There are many designs of air release valves. Most are derivatives of the floating ball and orifice principle—a ball in a chamber falls away from an orifice which allows air to escape around the ball until it floats and seals the orifice. The manufacturer should be consulted regarding valve sizing with reference to pipeline size, pres-



Parts list

- | | |
|------------------------------------|-----------------------------------|
| 1 Handwheel—cast iron | 11 Adaptor—cast iron |
| 2 Packing retainer—cast bronze | 12 Ball seat ring—stainless steel |
| 3 Thrust washers—cast bronze | 13 Operating shaft—forged steel |
| 4 Body—operator side—cast iron | 14 Lever—cast steel |
| 5 Body—trunnion side—cast iron | 15 Link—cast steel, bronze bush. |
| 6 Body vent plug—malleable iron | 16 Indicator—cast steel |
| 7 Ball—cast iron | 17 Sidemember—cast bronze |
| 8 Trunnion—bronze bushed cast iron | 18 Crosshead—cast bronze |
| 9 Body thrust screw—steel | 19 Leadscrew—stainless steel |
| 10 Adaptor seat—monel | 20 Housing—cast iron |

FIG 5.82 Basic ball valve (118).

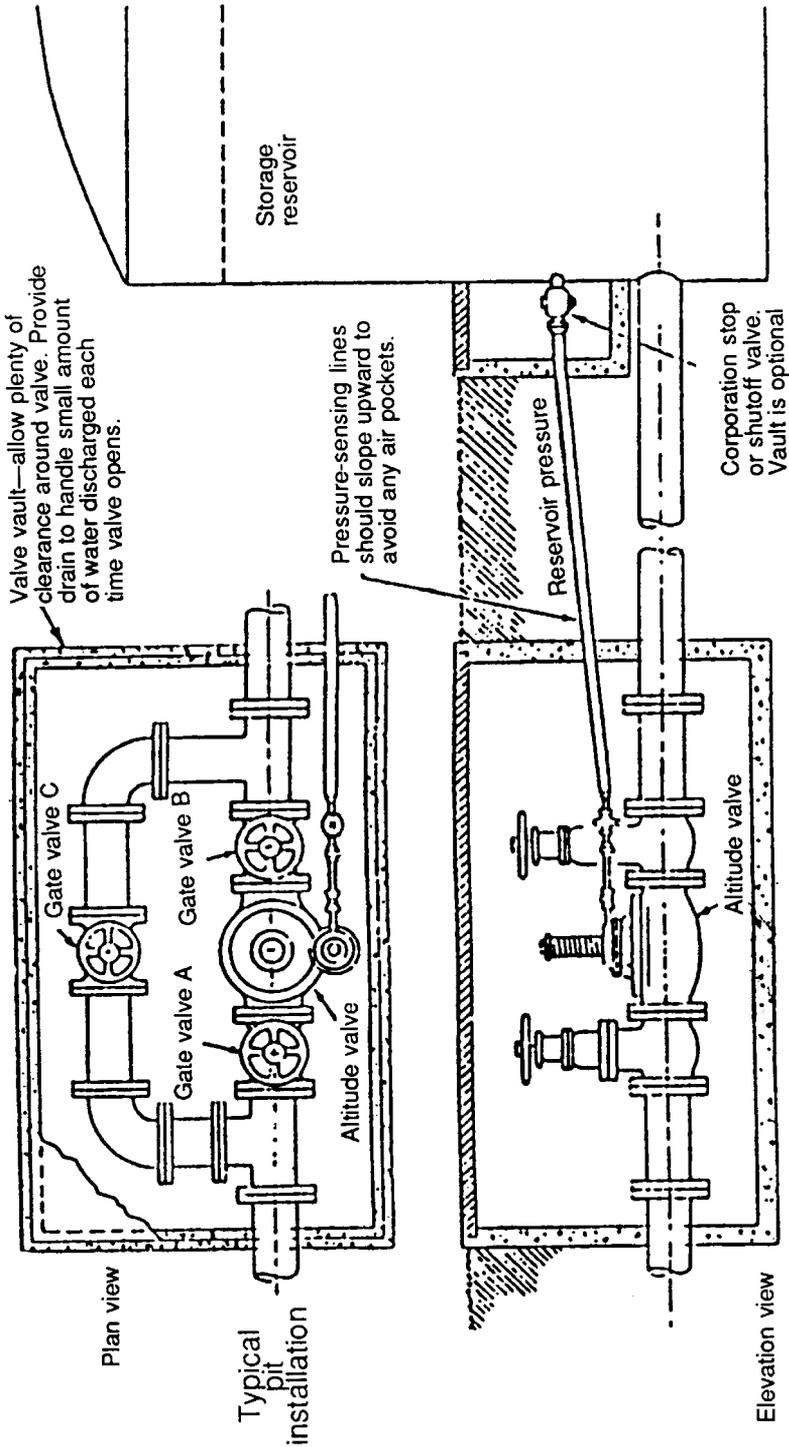


FIG 5.83 Altitude valve layout (119).

sure ratings, valve piping, and any special conditions. Placement of air release valves should be considered at the following points:

- At high points of pipelines
- At above-ground appurtenances when piping returns underground
- On pump discharge piping
- Immediately downstream from pressure-reducing valves
- Upstream from reducers, orifices, and other obstructions in the pipeline

Air and Vacuum Release Valves. Vacuums can occur in pipelines when they are being emptied. Large pipes with thin walls, such as steel pipe, have collapsed due to vacuums. Air and vacuum release valves have been developed to protect such pipelines. Manufacturers of air release valves usually manufacture air/vacuum release valves also, and they should be consulted regarding proper pressure rating and size (see Figure 5.84) (112).

Installation. Most gate and butterfly valves today are available for direct bury. Valves that require periodic maintenance and adjustments are normally installed in vaults. These include cone valves, altitude valves, air release valves, pressure-reducing valves, and other similar distribution valves.

Distribution valves should be placed so that they will remain accessible for operation and maintenance. This is particularly important for valves placed in vaults. Good records should be kept as to the exact location of the valve, and references for measurements should be permanent and visible. Good references for measurements include fire hydrants, benchmarks, concrete curbs, and special markers installed for this purpose. Often it is desirable to provide highly visible markers for valves which are located in areas of dense vegetation. In other areas where other underground utilities exist, it is usually advisable to color-code valve box taps for easy identification.

Vaults for valves should be large enough to provide adequate working space. Removable vault taps should be provided in case the entire valve should be replaced. A means for drainage should be provided; usually, a sloping floor with a sump is sufficient. Also, safety ladders should be provided for access to valves in deep vaults.

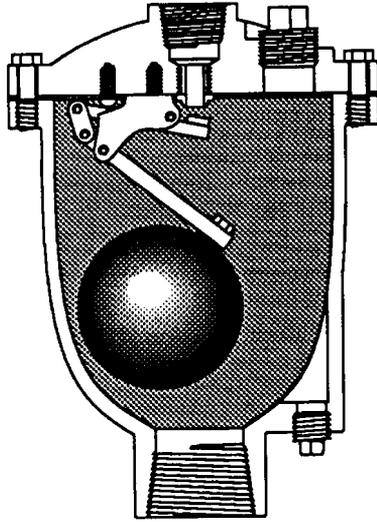
Operation and Maintenance. Most valves in a distribution system suffer from problems due to lack of operation rather than wearing out. A primary problem is sticking due to corrosion or scaling from the water flowing through the valve. Valves with large surface areas (such as horizontal gates, cone, and ball valves) which make contact with each other are particularly vulnerable to these problems.

A good valve maintenance program should include operation of large valves, over 16 in (40.6 cm), at least annually and preferably at six-month intervals. Smaller valves should be operated at least once every two years, depending on the tendency of the water to corrode or scale. Valve operations in vaults should be inspected when the valve is operated and lubricated as required. Unfortunately, there is a tendency to apply too much torque when operating valves that have not been properly maintained, usually resulting in broken stems and gears. Care should be taken to limit the torque applied and to operate the valve only a few turns open and closed at a time until it is operating freely.

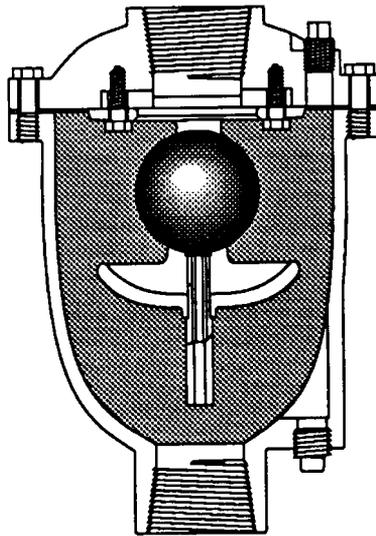
Altitude and air release valves should be inspected and maintained at intervals depending on service and environment—normally, at least semiannually. These intervals should be at least every six months.

Fire Hydrants

There are two types of fire hydrants—dry barrel and wet barrel. Dry-barrel hydrants have the main valve at the bottom of the barrel, whereas wet-barrel hydrants have valves at each hydrant outlet. AWWA Standard C502 covers dry-barrel fire hydrants and Standard C503 covers wet-barrel fire hydrants.



(a)



(b)

FIG 5.84 Air release and air and vacuum valves (112).

Flush-top hydrants are specially designed to not protrude above the ground or pavement. The operating nut and outlets are located in a cast iron box below ground level. The box has an easily removed cover for quick access. A variety of outlets are available; however, the most common are two-hose and one-pumper outlet configuration. These hydrants are normally placed where other types of hydrants may be aesthetically unpleasing or in areas subject to vehicular abuse.

Dry-Barrel Hydrants. The most common of the two types of hydrants is the dry barrel. Figure 5.85 shows a typical dry-barrel hydrant (112). It is used extensively in the United States in both cold and warm climates. Its primary advantage in cold climates is that water can drain or be pumped from the barrel, which eliminates damage due to freezing. These are normally supplied with two 2.5-in (6.4cm) hose outlets and one 4.5-in (11.4-cm) pumper suction outlet. However, other combinations are available. Commonly, hydrants are available with 6-in (15.2-cm) connections. However, other sizes are available for special situations.

There are four types of main valves:

1. Upward closing against a horizontal seat
2. Downward closing against a horizontal seat
3. Horizontal closing against a vertical seat
4. Vertically positioned gate valve

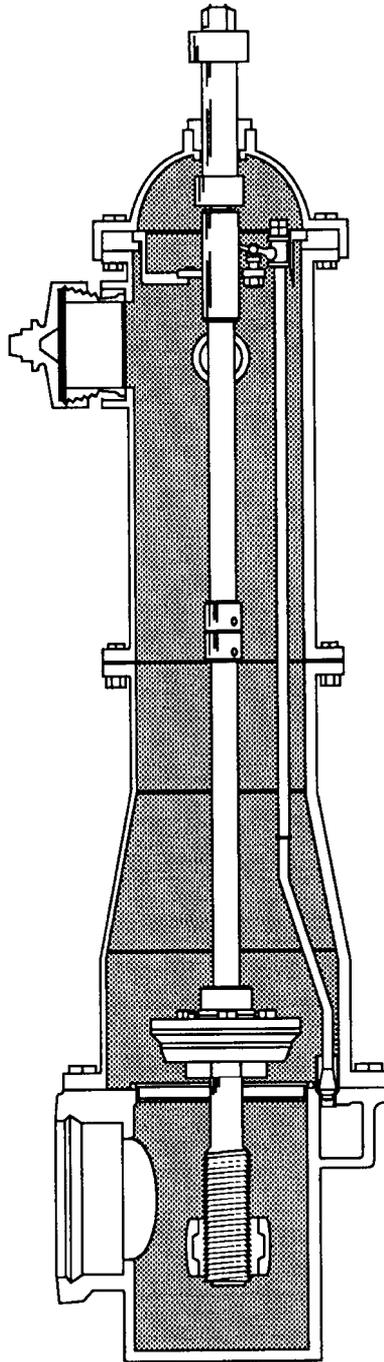
The traffic model of the dry-barrel hydrant is most popular because of the breakaway features of the head and valve stem, which minimize or eliminate damage to the main valve. This design consists of either breakaway bolts or breakaway ground flanges which connect the head to the hydrant barrel. There is also a breakaway valve-stem coupling located at the ground line. Repair kits are available and repairs can be made normally without closing off the water supply in the main serving the hydrant.

Wet-Barrel Hydrants. Wet-barrel hydrants, often referred to as California or West Coast hydrants, can only be used in areas that are free from freezing. A variety of outlets are available; however, the most common is the two-hose and one-pumper outlet configuration. Figure 5.86 shows a typical wet-barrel hydrant.

Installation. Fire hydrants should be installed in areas that are easily accessible by the fire department and yet do not function as obstacles for pedestrians and vehicles. They should be placed at least 2 ft (0.6 m) but no more than 6 ft (1.8 m) from a paved road and at sufficient distance from fixed objects to avoid interference in hydrant operation or repair; this distance is usually 3 ft (0.9 m). The proper depth to bury a hydrant should be selected so that the ground flange extends 1 or 2 in (2.5 or 5 cm) above the ground or pavement surface. This will provide the proper height for using hydrant wrenches to open the pumper outlet and provide space should bolts at the ground flange need to be removed.

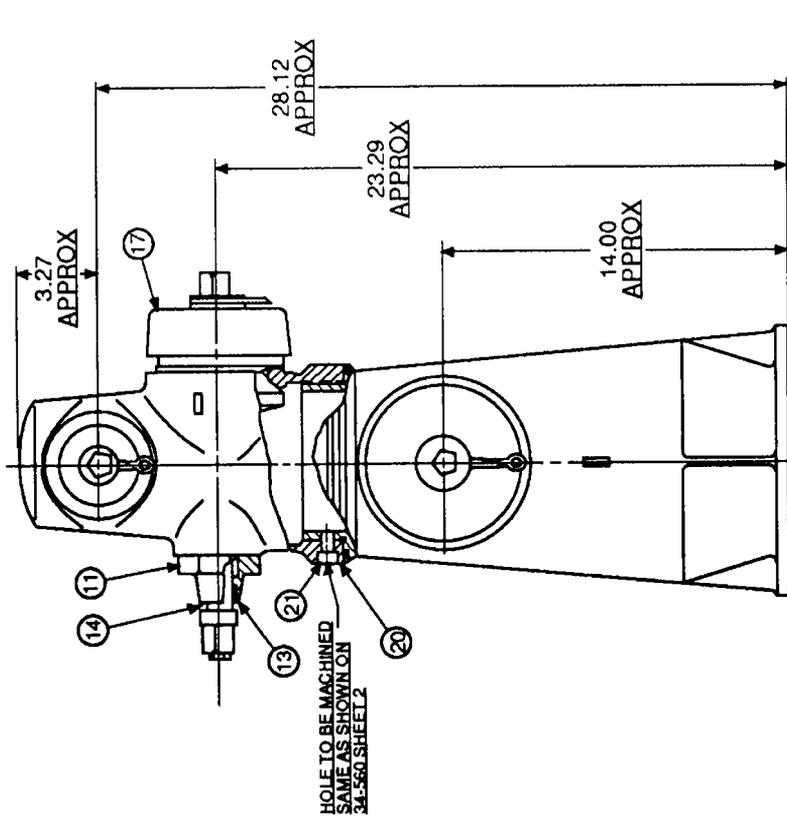
In areas where hydrants are subject to traffic damage, “hydrant guards” are installed to protect the hydrant. These are usually pipes filled with concrete or posts buried in the ground and protruding above ground to the same height as the hydrant. These guards are strategically placed to intercept vehicles before the hydrant is hit. However, they should be placed at sufficient distance from the hydrant so as not to interfere with its operation and maintenance.

It is good practice to place valves on hydrant lead pipes to provide for isolation for repair and maintenance purposes. These valves should be placed near the water main and away from the hydrant so that flow can be stopped easily in case of an accident. Thrust restraint at hydrants should be properly placed so that the hydrant and hydrant lead valves are restrained and not moved. This is usually accomplished by the use of thrust blocks, restrained joints, tie rods, or a combination of these methods. Concrete blocks should be placed so that the hydrant base will rest on them to prevent settlement, and in tight clay soils gravel should be placed around the drain port to allow for proper hydrant drainage if state health officials permit unplugged hydrant drains. Figure 5.87 shows a typical fire hydrant installation (121).



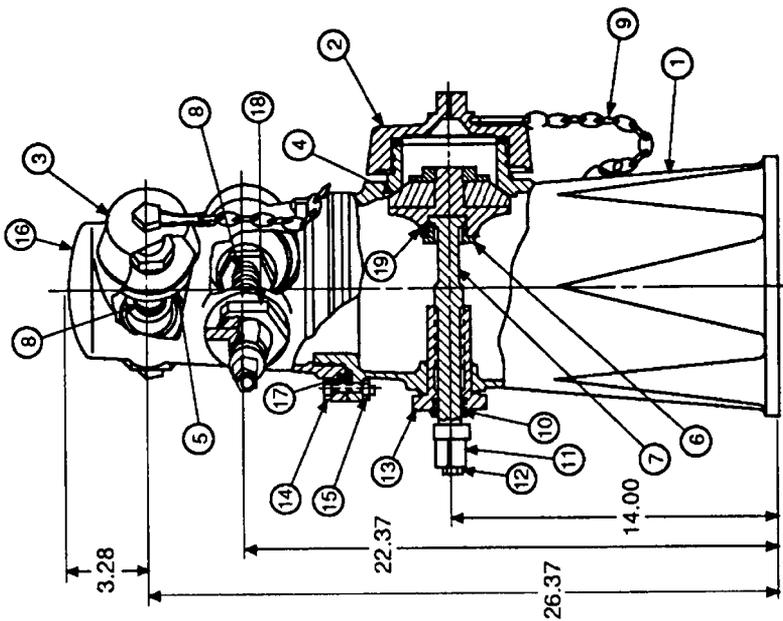
Open position

FIG 5.85 Dry-barrel fire hydrants (112).



(Eight hole flange pattern)

- | | | | |
|----|----------------------|----|--------------------------|
| 1 | Body 4" outlet | 12 | Insert |
| 2 | Hose cap | 13 | O ring |
| 3 | Disc holder assy. 4" | 14 | Stem 4" |
| 4 | Stem locknut | 15 | Stem 2-1/2" |
| 5 | Insert | 16 | Disc holder assy. 2-1/2" |
| 6 | Stem | 17 | Hose cap assy. 4" |
| 7 | Nut-operating | 18 | Hose cap assy. 2-1/2" |
| 8 | Nut | 19 | Chain |
| 9 | O ring | 20 | Stat-O-Seal |
| 10 | Body | 21 | Screw Locking |
| 11 | Insert | | |

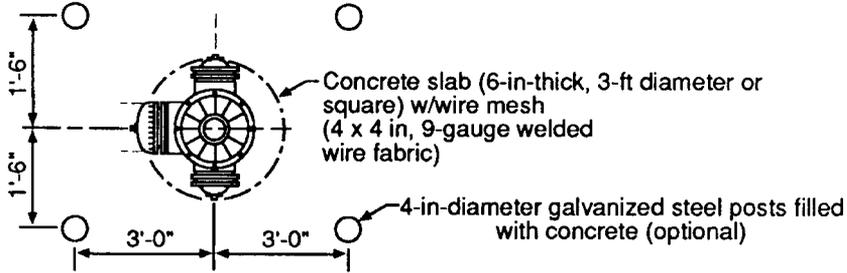


(Six hole flange pattern)

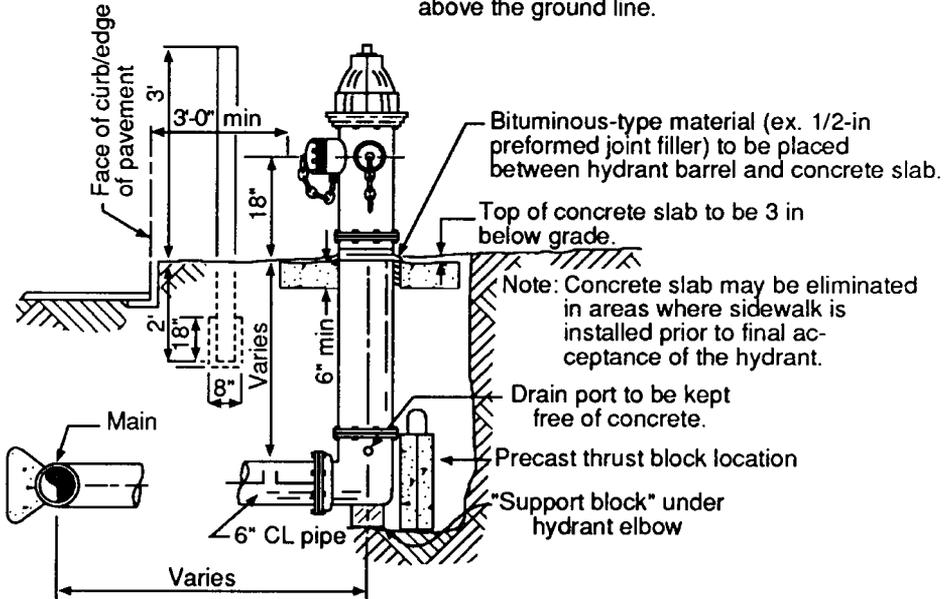
- | | | | |
|----|--------------------------|----|---------------|
| 1 | Body 4" outlet | 11 | Nut-operating |
| 2 | Hose cap 4" | 12 | Nut-stem |
| 3 | Hose cap 2-1/2" | 13 | Insert |
| 4 | Disc holder assy. 4" | 14 | Bolt |
| 5 | Disc holder assy. 2-1/2" | 15 | Nut |
| 6 | Stem locknut | 16 | Body |
| 7 | Stem | 17 | O ring |
| 8 | Stem | 18 | insert |
| 9 | Chain | 19 | Set screw |
| 10 | O ring | | |

FIG 5.86 Wet-barrel fire hydrant (1/20).

WATER SUPPLY



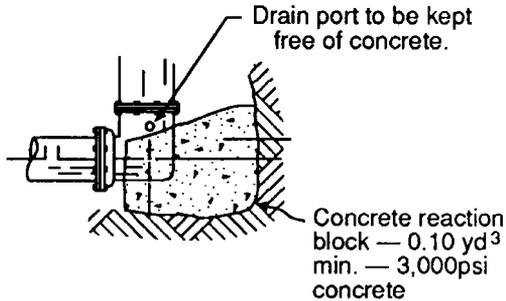
Note: All fire hydrants shall be painted with a high-grade enamel, federal safety yellow (OSHA approved) above the ground line.



Method 2-Concrete Reaction Block

Precast thrust block for hydrant restraint:

1. Hydrant shall be firmly supported under elbow.
2. All backfill shall be thoroughly compacted behind thrust block.
3. Use of precast thrust block shall be approved by Tampa Water Department prior to installation.



Method 1-Concrete Reaction Block

FIG 5.87 Fire hydrant installation. (Courtesy of Tampa Water Department.)

Inspection and Maintenance. The AWWA Manual M-17 recommends that hydrants be inspected annually and in freezing climates should receive inspections in the fall and again in the spring. Required maintenance should be performed at the time of inspection to minimize duplication and reduce expenses. The following procedure is recommended by AWWA for dry- or wet-barrel hydrants.

Inspection Procedure. Inspection procedures include the following:

1. Outlet nozzle cap should be removed and checked with a plumb bob for presence of water or ice standing in barrel of dry-barrel hydrants.
2. Seats should be checked for leakage with a listening device on dry-barrel hydrants and visually at each valve on wet-barrel hydrants.
3. Outlet nozzle caps should be replaced and the hydrant opened to full open position, checking ease of operation. Wet-barrel hydrants require that each valve be operated, using a special test outlet-nozzle cap. If stem action is tight, operation should be repeated several times until opening and closing action is smooth and free.
4. Water conditions may cause “hard water buildup” on steam threads. A series of opening and closing operations usually is sufficient to remove this buildup.
5. While the hydrant is under pressure, joints should be checked for leakage around outlet nozzles, at packing or seals, and past outlet nozzle caps.
6. If leakage is observed, outlet nozzles should be tightened or recaulked, compression packing lubricated and tightened, and O rings or similar seals and gaskets replaced. If leakage cannot be corrected with the tools at hand, the nature of the leakage should be recorded for prompt attention.
7. On dry-barrel hydrants, the main valves should be closed to the position at which the drains open and flow should be allowed through the drains under pressure for about 10 s to flush the drains. Then they should be closed completely.
8. Nozzle caps should be removed and a section of hose attached, if necessary, to direct the flow into the street. The hydrant should be opened and flushed to remove foreign material from the interior and lateral piping.
9. Main valve should be closed and the dry-barrel hydrants checked for drainage from the barrel. Drainage should be sufficiently rapid to create a suction if the hand is placed over a nozzle outlet during drainage. Seat leakage should be checked again with a listening device on dry-barrel hydrants and visually on wet-barrel hydrants.
10. If dry-barrel hydrants do not have a drain, any residual water in the barrel should be pumped out and the hydrant checked for leakage of the main valve with a listening device.
11. Check nozzle cap chains for free action on each cap. If binding is observed, open the loop around the cap until the action is free so as to prevent kinking during removal of the cap under emergency conditions.
12. Caps should be replaced, tightened with the hydrant wrench, then backed off slightly so they will not be excessively tight; they should be left tight enough to prevent removal by hand.
13. Operating-nut threads should be lubricated in accordance with manufacturer’s instructions.
14. On hydrants designed to fail at the ground-line joint on vehicle impact (traffic hydrants), the breakaway device should be checked. Couplings, cast lugs, special bolts, etc., should be checked for damage due to impact or corrosion.
15. Check for, and clear if necessary, any exterior obstruction that may interfere with hydrant use in a fire emergency.
16. The exterior of the hydrant should be cleaned and repainted if necessary.
17. The auxiliary valve should be in the full open position.
18. If a hydrant is inoperable, it should be tagged with clearly visible marking to prevent loss of time by fire-fighting crews if any emergency occurs before the hydrant is repaired. The condition should be reported to the fire department at once.

Flow Testing. AWWA Manual M-17 recommends the following procedure for flow testing of hydrants.

1. A hydrant at or near the location for which flow-test data are required should be selected; this will be the flowing hydrant. Another hydrant should be selected for the residual pressure readings, or a hose bibb or faucet should be located at which the readings are taken. If residual pressure is read at a hose bibb or faucet, care should be taken that there is no use of water in the building while the test is being performed.
2. At the start of the test, the normal system pressure should be observed and recorded. Then the selected hydrant is opened to allow flow. While the hydrant is flowing, the discharge pressure is determined by the use of a pitot gage, and that pressure recorded. Residual pressure should be observed and recorded.
3. Data reflecting the flow capabilities of the system under the pressure drop induced by the test can be converted to information regarding flow capabilities under a greater pressure drop (as would be induced by fire pumping) by the use of Tables 5.56, 5.57, and 5.58, as demonstrated in the following example.

A hydrant test gives the following results. At the location where residual pressure is to be measured, pressure before the hydrant is open at 65 lb/in² (4.6 kgf/cm²). While the hydrant is flowing, residual pressure is 40 lb/in² (2.8 kgf/cm²). At the flowing hydrant, a pitot reading of 27 lb/in² (1.9 kgf/cm²) is observed in the flow from a 2.5-in (6.4-cm) outlet.

Find the flow that would be produced if the residual pressure in the system were drawn down to 21 psi (1.5 kgf/cm²).

First, use Table 5.56 to determine that the flow rate from a 2.5-in (6.4-cm) outlet at 27 lb/in² (1.9 kgf/cm²) is 870 gpm (3.3 m³/min). (Tables relating a greater range of outlet sizes and pitot pressure readings are available in the sources listed in the bibliography.)

Next, using the formula given below and in Table 5.58, compute what the flow Q_R would be at a residual pressure of 20 psi (1.4 kgf/cm²).

$$Q_R = Q_F \times \frac{h_R^{0.54}}{h_F^{0.54}}$$

where Q_R = flow available at desired residual pressure

Q_F = flow during test

h_R = pressure drop to desired residual pressure

h_F = pressure drop during test

The value of Q_F is the observed flow of 870 gpm; h_R is the difference between the normal pressure and the desired residual pressure = 65 – 20 = 45 lb/in² and h_F is the actual drop in pressure observed = 65 – 40 = 25 lb/in².

Substituting in the formula,

$$Q_R = 870 \text{ gpm} \times \frac{(45 \text{ lb/in}^2)^{0.54}}{(25 \text{ lb/in}^2)^{0.54}}$$

From Table 5.58,

$$(45 \text{ lb/in}^2)^{0.54} = 7.81$$

$$(25 \text{ lb/in}^2)^{0.54} = 5.69$$

Substituting,

$$Q_R = 870 \times \frac{7.81}{5.69} = 1194 \text{ gpm at } 20 \text{ lb/in}^2$$

TABLE 5.56 Discharge Table for Circular Outlets of 2¼ to 2¹¹/₁₆ in. (Outlet pressure measured by pitot gage.)

Outlet pressure, lb/in ²	Outlet diameter, in							
	2¼	2 ¹ / ₁₆	2 ³ / ₈	2 ⁷ / ₁₆	2½	2 ⁹ / ₁₆	2 ⁵ / ₈	2 ¹¹ / ₁₆
	Discharge, gpm							
¼	70	70	80	80	80	90	90	100
½	100	100	110	110	120	120	130	140
¾	120	120	130	140	150	150	160	170
1	140	140	150	160	170	180	180	190
¼	150	160	170	180	190	200	210	220
½	170	180	190	200	210	220	230	240
¾	180	190	200	210	220	230	240	260
2	190	200	210	230	240	250	260	270
¼	200	220	230	240	250	260	280	290
½	220	230	240	250	270	280	290	310
¾	230	240	250	260	280	290	310	320
3	240	250	260	280	290	310	320	340
¼	250	260	270	290	300	320	330	350
½	250	270	280	300	310	330	350	360
¾	260	280	290	310	330	340	360	380
4	270	290	300	320	340	350	370	390
¼	280	300	310	330	350	360	380	400
½	290	300	320	340	360	370	390	410
¾	300	310	330	350	370	380	400	420
5	300	320	340	360	380	390	410	430
¼	310	330	350	370	390	400	420	440
½	320	340	350	370	390	410	430	450
¾	330	340	360	380	400	420	440	460
6	330	350	370	390	410	430	450	470
¼	340	360	380	400	420	440	460	480
½	350	370	390	410	430	450	470	490
¾	350	370	390	410	440	460	480	500
7	360	380	400	420	440	470	490	510
¼	370	390	410	430	450	480	500	520
½	370	390	410	440	460	480	510	530
¾	380	400	420	440	470	490	510	540
8	380	410	430	450	480	500	520	550
¼	390	410	440	460	480	510	530	560
½	400	420	440	460	490	510	540	560
¾	400	420	450	470	500	520	550	570
9	410	430	450	480	500	530	550	580
¼	410	440	460	480	510	540	560	590
½	420	440	470	490	520	540	570	600
¾	420	450	470	500	520	550	580	600
10	430	450	480	500	530	560	580	610
¼	440	460	480	510	540	570	590	620
½	440	470	490	520	540	570	600	630
¾	450	470	500	520	550	580	610	640

Outlet pressure, lb/in ²	Outlet diameter, in							
	2¼	2½	2¾	2⅞	2⅚	2⅞	2⅞	2⅞
	Discharge, gpm							
11	450	480	500	530	560	590	610	640
¼	460	480	510	530	560	590	620	650
½	460	490	510	540	570	600	630	660
¾	470	490	520	550	580	600	630	660
12	470	500	520	550	580	610	640	670
½	480	510	540	560	590	620	650	690
13	490	520	550	570	610	640	670	700
½	500	530	560	590	620	650	680	710
14	510	540	570	600	630	660	690	730
½	520	550	580	610	640	670	700	740
15	530	560	590	620	650	680	720	750
½	540	570	600	630	660	700	730	760
16	540	570	610	640	670	710	740	780
½	550	580	620	650	680	720	750	790
17	560	590	620	660	690	730	760	800
½	570	600	630	670	700	740	770	810
18	580	610	640	680	710	750	780	820
½	590	620	650	690	720	760	800	830
19	590	630	660	700	730	770	810	840
½	600	640	670	700	740	780	820	860
20	610	640	680	710	750	790	830	870
21	620	660	690	730	770	810	850	890
22	640	670	710	750	790	830	870	910
23	650	690	730	770	810	850	890	930
24	670	700	740	780	820	860	910	950
25	680	720	760	800	840	880	920	970
26	690	730	770	810	860	900	940	990
27	710	750	790	830	870	920	960	1010
28	720	760	800	840	890	930	980	1020
29	730	770	820	860	910	950	1000	1040
30	750	790	830	870	920	970	1010	1060
31	760	800	840	890	940	980	1030	1080
32	770	810	860	900	950	1000	1050	1100
33	780	830	870	920	970	1010	1060	1110
34	790	840	880	930	980	1030	1080	1130
35	810	850	900	940	990	1040	1090	1140
36	820	860	910	960	1010	1060	1110	1160

Note: To convert from inches to centimeters multiply by 2.54; to convert from lb/in² to kgf/cm², multiply by 0.0708; to convert from gpm to m³/min, multiply by 0.003785.

Source: From American Insurance Association, "Fire Flow Tests," Discharge Tables for Circular Outlets.

TABLE 5.57 Discharge Table for Circular Outlets of 4¼ to 4¹¹/₁₆ in. (Outlet pressure measured by pitot gage.)

Outlet pressure, lb/in ²	Outlet diameter, in							
	4¼	4 ¹ / ₁₆	4 ³ / ₈	4 ⁷ / ₁₆	4½	4 ⁹ / ₁₆	4 ⁵ / ₈	4 ¹¹ / ₁₆
	Discharge, gpm							
¼	240	250	260	260	270	280	290	300
½	340	350	360	370	390	400	410	420
¾	420	430	450	460	470	490	500	510
1	490	500	520	530	550	560	570	590
¼	540	560	590	590	610	630	640	660
½	600	610	630	650	670	690	700	720
¾	640	660	680	700	720	740	760	780
2	690	710	730	750	770	790	810	840
¼	730	750	770	800	820	840	860	890
½	770	790	810	840	860	890	910	940
¾	810	830	850	880	900	930	950	980
3	840	870	890	920	940	970	1000	1020
¼	880	900	930	960	980	1010	1040	1060
½	910	940	970	990	1020	1050	1070	1100
¾	940	970	1000	1030	1050	1080	1110	1140
4	970	1000	1030	1060	1090	1120	1150	1180
¼	1000	1030	1060	1090	1120	1150	1180	1220
½	1030	1060	1090	1120	1160	1190	1220	1250
¾	1060	1090	1120	1150	1190	1220	1250	1290
5	1090	1120	1150	1180	1220	1250	1280	1320
¼	1110	1150	1180	1210	1250	1280	1320	1350
½	1140	1180	1210	1240	1280	1310	1350	1390
¾								
6	1190	1230	1260	1300	1330	1370	1410	1450
¼	1220	1250	1290	1320	1360	1400	1440	1480
½	1240	1280	1310	1350	1390	1430	1470	1510
¾	1260	1300	1340	1380	1420	1450	1490	1540
7	1290	1330	1360	1400	1440	1480	1520	1560
¼	1310	1350	1390	1430	1470	1510	1550	1590
½	1330	1370	1410	1450	1490	1530	1570	1620
¾	1350	1390	1430	1480	1520	1560	1600	1640
8	1380	1420	1460	1500	1540	1580	1620	1670
¼	1400	1440	1480	1520	1570	1610	1650	1700
½	1420	1460	1500	1540	1590	1630	1680	1720
¾	1440	1480	1520	1570	1610	1650	1700	1750
9	1460	1500	1540	1590	1630	1680	1720	1770
¼	1480	1520	1570	1610	1660	1700	1750	1800
½	1500	1540	1590	1630	1680	1720	1770	1820
¾	1520	1560	1610	1650	1700	1750	1790	1840
10	1540	1580	1630	1670	1720	1770	1820	1870
¼	1560	1600	1650	1700	1740	1790	1840	1890
½	1580	1620	1670	1720	1760	1810	1860	1910
¾	1590	1640	1690	1740	1790	1830	1880	1940

Outlet pressure, lb/in ²	Outlet diameter, in							
	4/4	4 ¹ / ₁₆	4 ³ / ₈	4 ⁷ / ₁₆	4 ¹ / ₂	4 ⁹ / ₁₆	4 ⁵ / ₈	4 ¹¹ / ₁₆
	Discharge, gpm							
11	1610	1660	1710	1760	1810	1860	1910	1960
¼	1630	1680	1730	1780	1830	1880	1930	1980
½	1650	1700	1750	1800	1850	1900	1950	2000
¾	1670	1720	1760	1820	1870	1920	1970	2020
12	1690	1730	1780	1840	1890	1940	1990	2050
½	1720	1770	1820	1870	1930	1980	2030	2090
13	1750	1800	1850	1910	1970	2020	2070	2130
½	1790	1840	1890	1950	2000	2060	2110	2170
14	1820	1870	1930	1980	2040	2090	2150	2210
½	1850	1910	1960	2020	2080	2130	2190	2250
15	1880	1940	1990	2050	2110	2170	2230	2290
½	1910	1970	2030	2090	2150	2200	2260	2330
16	1940	2000	2060	2120	2180	2240	2300	2360
½	1970	2030	2090	2150	2210	2270	2330	2400
17	2000	2060	2120	2180	2250	2310	2370	2440
½	2030	2090	2150	2220	2280	2340	2400	2470
18	2060	2120	2180	2250	2310	2370	2440	2510
½	2090	2150	2210	2280	2350	2410	2470	2540
19	2120	2180	2240	2310	2380	2440	2510	2580
½	2140	2210	2270	2340	2410	2470	2540	2610
20	2170	2240	2300	2370	2440	2500	2570	2640
21	2220	2290	2360	2430	2500	2560	2630	2710
22	2280	2350	2420	2490	2560	2620	2700	2770
23	2330	2400	2470	2540	2610	2680	2760	2830
24	2380	2450	2520	2600	2670	2740	2820	2890
25	2430	2500	2580	2650	2720	2800	2870	2950
26	2480	2550	2630	2700	2780	2850	2930	3010
27	2530	2600	2680	2750	2830	2910	2990	3070
28	2580	2650	2730	2800	2880	2960	3040	3130
29	2620	2700	2770	2850	2940	3020	3090	3180
30	2670	2740	2820	2900	2990	3070	3150	3240
31	2710	2790	2870	2950	3030	3120	3200	3290
32	2750	2830	2920	3000	3080	3170	3250	3340
33	2790	2880	2960	3040	3130	3220	3300	3390
34	2830	2920	3000	3090	3170	3260	3350	3440
35	2870	2960	3040	3140	3220	3310	3400	3490
36	2910	3000	3080	3180	3270	3360	3450	3540

Note: To convert from inches to centimeters, multiply by 2.54; to convert from lb/in² to kgf/cm², multiply by 0.0703; to convert from gpm to m³/min, multiply by 0.003785.

Source: American Insurance Association, "Fire Flow Tests," Discharge Tables for Circular Outlets.

TABLE 5.58 Computing Fire Flow Test Results*

<i>h</i>	<i>h</i> ^{0.54}												
1	1.00	26	5.81	51	8.38	76	10.37	101	12.09	126	13.62	151	15.02
2	1.45	27	5.93	52	8.44	77	10.44	102	12.15	127	13.68	152	15.07
3	1.81	28	6.05	53	8.53	78	10.51	103	12.22	128	13.74	153	15.13
4	2.11	29	6.16	54	8.62	79	10.59	104	12.28	129	13.80	154	15.18
5	2.39	30	6.28	55	8.71	80	10.66	105	12.34	130	13.85	155	15.23
6	2.63	31	6.39	56	8.79	81	10.73	106	12.41	131	13.91	156	15.29
7	2.86	32	6.50	57	8.88	82	10.80	107	12.47	132	13.97	157	15.34
8	3.07	33	6.61	58	8.96	83	10.87	108	12.53	133	14.02	158	15.39
9	3.28	34	6.71	59	9.04	84	10.94	109	12.60	134	14.08	159	15.44
10	3.47	35	6.82	60	9.12	85	11.01	110	12.66	135	14.14	160	15.50
11	3.65	36	6.93	61	9.21	86	11.08	111	12.72	136	14.19	161	15.55
12	3.83	37	7.03	62	9.29	87	11.15	112	12.78	137	14.25	162	15.60
13	4.00	38	7.13	63	9.37	88	11.22	113	12.84	138	14.31	163	15.65
14	4.16	39	7.23	64	9.45	89	11.29	114	12.90	139	14.36	164	15.70
15	4.32	40	7.33	65	9.53	90	11.36	115	12.96	140	14.42	165	15.78
16	4.47	41	7.43	66	9.61	91	11.43	116	13.03	141	14.47	166	15.81
17	4.62	42	7.53	67	9.69	92	11.49	117	13.09	142	14.53	167	15.86
18	4.78	43	7.62	68	9.76	93	11.56	118	13.15	143	14.58	168	15.91
19	4.90	44	7.72	69	9.84	94	11.63	119	13.21	144	14.64	169	15.96
20	5.04	45	7.81	70	9.92	95	11.69	120	13.27	145	14.69	170	16.01
21	5.18	46	7.91	71	9.99	96	11.76	121	13.33	146	14.75	171	16.06
22	5.31	47	8.00	72	10.07	97	11.83	122	13.39	147	14.80	172	16.11
23	5.44	48	8.09	73	10.14	98	11.89	123	13.44	148	14.86	173	16.16
24	5.56	49	8.18	74	10.22	99	11.96	124	13.50	149	14.91	174	16.21
25	5.69	50	8.27	75	10.29	100	12.02	125	13.56	150	14.97	175	16.25

*Method of use: Insert in the formula the values of $h_R^{0.54}$ determined from the table and the value of Q_R . See text for equation.

$$Q_R = Q_F \times \frac{h_R^{0.54}}{h_F^{0.54}}$$

Q_R = Flow available at desired residual pressure.

Q_F = Flow during test.

h_R = Pressure drop to desired residual pressure.

h_F = Pressure drop during test. (Value of h to the 0.54 power.)

Source: From American Insurance Association, "Fire Flow Tests." Discharge Table for Circular Outlets.

If the distribution system is designed so that flow from a single hydrant outlet results in only a slight change in pressure, then it may be necessary to open both outlets on a hydrant or to use several hydrants simultaneously in order to obtain a meaningful pressure drop. When several hydrants are used, the normal and residual pressures should be taken at a location between the flowing hydrants. The calculation is similar to the above example, with Q_F set equal to the sum of the hydrant flows.

Distribution System Configuration

Where possible, mains should be placed in areas along the public right of way that are not subject to paving or obstacles to provide for ease of installation, repair, and maintenance. Dual mains should be installed on

each side of streets and heavily traveled highways which are wider than 75 ft (22.9 m). One pipe usually supplies the domestic and fire flow requirements for one side of the street, while the other main supplies only domestic requirements for the other side of the street. Occasionally, pipes are sized to supply both fire and domestic flows for each side of the street if it is extremely wide or the street is divided by a barrier such as a waterway, railroad, or some other obstacle. The primary advantage of the dual-main system is that the required length of pipe is reduced significantly, allowing for pressure loss in a service pipe to be as small as possible and to eliminate the necessity of cutting through pavement to repair such services.

The branch and loop are the two basic configurations for most water distribution systems. For both systems, distribution system pressures and valving are important considerations.

Branch Systems. The layout of the branch system is like that of a tree branch with smaller pipes branching off larger pipes through the area to be served. This system or a derivative of it is normally used to supply rural areas where the demand is relatively low and long distances must be covered. Disadvantages of this configuration are the possibility of a large number of customers being without service should a main break occur, because of the absence of an alternative main, and the possibility of water quality problems in the system because of no circulation. System flushing should be undertaken at regular intervals because of this circulation problem.

Loop Systems. The grid or loop configuration is most widely used today. This configuration consists of connected pipe loops throughout the area to be served. Good design practices for smaller systems call for feeder mains to form a loop approximately 1 mi (1600 m) in radius around the center of the town and additional feeder loops provided according to the particular layout and geography of the area to be served. The area inside and immediately surrounding the feeder loops should be gridded with connecting water mains on every street. Minimum size for the distribution mains in the distribution grid should be 6 in (15.2 cm) and the minimum size for the feeder mains will vary depending on the size of the loop and flow demand in a particular area. Normally the system is sized for water velocities in the pipe from 3 to 5 ft/s (0.9 to 1.5 m/s) or a desired hydraulic slope to provide a relatively evenly spaced pressure contour across the system.

A good design for large looped distribution systems is to space transmission mains, which are normally 24 in (61 cm) and larger, 1.5 to 2 mi (2400 to 3200 m) apart. Feeder mains are normally 16 to 20 in (40.6 to 50.8 cm) and should be spaced 3000 to 4000 ft (900 to 1200 m) apart. The remaining grid should be served with 6 to 12 in (15 to 30 cm) mains in every street with the size depending on the flows required. Minimum sizes should be 6 in (15 cm) in residential areas and 8 in (20 cm) in commercial and light industrial areas, depending on fire flow requirements (114).

System Pressures. Normal static pressure in distribution systems should be maintained from 60 to 75 lb/in² (4.2 to 5.3 kgf/cm²). High pressures of 90 to 100 lb/in² (6.3 to 7.0 kgf/cm²) should be avoided if possible because of the added burden of installing and maintaining pressure-reducing valves and other specialized equipment. It may be necessary for a distribution system to have one or more pressure zones when a difference in elevation of the terrain served is 200 ft (60 m) or more. The pressure in each zone is controlled either by pumps or pressure-regulating-valve arrangements. Care should be exercised in maintaining the correct pressure range in each zone to eliminate damage to customer's facilities due to too high or too low pressure.

Valve Spacing and Sizing. Transmission valves are normally located 1 to 2 mi (1600 to 3200 m) apart or at locations which will give the best possible control of the system. Valves placed in feeder mains should be located so that each loop can be effectively isolated. Maximum spacing should be approximately 4000 ft (1200 m). Valves 12 in (30 cm) and smaller should be placed to provide good main isolation capability without interrupting service to large numbers of customers. Normally these valves have at least 5000-ft (150-m) spacing.

Valve sizes are normally the size of the pipe that they are installed on for mains up to 24 in (60 cm) in diameter. Mains larger than 30 in (76 cm) may have line valves one size smaller than the main size without sacrificing too much head loss and normally result in substantial savings in material and installation costs.

Good practice calls for blowoff valves to be at least 6 in (15 cm) for 16-in (40-cm) and smaller mains, and at least 8 in (20.3 cm) for 20-in (50-cm) and larger mains.

Cross-Connections and System Contamination

A cross-connection is any direct connection between a drinking water distribution system and any nondrinkable fluid or substance (115). Cross-connections are a means for contaminants to enter drinking water and consequently jeopardize health. But, before more technical descriptions of cross-connections and their control are given, it is helpful to view cross-connections in perspective with the other means by which contaminants can enter a water distribution system.

Leaks. Joint leaks and other leaks in the water main system pose a threat to water quality wherever water is transported through an underground distribution system. Under certain conditions, contaminated groundwater can be forced into a leaking main under natural hydrostatic pressure, or can be aspirated into the main because of a venturi effect. In general, good system pressure and adequately sized mains prevent this type of contamination. If groundwater does not enter the main, the disinfectant residual required of all public distribution systems kills off the typically small numbers of organisms associated with this type of contamination. However, if the contaminant is chemical in nature, noticeable contamination appears. An example of this occurred when a leaking underground gasoline storage tank contaminated the groundwater surrounding a water main. Noticeable amounts of gasoline found in the drinking water were attributed to the aspiration of the groundwater through a water main leak.

Storage Tanks. Ground and elevated storage tanks may become sources of nonpotable water if they are not designed or operated properly. Most tanks are vented to the atmosphere and, if the vent is not properly screened, vermin can enter the tank, leave their excrement, or drown, and thus contaminate the water. If the tanks are allowed to stagnate, bacteria and other organisms can enter the vents and begin culturing as the disinfectant residual dissipates.

The design operation of filling and draining of storage tanks affect the residence time and mixing of waters of different ages (and residual disinfectant levels) in the tanks, which can lead to variations in the disinfection effectiveness in distribution system (170). Also, since water in some storage tanks may have a long residence time, the operation of the tank needs to be designed to optimize water quality based on hydraulics, mixing, and chemical kinetics (171).

Construction. The problems of sanitary plumbing and piping construction discussed here are obvious and the types of contamination that could occur as a result of repairs or new construction are virtually limitless. Water mains have been connected to sewer force mains, and toxic greases have been used to lubricate valves. Fortunately, plumbing schools, codes, and inspection programs control most problems.

Cross-Connections. Cross-connections are sometimes called the unseen hazard because they defy most persons' idea of common sense. If a garage attendant immerses a hose in a radiator, he does not see the potential problem this could create for the garage plumbing or for the public distribution system.

Probably more than half of all outlets supplied by a distribution system have been cross-connected at one time: hoses connected to hose bibs are used to apply pesticides, mix detergents in buckets, and blow out clogged sewer pipes. There are also thousands of permanent cross-connections: makeup lines to vats, boil-

ers, and air conditioning systems; connections to lawn sprinkler and fire systems; and connections to auxiliary supplies such as well, bay, pond, and reuse water. In fact, the list of cross-connections is so long and the types of potential contaminants so hazardous, that cross-connections are recognized as the single worst threat to the potability of water once it leaves the treatment plant.

Cross-Connections and Backflow. Cross-connections lie dormant until backflow occurs. Unfortunately, the causes of backflow are difficult to eliminate because backflow is usually initiated by accidents or unexpected circumstances. Water potability is protected, therefore, by using backflow preventers wherever there is a cross-connection. To understand backflow preventers, one must first differentiate and define the two types of backflow: back-siphonage and back pressure.

Back-siphonage occurs when atmospheric pressure exceeds supply pressure. Virtually all outlets are subject to back-siphonage when given the right circumstances. Back pressure occurs when the pressure at a point of use exceeds the supply pressure. Back pressure may be a problem whenever heat, head, or pumps are used. Figure 5.88 describes one type of backflow preventer and Table 5.59 lists the most common causes of backflow (121). Only a few types of backflow prevention devices can be used to protect against both back pressure and back-siphonage. As a result, a clear understanding of the differences between the two types of backflow is important before the correct type of backflow prevention device can be chosen to protect against a specific cross-connection.

Degree of Hazard. Besides distinguishing the type of backflow involved, the degree of hazard must also be appraised for a cross-connection before an appropriate backflow prevention device can be selected. Degree of hazard is a qualification of the potential or actual harm that may result from the creation of a cross-connection.

The degree of hazard is based strictly on the toxicity of the potential contaminant. Most cross-connection manuals divide hazardousness into three categories: high, moderate, and low. High-hazard cross-connections are those where the potential contaminant threatens health by poisoning or the spread of disease. Moderate-hazard cross-connections cause highly objectionable contamination but do not pose a health hazard. Low- or minor-hazard cross-connections cause moderately objectionable contamination, but do not threaten health.

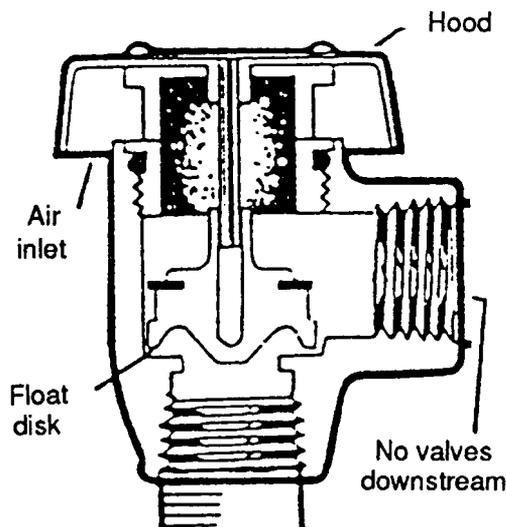


FIGURE 5.88 Atmospheric vacuum breaker (121).

TABLE 5.59 Most Common Causes of Backflow

Causes	Preventive measures
I. Back siphonage	
A. Originating in water main	(Other than backflow preventives)
1. Systemwide low pressure caused by drought or pumping plant failure.	None
2. Water main shutdown	None
3. High usage because of fire demand	Adequately sized mains
4. Water main break, particularly dangerous in hilly or mountainous areas	None
5. Large capacity fire pumps drawing suction from water main	Low-pressure cutoff switches or pump suction control valves
6. Domestic booster pumps drawing suction during main shutdown or times of low pressure	Low-pressure cutoff switches
7. High usage with a partially or completely closed valve or a key main, forcing large flows through smaller mains	Valve maintenance program
B. Originating in plumbing system	
1. Water shutdown for repairs, etc.	None
2. Where booster pumps drawing suction from house supply that feeds other outlets.	Good design, low-pressure cutoff switch
3. Undersized and overextended plumbing	Good design
4. Insufficient pressure to support water columns in tall risers	None
II. Backpressure (Originates from private plumbing or equipment only)	
1. Elevated storage tanks or supply lines, which by virtue of their elevation would create head sufficient to cause backflow if supply pressure is lowered	None
2. Makeup lines or connections to systems pressurized by pumps	None
3. Makeup lines or connections to equipment that could generate steam or thermal expansion	None

Backflow Prevention Devices. Once a cross-connection has been identified, a backflow prevention device should be selected that will adequately protect the potability of the water distribution system. Table 5.60 gives a listing of currently accepted backflow prevention devices. As indicated in the table, some devices can be tested under working conditions. These devices have test cocks and are usually required by either the water purveyor, plumbing inspection bureau, or health department, to have annual testing. Since all mechanical devices in a water system are subject to corrosion and wear, regular testing, especially in line testing, ensures a device's proper operation. Devices that cannot be tested are less expensive and should be considered less reliable over time. However, vacuum-breaker-type backflow preventers, when installed correctly and not subject to back pressure or continuous pressure, have a relatively high degree of reliability. Correct installation implies that they will receive adequate exercising and cleaning of moving parts designed to admit air.

A vacuum breaker that leaks water from its air opening is actually working since air could just as easily enter the device to break a vacuum. Too often, a vacuum breaker maintenance program, as in hospitals, consists of repairing vacuum breakers that leak water. From the point of view of backflow prevention, vacuum breakers that do not leak are more likely to be out of repair, since the float disk (see Figure 5.88) may be per-

TABLE 5.60 Backflow Prevention Devices

Common abbreviations	Common name	Type of backflow	Degree of hazard	Testable in line under working conditions	Design standards	Size Range, in	General installation requirements	Remarks
RP, RPZ, RPP, RPBD	Reduced pressure principle backflow preventer	Good against backpressure and back siphonage	Good against all toxic and nontoxic substances	Yes	AWWA C506-78 ASSE 1013 USC	3/4–10	Installed minimum of 12 in above ground or flood level	Best overall backflow preventer currently available.
AG	Air gap	Good against backpressure and back siphonage	Good against all toxic and nontoxic substances	N.A. (non-mechanical)	ANSI A-112.1.2-1973 BOCA Basic Plumbing Code	N.A.	A distance of two times the diameter of the supply pipe, above the flood rim level of the tank, vat, container, etc.	Excellent backflow preventer but subject to tampering or easy circumvention
DCVA, DCU, DC	Double check valve assembly	Good against backpressure and back siphonage	Good against nontoxic substances only	Yes	AWWA C 506-78 ASSE #1015, USC	3/4–10	Installed minimum of 12 in above ground or flood level	
DDCV, DDC	Double detector valve or double check detector check assembly	Good against backpressure and back siphonage	Good against nontoxic substances only	Yes	USC	4–10	Installed minimum of 12 in above ground or flood level	For use on unmetered fire lines, consists of DCVA with a small DCVA and meter on a bypass.

(continues)

TABLE 5.60 Backflow Prevention Devices (*continued*)

Common abbreviations	Common name	Type of backflow	Degree of hazard	Testable in line under working conditions	Design standards	Size Range, in	General installation requirements	Remarks
PVB	Pressure vacuum breaker or antisiphon vacuum breaker, pressure type	Good against back siphonage only	Good against all toxic and nontoxic substances	Yes	ASSE #1020 USC	1/2–10	Installed minimum of 12" (measured from critical level) above highest outlet	2 1/2–0 in sizes not normally used; may be installed under continuous pressure, i.e., valves permitted downstream of device
AVB	Atmospheric vacuum breaker or antisiphon vacuum breaker, noncontinuous pressure type Barometric loop	Good against back siphonage only	Good against many toxic and all nontoxic substances	Yes (but visual test)	ASSE #1001	3/4–3	Installed minimum of 6 in (measured from critical level) above highest outlet	May not be installed under continuous pressure; no more than 12 h of continuous service in a day. See text for further comments
		Good against back siphonage only	Good against all toxic and nontoxic substances	N.A. (non-mechanical)	None known	N.A.	Crest if loop must be 34 ft above highest outlet (may be less than 34 ft depending upon elevation and chemical character of water	

RP-1AV	Backflow preventer with intermediate atmospheric vent	Good against backpressure and back siphonage	Good against nontoxic substances only	No	ASSE #1012	1/2, 3/4	Vent must not be cross-connected
HBVB	Hose bib vacuum breaker or hose connector vacuum breaker Laboratory faucet vacuum breaker Dual check valve	Good against back siphonage and up to 10 ft of head pressure Good against back siphonage only Good against back siphonage and backpressure	Good against nontoxic substances Good against toxic and nontoxic substances Good against some toxic and all nontoxic substances	No	ASSE #1011 ASSE #1035 (under development) ASSE #1024	3/4 (hose-thread) 3/8 1/2-1	No more than 12 h continuous service in a day Should be installed on all hose threaded spigots For use on gooseneck type laboratory faucets May be installed in line, above or below ground level Installed at carbonator
	Dual check valve type of backflow preventer for carbonated beverage dispensers Vacuum breaker	Good against back siphonage and backpressure Good against back siphonage only	Good against some toxic and all nontoxic substances Good against some toxic and all nontoxic substances Good against some toxic and all nontoxic substances	No	ASSE #1032 ANSI A 40.6-1942	3/8	May be repaired. Must be replaced. Primary use is in homes May be repaired Used on flush valve toilets and urinals

Note: 1 in = 2.54 cm; 1 ft = 0.3 m.

TABLE 5.61 Listing of AWWA Standards

Designation	AWWA Standard for
<i>Source</i>	
Groundwater and Wells	
A100-90	Water Wells
<i>Treatment</i>	
Filtration	
B100-89	Filtering Material (Includes addenda B100a-91 and B100b-92)
B101-94	Precoat Filter Media
Softening	
B200-93	Sodium Chloride
B201-92	Soda Ash
B202-93	Quicklime and Hydrated Lime
<i>Disinfection Chemicals</i>	
B300-92	Hypochlorites
B301-92	Liquid Chlorine
B302-95	Ammonium Sulfate
B303-95	Sodium Chlorite
Coagulation	
B402-95	Ferrous Sulfate
B403-93	Aluminum Sulfate—Liquid, Ground, or Lump
B404-92	Liquid Sodium Silicate
B405-94	Sodium Aluminate
B406-92	Ferric Sulfate
B407-93	Liquid Ferric Chloride
B408-93	Liquid Polyaluminum Chloride
B451 -92	Poly (Diallyldimethylammonium Chloride)
B452-90	EPI-DMA Polyamines
B453-96	Polyacrylamide
Scale and Corrosion Control	
B501-93	Sodium Hydroxide (Caustic Soda)
B502-94	Sodium Polyphosphate, Glassy (sodium hexametaphosphate)
B503-94	Sodium Tripolyphosphate
B504-94	Monosodium Phosphate Anhydrous
B505-95	Disodium Phosphate, Anhydrous
B510-95	Carbon Dioxide
B511-90	Potassium Hydroxide
B512-91	Sulfur Dioxide
B550-90	Calcium Chloride
Taste and Odor Control	
B600-90	Powdered Activated Carbon
B601-93	Sodium Metabisulfite
B602-91	Copper Sulfate
B603-93	Potassium Permanganate
B604-90	Granular Activated Carbon
Prophylaxis	
B701-94	Sodium Fluoride
B702-94	Sodium Fluorosilicate
B703-94	Fluorosilicic Acid

TABLE 5.61 Listing of AWWA Standards (*continued*)

Designation	AWWA Standard for
<i>Pipe and Accessories</i>	
Ductile-Iron Pipe and Fittings	
C104/A21.4-95	ANSI Standard for Cement-Mortar Lining for Ductile-Iron Pipe and Fittings for Water
C105/A21.5-93	ANSI Standard for Polyethylene Encasement for Ductile-Iron Pipe Systems
C110/A21.10-93	ANSI Standard for Ductile-Iron and Gray-Iron Fittings, 3 in through 48 in (75 mm through 1200 mm), for Water and Other Liquids
C111/A21.11-95	ANSI Standard for Rubber-Gasket Joints for Ductile-Iron Pressure Pipe and Fittings
C115/A21.15-94	ANSI Standard for Flanged Ductile-Iron Pipe With Ductile-Iron or Gray-Iron Threaded Flanges
C150/A21.50-91	ANSI Standard for Thickness Design of Ductile-Iron Pipe
C151/A21.51-91	ANSI Standard for Ductile-Iron Pipe, Centrifugally Cast, for Water and Other Liquids
C153/A21.53-94	ANSI Standard for Ductile-Iron Compact Fittings, 3 in through 24 in (76 mm through 610 mm) and 54 in through 64 in (1400 mm through 1,600 mm), for Water Service
Steel Pipe	
C200-91	Steel Water Pipe—6 in (150 mm) and Larger
C203-91	Coal-Tar Protective Coatings and Linings for Steel Water Pipelines—Enamel and Tape—Hot-Applied
C205-95	Cement-Mortar Protective Lining and Coating for Steel Water Pipe—4 in and Larger—Shop Applied
C206-91	Field Welding of Steel Water Pipe
C207-94	Steel Pipe Flanges for Waterworks Service— Sizes 4 in through 144 in (100 mm through 3600 mm)— <i>Includes erratum C207-94</i>
C208-83(R89)	Dimensions for Fabricated Steel Water Pipe Fittings
C209-95	Cold-Applied Tape Coatings for the Exterior of Special Sections, Connections, and Fittings for Steel Water Pipelines
C210-92	Liquid-Epoxy Coating Systems for the Interior and Exterior of Steel Water Pipelines
C213-91	Fusion-Bonded Epoxy Coating for the Interior and Exterior of Steel Water Pipelines
C214-95	Tape Coating Systems for the Exterior of Steel Water Pipelines
C21 5-94	Extruded Polyolefin Coatings for the Exterior of Steel Water Pipelines
C21 6-94	Heat-Shrinkable Cross-Linked Polyolefin Coatings for the Exterior of Special Sections, Connections, and Fittings for Steel Water Pipelines
C217-95	Cold-Applied Petrolatum Tape and Petroleum Wax Tape Coatings for the Exterior of Special Sections, Connections, and Fittings for Buried Steel Water Pipelines
C21 B-95	Coating the Exterior of Aboveground Steel Water Pipelines and Fittings
C219-91	Bolted, Sleeve-Type Couplings for Plain-End Pipe
C220-92	Stainless-Steel Pipe, 4 in (100 mm) and Larger
Concrete Pipe	
C300-B9	Reinforced Concrete Pressure Pipe, Steel-Cylinder Type, for Water and Other Liquids (Includes addendum C300a-93)

(continues)

TABLE 5.61 Listing of AWWA Standards (*continued*)

Designation	AWWA Standard for
Concrete Pipe (<i>cont.</i>)	
C301-92	Prestressed Concrete Pressure Pipe, Steel-Cylinder Type, for Water and Other Liquids
C302-95	Reinforced Concrete Pressure Pipe, Noncylinder Type
C303-95	Concrete Pressure Pipe, Bar-Wrapped Steel Cylinder Type
C304-92	Design of Prestressed Concrete Cylinder Pipe
Asbestos-Cement Pipe	
C400-93	Asbestos-Cement Pressure Pipe, 4 in through 16 in (100mm through 400 mm), for Water Distribution Systems
C401-93	Selection of Asbestos-Cement Pressure Pipe, 4 in through 16 in (100 mm through 400 mm), for Water Distribution Systems
C402-95	Asbestos-Cement Transmission Pipe, 18 in through 42 in (450 mm through 1050 mm), for Water Supply Service
C403-95	Selection of Asbestos-Cement Transmission and Feeder Main Pipe, Sizes 18 in through 42 in (450 mm through 1050 mm) for Water Supply Service
Valves and Hydrants	
C500-93	Metal-Seated Gate Valves for Water Supply Service (Includes addendum C500a-95)
C501-92	Cast-Iron Sluice Gates
C502-94	Dry-Barrel Fire Hydrants (Includes addendum C502a-95)
C503-88	Wet-Barrel Fire Hydrants
C504-94	Rubber-Seated Butterfly Valves
C507-91	Ball Valves 6 in through 48 in (150 mm through 1200 mm)
C508-93	Swing-Check Valves for Waterworks Service, 2 in (50 mm) through 24 in (50 mm through 600 mm) NPS (Includes addendum C508a-93)
C509-94	Resilient-Seated Gate Valves for Water Supply Service (Includes addendum C509a-95)
C51 0-92	Double Check Valve Backflow-PreventiOn Assembly
C51 1-92	Reduced-Pressure Principle Backflow-PreventiOn Assembly
C512-92	Air Release, Air/Vacuum, and Combination Air Valves for Waterworks Service
C540-93	Power-Actuating Devices for Valves and Sluice Gates
C550-90	Protective Epoxy Interior Coatings for Valves and Hydrants
<i>Pipe Installation</i>	
C600-93	Installation of Ductile-Iron Water Mains and Their Appurtenances
C602-95	Cement-Mortar Lining of Water Pipelines In Place—4 in (100 mm) and Larger
C603-90	Installation of Asbestos-Cement Pressure Pipe
C605-94	Underground Installation of Polyvinyl Chloride (PVC) Pressure Pipe and Fittings for Water
C606-87	Grooved and Shouldered Joints
<i>Disinfection of Facilities</i>	
C651-92	Disinfecting Water Mains
C652-92	Disinfection of Water-Storage Facilities
C653-95	Disinfection of Water Treatment Plants
C654-95	Disinfection of Wells

TABLE 5.61 Listing of AWWA Standards (*continued*)

Designation	AWWA Standard for
<i>Meters</i>	
C700-95	Cold-Water Meters—Displacement Type, Bronze Main Case
C701-88	Cold-Water Meters—Turbine Type, for Customer Service
C702-92	Cold-Water Meters—Compound Type
C703-86	Cold-Water Meters—Fire Service Type
C704-92	Propeller-Type Meters for Waterworks Application
C706-91	Direct-Reading, Remote-Registration Systems for Cold-Water Meters (Includes addendum C706a-92)
C707-82(R92)	Encoder-Type Remote-Registration Systems for Cold-Water Meters
C708-91	Cold-Water Meters—Multijet Type
C710-95	Cold-Water Meters—Displacement Type, Plastic Main Case
<i>Service Lines</i>	
C800-89	Underground Service Line Valves and Fittings (Also Included: Collected Standards for Service Line Materials)
<i>Plastic Pipe</i>	
C900-89	Polyvinyl Chloride (PVC) Pressure Pipe, 4 in through 12 in, for Water Distribution (Includes addendum C900a-92)
C901-88	Polyethylene (PE) Pressure Pipe and Tubing, 1/2 in. through 3 in, for Water Service
C902-88	Polybutylene (PB) Pressure Pipe and Tubing, 1/2 in through 3 in, for Water
C905-88	Polyvinyl Chloride (PVC) Water Transmission Pipe, Nominal Diameters 14 in. through 36 in
C906-90	Polyethylene (PE) Pressure Pipe and Fittings, 4 in through 63 in, for Water Distribution
C907-91	Polyvinyl Chloride (PVC) Pressure Fittings for Water—4 in through 8 in (100 mm through 200 mm)
C950-95	Fiberglass Pressure Pipe
<i>Storage</i>	
D100-84	Welded Steel Tanks for Water Storage (Includes addendum D100a-89)
D101-53(R86)	Inspecting and Repairing Steel Water Tanks, Standpipes, Reservoirs, and Elevated Tanks for Water Storage
D1 03-87	Factory-Coated Bolted Steel Tanks for Water Storage
0104-91	Automatically Controlled, Impressed-Current Cathodic Protection for the Interior of Steel Water Tanks
D1i0-95	Wire- and Strand-Wound Circular Prestressed Concrete Water Tanks
D115-95	Circular Prestressed Concrete Water Tanks With Circumferential Tendons
D120-84(R89)	Thermosetting Fiberglass-Reinforced Plastic Tanks
D130-87	Flexible-Membrane-Lining and Floating-Cover Materials for Potable-Water Storage
<i>Pumping</i>	
E101-88	Vertical Turbine Pumps—Line Shaft and Submersible Types
<i>Plant Equipment</i>	
FI01-91	Contact-Molded, Fiberglass-Reinforced Plastic Wash Water Troughs and Launders
FI02-91	Matched-Die-Molded, Fiberglass-Reinforced Plastic Weir Plates, Scum Baffles, and Mounting Brackets

manently adhered to the air inlet opening. Atmospheric vacuum breakers (AVBs) with a permanently sealed air inlet are often called “expensive elbows.”

AWWA STANDARDS

Since the early 1900's, the American Water Works Association (AWWA) has been developing standards to promote clean, safe drinking water and to enhance water utility operations. Some 120 AWWA standards address hundreds of products and procedures. To ensure that the standards remain current, every AWWA standard is reviewed approximately every five years and revised accordingly. Also, each AWWA standard is approved by the American National Standards Institute (ANSI) (172).

Each AWWA standard has a designation consisting of two elements connected by a hyphen. The first element identifies a topic area, and the second reports the year of approval or latest revision. For example, A100-90 is designated as a standard in the A100 series, groundwater and wells, that was approved, or last revised, in 1990.

A listing of AWWA Standards is presented in Table 5.61 (pages 5.218–5.221) under the following major categories: source, treatment, disinfection chemicals, pipe and accessories, pipe installation, disinfection of facilities, meters, service lines, plastic pipe, storage, pumping, plant equipment.

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CHAPTER 6

WASTEWATER DISPOSAL

Robert A. Corbitt, P.E.

Wastewater is the combination of liquid and water-transported wastes from homes, commercial buildings, industrial facilities, and institutions, along with any groundwater infiltration and surface water and stormwater inflow that may enter the sewer system (*I*).

At a minimum, treatment is required for suspended solids and for dissolved organics. Special processes may be necessary to achieve removal of specific pollutants, such as phosphorus from a municipal source or heavy metals from a plating facility. The minimum levels of treatment are established by regulation. For example, in the United States, 85% removal of oxygen-demanding organics and suspended solids and disinfection is the minimum level of treatment for domestic wastewaters. Additional treatment, including nutrient removal, is dictated by receiving-stream assimilative capacity and downstream water uses.

Physical processes are used to remove suspended solids. Screens remove debris and other large solids, and gravity or aerated grit chambers capture sandy matter, either of which may damage or interfere with subsequent pumping and treatment units. Gravity sedimentation normally is used to remove finer (organic) suspended solids. For special applications, centrifugation, dissolved air flotation, and filtration are used to remove suspended solids.

Dissolved organics generally are treated with biological processes. The more common systems are aerobic (with oxygen) and include aerobic or facultative pond, trickling filter, and activated sludge processes. Concentrated wastes, such as primary sludges, or high-strength industrial wastewaters, such as meat processing or brewery wastes, are considered for anaerobic (without oxygen) treatment processes.

Sludges, principally from biological processes, require special handling. The sequence of processes includes stabilization, conditioning, dewatering, drying, and residual disposal. Land application and landfilling are the most practiced means of final disposal. Special concerns for land-applied and composted sludges arise due to the concentration of contaminants, such as heavy metals, and presence of pathogens in these sludges.

Of the many other types of treatment process, increased attention is now given to natural systems, which historically included pond systems and, later, various modes of land application. Newer technology uses natural and constructed wetlands to provide high-quality effluents.

WASTEWATER POLLUTANTS: SOURCE AND EFFECT

Because of the diversity of wastewater sources, the medium is generally complex and variable, which means that the impact of wastewaters on the environment can vary greatly. In the following sections the nature, sources, and environmental effects of wastewaters are discussed.

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Nature of Wastewaters

A list of important wastewater contaminants and reasons for their importance is provided in Table 6.1. A detailed discussion of analytical methods for each contaminant may be found in *Standard Methods for the Examination of Water and Wastewater* (3).

Suspended Solids. Suspended solids are deemed important because solids can create an unsightly situation, sludge deposits, and a demand for oxygen. The standard way of testing a wastewater for suspended solids is to filter the wastewater through a 0.45- μm porosity filter. Anything on the filter after drying at about 103°C is considered a portion of the suspended solids. Table 6.2 provides another classification system for the solids found in wastewater.

Biodegradable Organics. As indicated in Table 6.1, biodegradable organics are also important to consider because as these materials are degraded in aquatic environments, dissolved oxygen is consumed. Since oxygen is not very soluble in water (see Table 6.3), it is rather easy to deplete oxygen levels in a stream when the flow of organic substances into the stream is not controlled properly. In those situations where all of the oxygen in the water is depleted, anaerobic conditions are said to have been created. Most substances can also be degraded under anaerobic conditions, but the process is generally slow and foul odors are often generated.

TABLE 6.1 Contaminant Importance in Wastewater Treatment (2)

Contaminants	Reasons for importance
Physical	
Suspended solids	Suspended solids are important for esthetic reasons and because they can lead to the development of sludge deposits and anaerobic conditions.
Chemical	
Biodegradable organics	Composed principally of proteins, carbohydrates, and fats, biodegradable organics are measured most commonly in terms of BOD (biochemical oxygen demand) and COD (chemical oxygen demand). If discharged untreated to the environment, the biological stabilization of these materials can lead to the depletion of natural oxygen resources and to the development of septic conditions.
Nutrients	Carbon, nitrogen, and phosphorus are essential nutrients for growth. When discharged to the aquatic environment, these nutrients can lead to the growth of undesirable aquatic life. When discharged in excessive amounts on land, they can also lead to the pollution of groundwater.
Refractory organics	These organics tend to resist conventional biological methods of wastewater treatment. Typical examples include surfactants, phenols, and agricultural pesticides.
Heavy metals	Due to their toxic nature, certain heavy metals can negatively impact upon biological waste treatment processes and stream life.
Dissolved inorganic solids	Inorganic constituents such as calcium, sodium, and sulfate are added to the original domestic water supply as a result of water use and may have to be removed if the wastewater is to be reused.
Biological	
Pathogens	Communicable diseases can be transmitted by the pathogenic organisms in wastewater.

TABLE 6.2 General Classification of Wastewater Solids

Particle classification	Particle size, mm
Dissolved	less than 10^{-6}
Colloidal	10^{-6} to 10^{-3}
Suspended	greater than 10^{-3}
Settleable	greater than 10^{-2}

Some of the more common causative agents of offensive odors are provided in Table 6.4. A schematic that illustrates the ultimate fate of carbon, nitrogen, sulfur, hydrogen, and phosphorus under aerobic and anaerobic conditions is provided in Table 6.5 (page 6.5).

Because of the importance of estimating the oxygen-demanding potential of wastewaters, several techniques have been developed over the years. Among the more commonly utilized procedures are the biochemical oxygen demand (BOD), chemical oxygen demand (COD), and total organic carbon (TOC) tests. The BOD test is a batch reactor procedure in which microbes are typically allowed to degrade organic matter in the sample for a five-day period (BOD_5) at 20°C . Through noting the volume of sample placed in the reactor (or BOD bottle) and determining the amount of oxygen used in the bottle over the five-day period, the BOD of the waste may be calculated. Figure 6.1 illustrates that the BOD_5 of a sample is generally in the area of 60 to 70% of the total carbonaceous demand or ultimate BOD (BOD_{ult}). Figure 6.1 also shows that after about 10 to 12 days there may be an oxygen demand due to nitrification. Nitrification refers to the process in which ammonia (NH_3) is transformed into nitrites (NO_2^-) and nitrates (NO_3^-) (see Table 6.5).

In the COD test, a wastewater sample is placed in a flask containing chromic acid (dichromate ions and sulfuric acid), a strong oxidizing solution. After refluxing the sample-oxidant mixture on a burner for 2 hours, the mixture is removed and the amount of dichromate remaining in the flask is determined through a redox titration. The amount of dichromate depleted during the test is proportional to the COD of the sample.

There are a few different systems for measuring the TOC of a sample, but one of the more common ways involves injecting the wastewater into a high-temperature oven where the sample is incinerated under aerobic conditions. The carbon dioxide (CO_2) that forms in the process from the organic carbon in the sample is then measured by means of infrared spectroscopy.

Definitions for the oxygen demand parameters mentioned above and other related terms which are sometimes important are provided in Table 6.6 (page 6.6).

Pathogens. The third contaminant listed as being important in Table 6.1 is pathogens. There are a great number of organisms capable of causing disease; some of the more common agents are provided in Table 6.7 (page 6.7). Examples of bacterial, protozoan, viral, and helminthic pathogens are provided in the table.

Nutrients. There are many different micro- and macronutrients required by life forms, but the nutrients of primary concern in wastewater are carbon, nitrogen, and phosphorus. Domestic wastewaters (meaning those emanating from households as opposed to industry) generally contain more carbon than nitrogen, and more nitrogen than phosphorus, which is also the ratio of these elements required by life forms. Another nutrient that can be very important is sulfur. Schematics of anaerobic and aerobic cycles for carbon-, nitrogen-, and sulfur-containing compounds are presented in Figures 6.2 and 6.3 (pages 6.8 and 6.9).

Refractory Organics. Refractory or recalcitrant organic chemicals are also important to consider and may be defined as those materials which tend to resist biodegradation. Chlorinated pesticides, detergents with

TABLE 6.3 Solubility of Oxygen in Water (3)*

Temperature, °C	Chloride concentration in water, mg/L				
	0	5000	10,000	15,000	20,000
0	14.60	13.72	12.90	12.13	11.41
1	14.19	13.35	12.56	11.81	11.11
2	13.81	12.99	12.23	11.51	10.83
3	13.44	12.65	11.91	11.22	10.56
4	13.09	12.33	11.61	10.94	10.30
5	12.75	12.02	11.32	10.67	10.05
6	12.43	11.72	11.05	10.41	9.82
7	12.12	11.43	10.78	10.17	9.59
8	11.83	11.16	10.53	9.93	9.37
9	11.55	10.90	10.29	9.71	9.16
10	11.27	10.65	10.05	9.49	8.96
11	11.01	10.40	9.83	9.28	8.77
12	10.76	10.17	9.61	9.08	8.58
13	10.52	9.95	9.41	8.89	8.41
14	10.29	9.73	9.21	8.71	8.24
15	10.07	9.53	9.01	8.53	8.07
16	9.85	9.33	8.83	8.36	7.91
17	9.65	9.14	8.65	8.19	7.78
18	9.45	8.95	8.48	8.03	7.61
19	9.26	8.77	8.32	7.88	7.47
20	9.07	8.60	8.16	7.73	7.33
21	8.90	8.44	8.00	7.59	7.20
22	8.72	8.28	7.85	7.45	7.07
23	8.56	8.12	7.71	7.32	6.95
24	8.40	7.97	7.57	7.19	6.83
25	8.24	7.83	7.44	7.06	6.71
26	8.09	7.69	7.31	6.94	6.60
27	7.95	7.55	7.18	6.83	6.49
28	7.81	7.42	7.06	6.71	6.38
29	7.67	7.30	6.94	6.60	6.28
30	7.54	7.17	6.83	6.49	6.18
31	7.41	7.05	6.71	6.39	6.08
32	7.28	6.94	6.61	6.29	5.99
33	7.16	6.82	6.50	6.19	5.90
34	7.05	6.71	6.40	6.10	5.81
35	6.93	6.61	6.30	6.01	5.72
36	6.82	6.51	6.20	5.92	5.64

*At a total pressure of 101.3 kPa. Under any other barometric pressure P , obtain the solubility S^1 (mg/L) from the corresponding value in the table by the equation:

$$S^1 = S \frac{P - p}{760 - p}$$

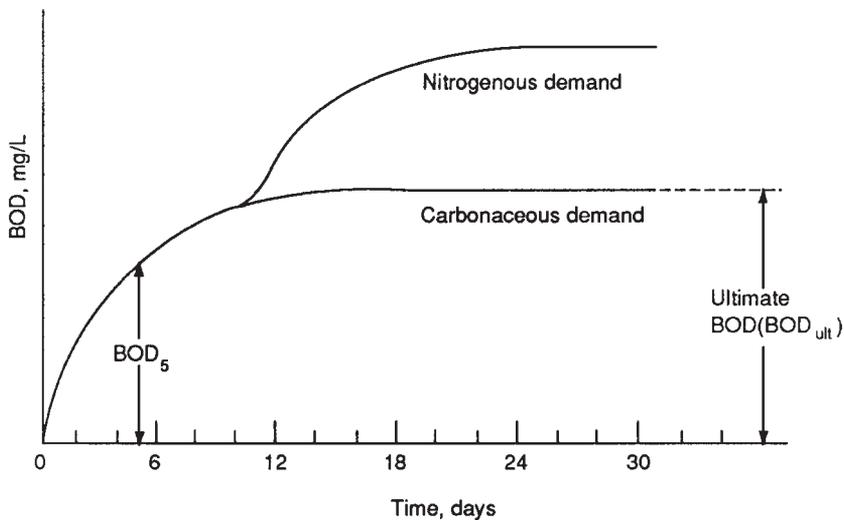
in which S is the solubility at 101.3 kPa and p is the pressure (mm) of saturated water vapor at the water temperature. For elevations less than 1000 m and temperatures below 25°C, ignore p . The equation then becomes:

$$S^1 = S \frac{P}{760}$$

Dry air is assumed to contain 20.90% oxygen.

TABLE 6.4 Major Categories of Offensive Odors (4, 5)

Compound	Typical Formula	Odor quality
Amines	CH_3NH_2 , $(\text{CH}_3)_3\text{N}$	Fishy
Ammonia	NH_3	Ammoniacal
Diamines	$\text{NH}_2(\text{CH}_2)_4\text{NH}_2$, $\text{NH}_2(\text{CH}_2)_5\text{NH}_2$	Decayed flesh
Hydrogen sulfide	H_2S	Rotten eggs
Mercaptans	CH_3SH , $\text{CH}_3(\text{CH}_2)_3\text{SH}$	Skunk
Organic sulfides	$(\text{CH}_3)_2\text{S}$, CH_3SSCH_3	Rotten cabbage
Skatole	$\text{C}_8\text{H}_5\text{NHCH}_3$	Fecal

**FIGURE 6.1** Typical biochemical oxygen demand (BOD) curve.**TABLE 6.5** Aerobic and Anaerobic End Products* (6)

Through aerobic stabilization in presence of DO	Through anaerobic decomposition in absence of DO
$\text{CO}_3^{-2} \leftarrow \text{CO}_2$	$\text{C} \rightarrow \text{CH}_4 \text{ and } \text{CO}_2$
$\text{NO}_3^- \leftarrow \text{NO}_2^- \leftarrow \text{NH}_3$	$\text{N} \rightarrow \text{NH}_3$
	$\text{SO}_4^{-2} \leftarrow \text{S} \rightarrow \text{H}_2\text{S}$
	$\text{H}_2\text{O} \leftarrow \text{H} \rightarrow \text{organic by-products or } \text{NH}_3$
	$\text{PO}_4^{-3} \leftarrow \text{P}$

*Also see Figures 6.2 and 6.3.

TABLE 6.6 Oxygen Demand and Organic Carbon Parameters (7)

BOD ₅	Biochemical or biological oxygen demand exerted in 5 days; the oxygen consumed by a waste through bacterial action; generally about 45–55% of the THOD.
COD	Chemical oxygen demand. The amount of a strong chemical oxidant (chromic acid) that is reduced by a waste; the results are expressed in terms of an equivalent amount of oxygen; generally about 80% of the THOD.
THOD	Theoretical oxygen demand. The amount of oxygen theoretically required to completely oxidize a compound to CO ₂ , H ₂ O, PO ₄ ³⁻ , SO ₄ ²⁻ , and NO ₃ .
TOC	Total organic carbon; generally about 30% of the THOD.
BOD _{ult}	The ultimate BOD exerted by a waste in an infinite time.
TOD	Immediate oxygen demand. The amount of oxygen consumed by a waste within 15 min (chemical oxidizers and bacteria not used).
DO	Dissolved oxygen. See Table 6.3. (A “negative” DO is a positive IOD.)

aryl and alkyl sulfonate groups, polyethylene, polyvinyl chloride, and fluorinated hydrocarbons are common examples of refractory compounds.

Heavy Metals. Any cation having an atomic weight greater than 23 (atomic weight of sodium) is considered a heavy metal; so, wastewaters obviously contain numerous types of heavy metals. Because many heavy metals can be toxic and/or carcinogenic, the types and levels of metals in wastewater are important to consider. Levels of various metals which are deemed permissible in freshwater and drinking water are provided in Chapter 3.

Dissolved Inorganic Solids. Because many inorganic salts will dissolve in water, dissolved inorganic solids is a very broad category. One can determine the amount of dissolved inorganic solids in wastewater by filtering a sample through a 0.45- μ m filter, collecting the filtrate, and vaporizing first the water (at 103°C) and then the organic fraction (at 550°C) from the filtrate. The amount of material left in the vessel after incineration at 550°C is referred to as the fixed or inorganic dissolved solids level.

Toxic Organic Compounds. There is also great concern that wastewaters may sometimes contain hazardous organic materials. Table 6.8 (page 6.10) provides some examples of organic compounds that are considered toxic and/or carcinogenic (also see Chapter 3).

Sources of Wastewater Contaminants

The routes taken by pollutants to reach wastewater can range from being very direct to being quite tortuous. For example, the introduction of contaminants through industrial processes and household uses is generally quite direct and is therefore rather well understood. With regard to the more indirect routes, the complexities of these sources is appreciated, but many aspects related to pollutant dispersion and transformation need to be studied further. Some examples of the more complex pathways are pollutant transport on a global basis, e.g., the distribution of DDT, nonpoint pollutant transfer, and the genesis of acid rain. Figures 6.4 and 6.5 (pages 6.11 and 6.12) more graphically describe some of the complications in determining the sources of pollutants. Figure 6.4 also serves to demonstrate that any pollutant capable of reaching a water supply or human has a good opportunity of becoming a wastewater constituent. The true significance of pollutant routing has only recently been realized.

TABLE 6.7 Pathogens Commonly Found in Wastewater (6)

Disease	Agent	Comment
Bacterial agents		
Anthrax	<i>Bacillus anthracis</i>	Spores are resistant to treatment.
Bacillary dysentery (Shigellosis)	<i>Shigella dysenteriae</i> <i>Shigella flexneri</i> <i>Shigella boydii</i> <i>Shigella sonnei</i>	Disease transmitted via water, direct contact, milk, food, and flies. Ample water for cleanliness facilitates prevention.
Brucellosis	<i>Brucella</i> spp.	Normally transmitted by infected milk or by contact. Wastewater is also a suspected transmission route.
Cholera	<i>Vibrio cholerae</i>	Initial wave of epidemic cholera is waterborne. Secondary cases and endemic cases are by contact, food, and flies.
Food poisoning	<i>Salmonella</i> spp.	Commonly isolated from wastewater.
Leptospirosis (Weil's disease)	<i>Leptospira iceterohaemorrhagiae</i>	Carried by sewer rats.
Paratyphoid fever	<i>Salmonella paratyphi</i> <i>Salmonella schottmulleri</i> <i>Salmonella hirschfeldi</i>	Few outbreaks are waterborne. Ample water facilitates cleanliness.
Tuberculosis	<i>Mycobacterium tuberculosis</i>	Wastewater is possible mode of transmission.
Tularemia	<i>Pasteurella tularensis</i>	Predominantly transmitted by infected animals and arthropods, but water route also important.
Typhoid fever	<i>Salmonella typhi</i>	Principal vehicles are water and food. Case distribution of waterborne outbreaks has a defined pattern in time and place.
Viral agents		
Infectious hepatitis	A filterable virus, recently isolated	Epidemics are due to transmission by water, milk, and food, including oysters and clams.
Poliomyelitis	Poliovirus	Actually, this is only one example from a group of over 100 enteric viruses that might be found in wastewater. Other examples include the coxsackieviruses, echoviruses, reoviruses, adenoviruses, and rotaviruses.
Helminthic agents		
Guinea worm disease (dracontiasis)	The roundworm, <i>Dracunculus medinensis</i> ; gravid female, 1 m long, migrates to skin	Unknown in North America. The infective cycle consists of the larva migrating through human skin into water where it associates with a crustacean (e.g., cyclops) and can be ingested by humans.
Nematodes	<i>Ascaris</i> spp. <i>Enterobius</i> spp.	Found in wastewater effluents and dried sludges used as fertilizer.
Schistosomiasis	<i>Schistosoma</i> spp.	Felt to be inactivated by efficient wastewater treatment operations.
Tapeworms	<i>Taenia</i> spp.	Eggs very resistant. Danger to cattle on land irrigated with wastewater sludge.

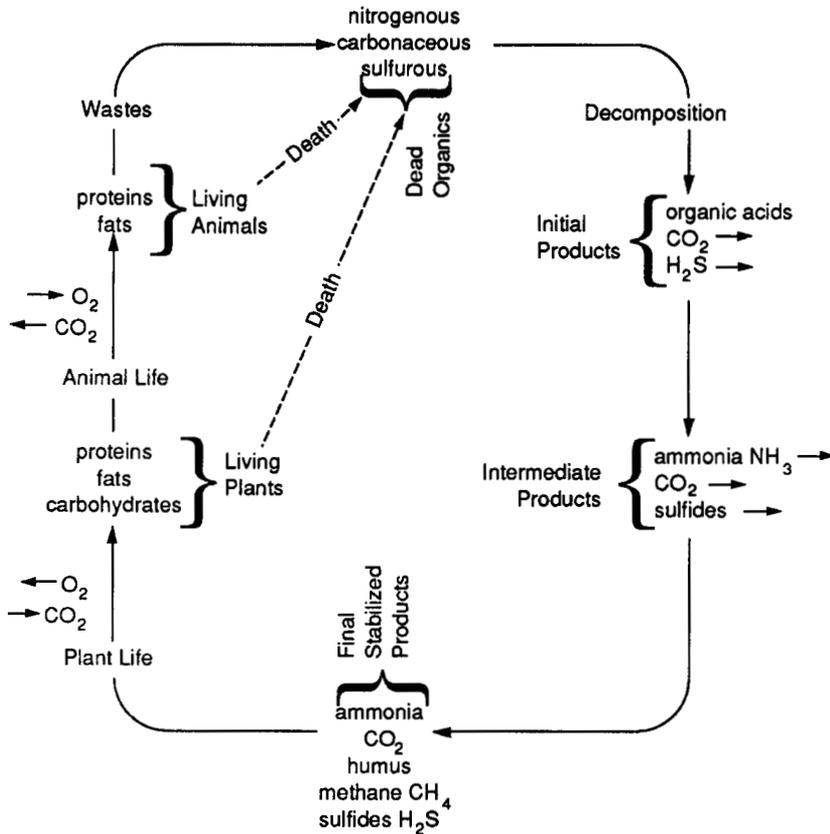


FIGURE 6.2 Anaerobic nitrogen, carbon, and sulfur cycles (8).

Information concerning the sources of various wastewater constituents of concern is presented in Table 6.9 (page 6.13), and Figure 6.6 (page 6.14) provides data comparing the relative contributions of some of the more common and important pollutants (BOD, suspended solids, nitrogen, and phosphorus) from different sources.

Effects of Wastewater Contaminants

The effects of pollutants on aquatic systems are primarily a function of the amount and nature of the contaminants introduced. For example, if a small quantity of a biodegradable agent is introduced into a large stream, the agent will be readily assimilated and have little or no impact on the stream. However, if a large quantity of the same compound were introduced, the agent might cause a dramatic shift in the population of organisms in the stream, a serious decrease in the dissolved oxygen level, etc. These concepts are commonly depicted through graphs of energy levels and oxygen sag. When biodegradable contaminant levels (or food levels) are high, the digestion rate of the food is high. As the food level is decreased and more of the energy source is transformed into stable products, the rate of microbial degradation decreases.

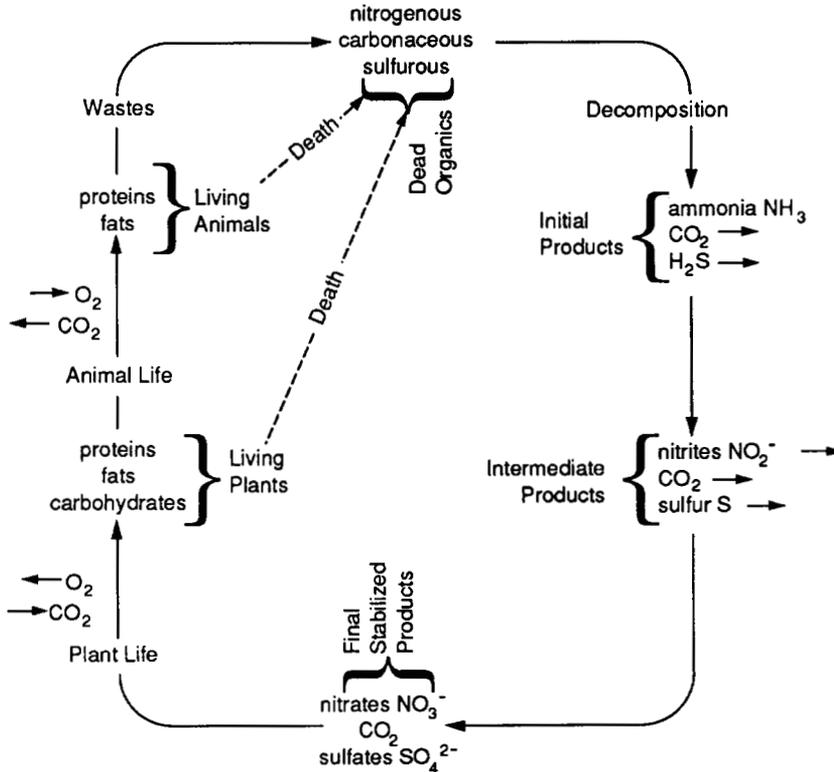


FIGURE 6.3 Aerobic nitrogen, carbon, and sulfur cycles (8).

During the aerobic decomposition of foodstuffs, microbes will use oxygen that could, if the organic loading were not compatible with the assimilative capacity of the stream, result in oxygen depletion curves similar to those depicted in Figure 6.7 (page 6.15). One of the more significant demands for dissolved oxygen in the aquatic environment is exerted by nitrogenous compounds (see Figure 6.1 and Table 6.5). In a stream environment, one might see a relationship between nitrogen species similar to that presented in Figure 6.8 (page 6.16).

Both biodegradable and toxic agents can exert lethal effects, which may be manifested in the form of decreased species diversity. The number of organisms may increase or decrease dependent on the nature of the pollutant. Figures 6.7 and 6.9 (page 6.16) illustrate how pollutants can affect the levels and types of aquatic organisms present. Table 6.9 (page 6.13) contains some important statements about the environmental effects of several wastewater contaminants.

WASTEWATER CHARACTERIZATION

The characterization of wastewater in terms of quantity and composition is primarily a function of the origin of the wastewater. The Federal Clean Water Act defines *pollutant* as meaning “dredged spoil, solid waste, incinerator residue, sewage, garbage, sewage sludge, munitions, chemical wastes, biological materials, ra-

TABLE 6.8 Occupational Exposure to Carcinogenic Substances (6, 9)

Compound	Site	Comment
1. Organic substances for which there is wide agreement on carcinogenicity		
4-Aminodiphenyl	Bladder	A contaminant in diphenylamine
Benzidine	Bladder	Ingredient of aniline dyes, plastics, and rubber
Beta-naphthylamine (2-NA)	Bladder	Dye and pesticide ingredient; synonym, 2-naphthylamine; exposed workers have 30 to 60 times more bladder cancer
Bis(chloromethyl)ether	Lung	Used in making exchange resins; exposed workers have seven times more lung cancer; synonym, BCME
Vinyl chloride	Liver	Angiosarcoma cases among PVC workers
2. Additional organic substances on USDA-OSHA cancer-causing substances list		
Alpha-naphthylamine (1-NA)	Bladder	Human carcinogenesis implicated; used in making dyes, herbicides, food colors, color film; an antioxidant
Ethyleneimine	Unknown	Carcinogenic in animals; used in paper and textile processing and making of herbicides, resins, rocket and jet fuels
3,3'-Dichlorobenzidine	Unknown	Carcinogenic in animal species; exposure accompanies benzidine and beta-naphthylamine
Methyl chloromethyl methyl ether	Unknown	Carcinogenic in animals; synonym, CMME; BCME contaminates CMME; used in resin-making, textile, and drug production
4,4'-Methylene bis (2-chloroaniline)	Unknown	Synonym MOCA. Tumorigenic in rats and mice. Skin absorption may be the hazard. Curing agent for isocyanate polymers
3. Industrial substances suspected of carcinogenic potential for humans		
Antimony trioxide production		Epichlorhydrin
Benzene (skin)		Hexamethyl phosphoramidate (skin)
Benz(a)pyrene		Hydrazine
Beryllium		4,4'-Methylene bis (2-chloroaniline) (skin)
Cadmium oxide production		4,4'-Methylene dianiline
Chloroform		Monomethyl hydrazine
Chromates of lead and zinc		Nitrosamines
3,3'-Dichlorobenzidine		Propane sulfone
Dimethylcarbamyl chloride		Beta-propiolactone
1,1'-Dimethyl hydrazine		Vinyl cyclohexene dioxide
Dimethyl sulfate		

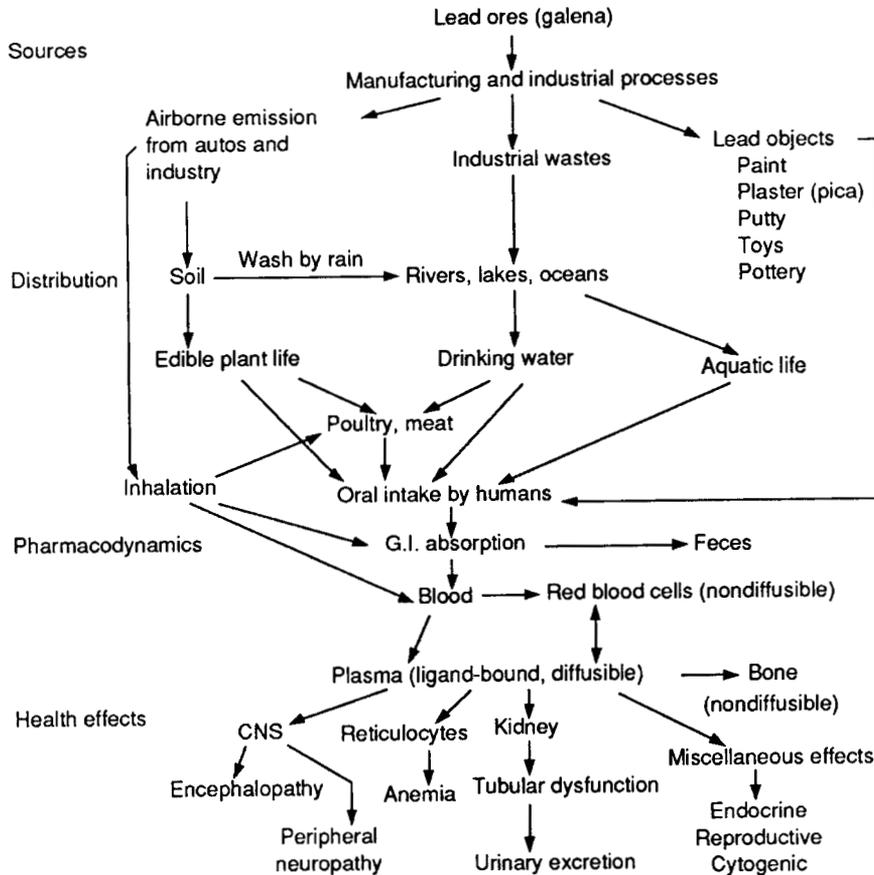


FIGURE 6.4 Environmental pathways and health effects of lead (10).

diocative materials, heat, wrecked or discarded equipment, rock, sand, cellar dirt, and industrial, municipal, and agricultural waste discharged into water” (15). Despite this broad, all-encompassing definition, wastewater can typically be characterized as either domestic, industrial, or some combination of both. Accordingly, this section has been structured to provide the essential aspects of wastewater characterization in two elements; namely, domestic wastewater and industrial wastewater.

Domestic Wastewater

Domestic wastewater is the spent water originating from all aspects of human sanitary water usage. It typically constitutes a combination of flows from the kitchen, bathroom, and laundry, encompassing laboratories, toilets, baths, kitchen sinks, garbage grinders, dishwashers, washing machines, and water softeners. Domestic wastewater, as the name implies, principally originates in residences and is also referred to as sanitary sewage. As such, commercial, institutional, and industrial establishments contribute a domestic wastewater component to the sewer system resulting from human sanitary activity. The characterization of domes-

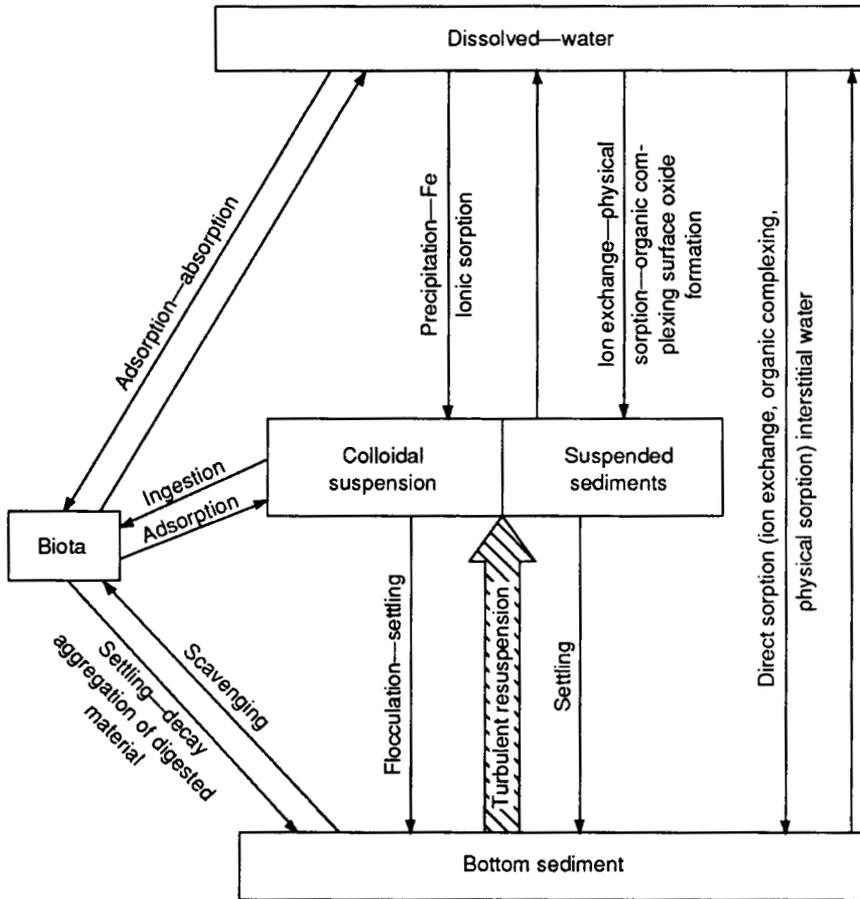


FIGURE 6.5 Hydrosphere and biosphere transfer routes (11).

tic wastewater involves an examination of both essential elements of all wastewater characterization; namely, composition and flow characteristics.

Composition. The composition of domestic wastewater refers to the actual quantities of physical, chemical, and biological constituents present (4). The water supplied to a community will typically contain mineral and organic matter, to which the human sanitary and domestic activity adds feces, urine, paper, soap, dirt, food wastes, minerals from water softeners, and other substances. Some of these waste materials remain in suspension, while others go into solution or become so finely divided that they become colloidal in nature (17). The physical, chemical, and biological constituents present in typical domestic wastewater are shown in Table 6.10 (page 6.17).

Physical and Chemical Constituents. The concentration of physical and chemical pollutants in domestic wastewater is quite variable and particularly dependent upon local conditions. In addition, the concentration of individual constituents is normally expected to vary with the time of the day, the day of the week, and the month of the year. This variation results from the normal cycle of human activity during a given day, the dif-

TABLE 6.9 Typical Sources and Effects of Wastewater Constituents (6)

Component group	Effects	Typical sources
Biooxidizables expressed as BOD ₅	Deoxygenation, anaerobic conditions, fish kills, odors	Large amounts of soluble carbohydrates: sugar refining, canning, distilleries, breweries, milk processing, pulping, and paper making
Primary toxicants: As, CN, Cr, Cd, Cu, F, Hg, Pb, Zn	Fish kills, cattle poisoning, plankton kills, accumulations in flesh of fish and mollusks	Metal cleaning, plating, pickling; phosphate and bauxite refining; chlorine generation; battery making; tanning
Acids and alkalines	Disruption of pH buffer systems, disordering previous ecological system	Coal-mine drainage, steel pickling, textiles, chemical manufacture, wool scouring, laundries
Disinfectants: Cl ₂ , H ₂ O ₂ , formalin, phenol	Selective kills of microorganisms, taste, and odors	Bleaching of paper and textiles; rocketry; resin synthesis; penicillin preparation; gas, coke, and coal-tar making; dye and chemical manufacture
Ionic forms: Fe, Ca, Mg, Mn, Cl, SO ₄	Changed water characteristics: staining, hardness, salinity, encrustations	Metallurgy, cement making, ceramics, oil-well pumpage
Oxidizing and reducing agents: NH ₃ , NO ₂ ⁻ , NO ₃ ⁻ , S ⁻ , SO ₃ ⁻²	Altered chemical balances ranging from rapid oxygen depletion to overnutrition, odors, selective microbial	Gas and coke making, fertilizer plants, explosive manufacture, dyeing and synthetic fiber making, wood pulping, bleaching growths
Evident to sight and smell	Foaming, floating, and settleable solids; odors; anaerobic bottom deposits; oils, fats, and grease; waterfowl and fish injuries	Detergent wastes, tanning, food and meat processing, beet sugar mills, woolen mills, poultry dressing, petroleum refining
Pathogenic organisms: <i>B. anthracis</i> , <i>Leptospira</i> , fungi, viruses	Infections in humans, reinfection of livestock, plant diseases from fungi-contaminated irrigation water; risks to humans slight	Abattoir wastes, wool processing, fungi growths in waste treatment works, poultry-processing waste waters

ference between weekday and weekend domestic water usage, and fluctuations with the season of the year. The normal range of concentrations and average values for the pollutants encountered in domestic wastewater are presented in Table 6.11 (page 6.17).

In addition to the pollutants introduced into domestic wastewater by human sanitary activity, there is also typically an increase in mineral content experienced with the use of water for domestic purposes. This mineral pickup results from ordinary domestic use, from the addition of highly mineralized water from private wells and groundwater, and from water softening, which in many areas may represent the major factor in this phenomenon (4). Data on the typical range of increase in the mineral content resulting from domestic water usage are provided in Table 6.12 (page 6.18).

Biological Constituents. The biological constituents found in domestic wastewater are extremely impor-

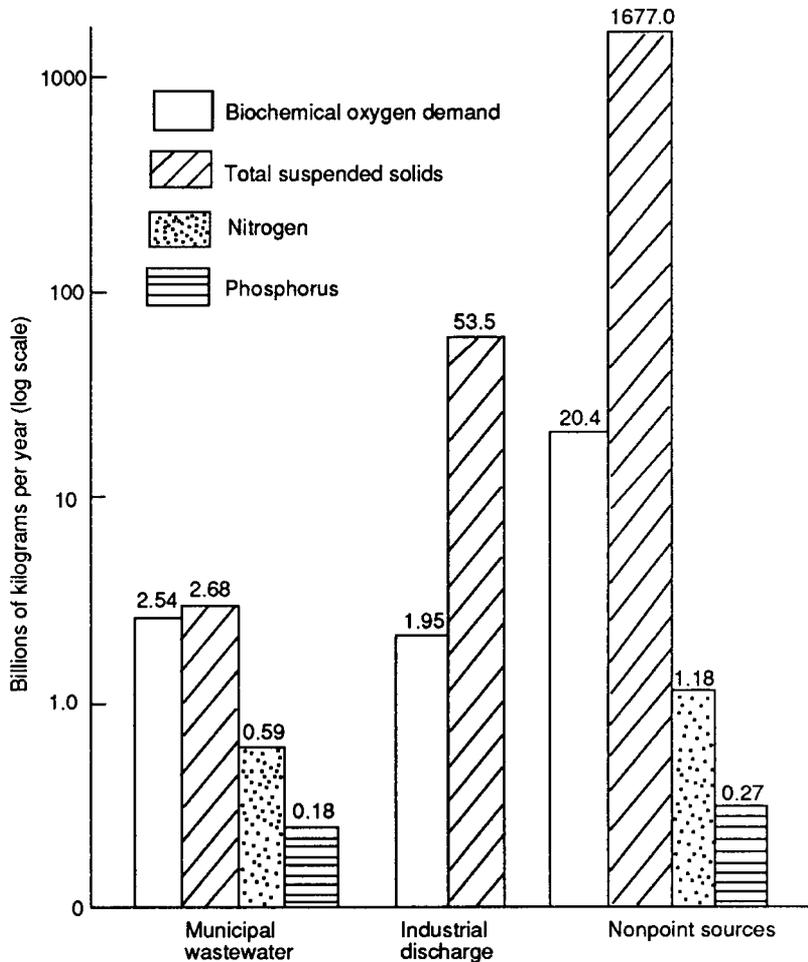


FIGURE 6.6 Estimated nationwide loadings of pollutants (12, 13).

tant, due to the public health considerations involved, which stem from the source and nature of the microorganisms present. Many of the waste substances present in domestic wastewater are organic and serve as food for saprophytic microorganisms that live on dead organic matter. As a result, domestic wastewater is unstable, biodegradable, and putrescible (17).

In addition to the microorganisms that enter domestic wastewater at its source, many also come from the soil by infiltration into the collection system. Thus, it is possible to find almost every type of microorganism in sewage. Approximately 10^5 bacteria per milliliter will be found in fresh domestic wastewater, whereas the bacteria count will rise to 10^7 to 10^8 bacteria per milliliter in stale sewage (18).

Of the microorganisms present in domestic wastewater, bacteria, protozoa, fungi, algae, and viruses are of the greatest significance to the environmental engineer. Bacteria play a fundamental role in the decomposition and stabilization of organic matter both in nature and in wastewater treatment plants. Additionally, of

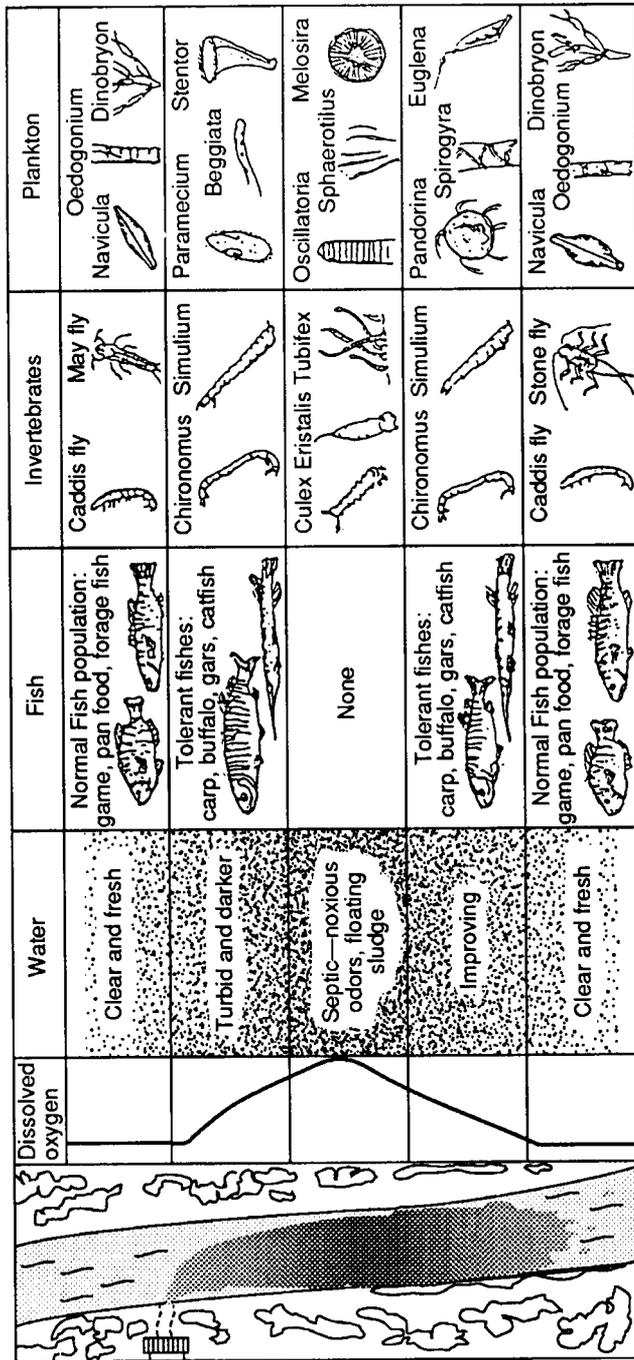


FIGURE 6.7 Effects of stream pollution and recovery (14).

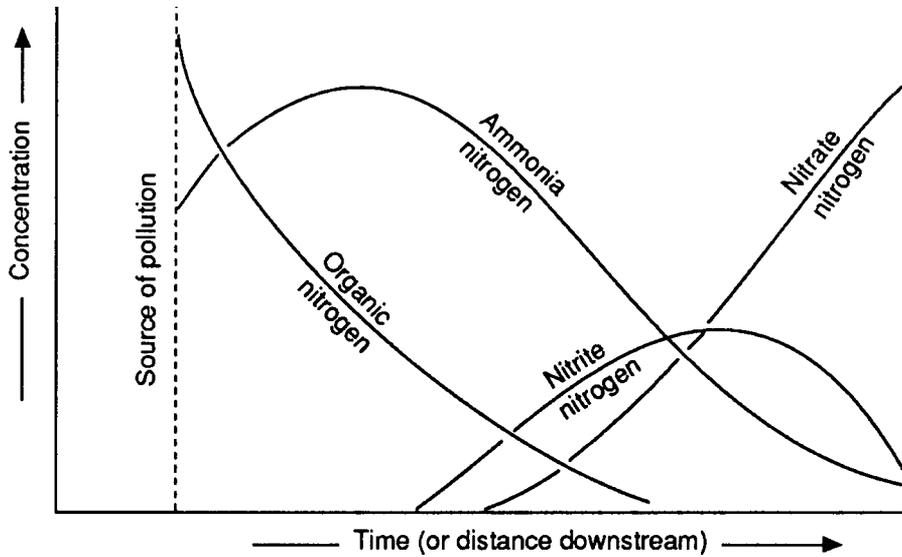


FIGURE 6.8 Typical reaction of nitrogen compounds downstream of organic pollution (8).

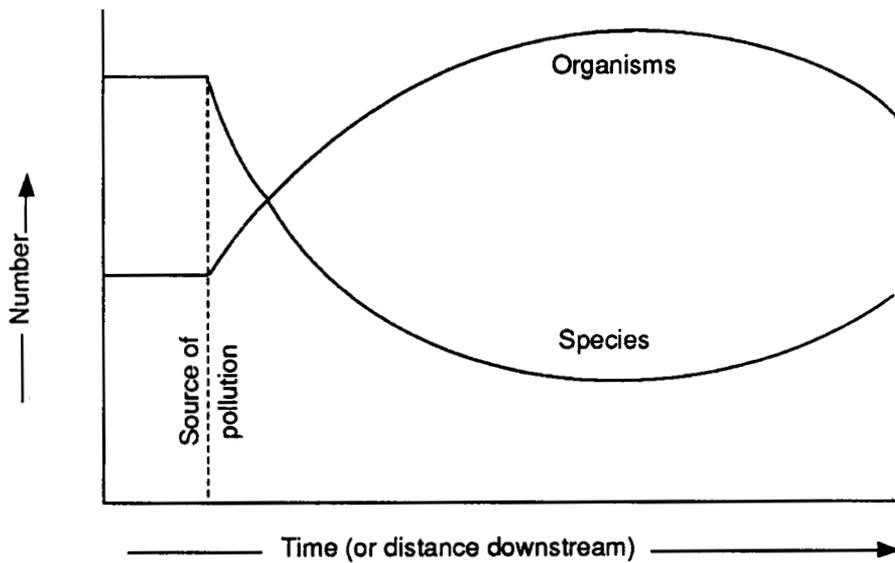


FIGURE 6.9 Species population and organism diversity downstream of pollution (8).

TABLE 6.10 Physical, Chemical, and Biological Constituents of Domestic Wastewater (16)

Physical	Chemical	Biological
Solids	Organics	Plants
Temperature	Proteins	Animals
Color	Carbohydrates	Viruses
Odor	Fats and oils	
	Surfactants	
	Inorganics	
	pH	
	Chlorides	
	Alkalinity	
	Nitrogen	
	Phosphorus	
	Heavy metals	
	Gases	
	Oxygen	
	Hydrogen sulfide	
	Methane	

TABLE 6.11 Typical Composition of Untreated Domestic Wastewater (4)

Pollutant	Concentration (mg/L, except settleable solids)	
	Range	Average
Solids, total:	350–1200	720
Dissolved, total*	250–850	500
Fixed	145–525	300
Volatile	105–325	200
Suspended, total	100–350	220
Fixed	20–75	55
Volatile	80–275	165
Settleable solids, ml/L	5–20	10
Biochemical oxygen demand, 5-day, 20°C (BOD ₅ , 20°C)	110–400	220
Total organic carbon (TOC)	80–290	160
Chemical oxygen demand (COD)	250–1000	500
Nitrogen (total as N):	20–85	40
Organic	8–35	15
Free ammonia	12–50	25
Nitrites	0	0
Nitrates	0	0
Phosphorus (total as P):	4–15	8
Organic	1–5	3
Inorganic	3–10	5
Chlorides*	30–100	50
Alkalinity (as CaCO ₃)*	50–200	100
Oil and grease	50–150	100

*Values should be increased by amount in domestic water supply.

TABLE 6.12 Typical Increase in Mineral Content from Domestic Water Use (4)

Constituent	Increment range,* mg/L
Anions	
Bicarbonate (HCO ₃)	50–100
Carbonate (CO ₃)	0–10
Chloride (Cl)	20–50 [†]
Nitrate (NO ₃)	20–40
Phosphate (PO ₄)	20–40
Sulfate (SO ₄)	15–30
Cations	
Calcium (Ca)	15–40 [‡]
Magnesium (Mg)	15–40 [‡]
Potassium (K)	7–15
Sodium (Na)	40–70
Other data	
Aluminum (Al)	0.1–0.2
Boron (B)	0.1–0.4
Iron (Fe)	0.2–0.4
Manganese (Mn)	0.2–0.4
Silica (SiO ₂)	2–10
Total alkalinity	100–150 [‡]
Total dissolved solids (TDS)	150–400

*Reported national average range of mineral pickup by domestic usage. Does not include commercial and industrial additions.

[†]Excluding the addition from domestic water softeners.

[‡]Reported as CaCO₃.

the pathogenic organisms found in domestic wastewater, bacteria are the most numerous and are responsible for diseases of the gastrointestinal tract, such as typhoid and paratyphoid fever, dysentery, diarrhea, and cholera (4). These pathogenic microorganisms are excreted by human beings who are infected with, or are carriers of, a particular disease. Since the identification of pathogens in wastewater is extremely difficult and time consuming, coliform bacteria are utilized in the analysis of wastewater for the presence of human wastes and hence pathogenic organisms.

Protozoa feed on bacteria and are essential in the operation of biological treatment processes and in the purification of streams, since they maintain a natural balance among the different groups of microorganisms (4). Fungi are “molds” that have the ability to survive at low pH values, making them significant in biologically treating some industrial wastes and composting solid organic wastes. Algae are significant because of their ability to reproduce rapidly under the proper conditions and create algal blooms in surface waters. In addition to the nuisance associated with such blooms, algae also create taste and odor problems when present in water, thus diminishing its value for water supply purposes.

There is increasing awareness that enteric viruses can be waterborne and that viruses that are excreted by human beings may become a major hazard to public health. A great deal of current study is directed toward identifying and quantifying viruses in wastewater and the mechanisms of travel and removal in various media and treatment facilities. It is known that some viruses will live as long as 41 days in water or wastewater, and six days in a normal river environment. A number of outbreaks of infectious hepatitis have been traced to transmission of the virus through water supplies (4).

Flow Characteristics. Since domestic wastewater is a by-product of life and living processes, the quantity of sewage generated in a given area is dependent on the number of people residing in the area and the contribution of each individual to the sewer system. Therefore, there are two essential components to be considered in establishing average wastewater flow rates for a given area: population and per capita sewage contribution. In addition, the fluctuations normally experienced on an hourly, daily, and monthly basis must be taken into account in any consideration of domestic wastewater flow characteristics. These factors are most significant for the designer faced with the problem of establishing a design basis for collection and treatment facilities.

Water Supply Records. To establish flow characteristics for an existing sewer system, direct field measurements and review of existing records are advised. For new systems where this is not possible, public water supply records are often used as an aid in estimating wastewater flows. In utilizing water usage records, it is necessary to keep in mind that there is an amount of water that is actually consumed; this constitutes the difference between the quantity supplied and that discharged as sewage. This consumptive use refers to the volume of water lost, evaporated, transpired, or otherwise consumed. It typically includes leakage, street washing, lawn sprinkling, fire fighting, and water used by industry in raising and condensing steam, bottling, canning, and other industrial operations (17). Studies have indicated that for residential areas, approximately 30% of the total quantity of water supplied is consumed, and the remaining 70% is discharged to the sewer as domestic wastewater (4).

A detailed study conducted by the U.S. Geological Survey in 1970 indicated that the average quantity of water withdrawn for public water supplies in the United States was about 166 gal per capita per day (628 L per capita per day) (22). The study reported data for specific municipal systems in all 50 states and showed a wide variation in withdrawal rates depending upon geographic location, climate, community size, extent of industrialization, and other factors. The results of the study serve to emphasize the need to obtain and analyze data for a particular area being evaluated if at all possible, instead of relying on average values.

Per-Capita Sewage Flow. Use of the data cited above yields an average per capita sewage flow of 116 gal/day (628×0.7 , or 440 L/day). However, as indicated, there is no normal per capita sewage flow that is applicable to all communities. Variations are due not only to location, climate, and community size, but also to such related factors as (1) standard of living, (2) water pricing, (3) water quality, (4) infiltration, (5) distribution system pressure, (6) extent of meterage, and (7) systems management (17). Per capita sewage flow can be expected to typically vary between 75 to 125 gal/day (285 and 475 L/day) depending upon all of the factors indicated (20).

Review of historical data clearly shows an upward trend in the use of water in the home, and a consequent increase in per capita domestic wastewater flow. Modern household appliances, such as garbage grinders, dishwashers, automatic clothes washers, and the increased use of home water softeners have all contributed to this trend. The continuation of this tendency to use more water in the home will depend to a large extent on the availability of water in a given community, changes in lifestyle and standard of living, and perhaps even on the development of new home water-consuming devices. Typical water flow rates for various devices currently in use are provided in Table 6.13. An estimate of the source of wastewater within an average home breaks down as follows: 35% from showers, washbasins, etc.; 30% toilet wastes; 20% laundry wastes; and 15% kitchen wastes (23).

As discussed above, commercial and industrial establishments contribute to the total domestic wastewater flow from a particular community. Where flow records are not available and direct measurement is not possible, other methods must be used to estimate this flow component. For establishments having bedrooms, such as hotels, motels, and rooming and boarding houses, sewage flow may be estimated based on the number of bedrooms involved. For restaurants, the number of patrons or number of meals served may be the best criterion. Another frequently used method is to base sewage flow on the number and type of plumbing fixtures in the facility in question. Experience with sewage flow from specific types of establishments has led to the development of Table 6.14, which provides typical average domestic wastewater quantities for a variety of commercial facilities.

Population Projections. The development of population estimates in recent years has become an exacting

TABLE 6.13 Typical Rates of Water Use for Various Devices (4, 42)

Device	Range of flow	
	SI units	Customary units
Automatic home laundry machine	110–200 L/load	29–53 gal/load
Automatic home-type dishwasher	15–30 L/load	4–8 gal/load
Automatic home-type washing machine	130–200 L/use	34–53 gal/use
Bathtub	90–110 L/use	24–29 gal/use
Continuous-flow drinking fountain	4–5 L/min	1 gal/min
Dishwashing machine, commercial*		
Conveyor type, at 100 kN/m ²	15–25 L/min	4–7 gal/min
Stationary rack type, at 100 kN/m ²	25–35 L/min	7–9 gal/min
Fire hose, 38 mm, 13-mm nozzle, 20-m head	140–160 L/min	37–42 gal/min
Garbage disposal unit, home-type	6000–7500 L/wk	1600–2000 gal/wk
Garbage grinder, home-type	4–8 L/person/day	1–2 gal/person/day
Garden hose, 16 mm, 8-m head	10–12 L/min	3 gal/min
Garden hose, 19 mm, 8-m head	16–20 L/min	4–5 gal/min
Lawn sprinkler	6–8 L/min	2 gal/min
Lawn sprinkler, 280 m ² lawn, 25 min/wk	6000–7500 L/wk	1600–2000 gal/wk
Shower head, 16 mm, 8-m head	90–110 L/use	24–29 gal/min
Washbasin	4–8 L/use	1–2 gal/use
Water closet, flush valve, 170 kN/m ²	90–110 L/min	24–29 gal/use
Water closet, tank	15–25 L/use	4–7 gal/use

*Does not include water to fill wash tank.

science, worked out with great care and in considerable detail by demographers. Such data today are usually available from local, regional, and state planning agencies, whereas in the past, environmental engineers were typically responsible for developing their own projections. If planning agency information is not available, a number of different methods may be employed for projecting future population. These include (21)

- Arithmetical increase per year or per decade
- Uniform percentage rate of growth based on recent census periods
- Decreasing percentage rate of increase
- Graphical comparison with the growth of other similar but larger cities
- Graphical extension of the curve of past growth
- Verhulst's theory (logistic trend method)
- Ratio and correlation
- Component analysis
- Employment forecast

The essential data required for projecting future population may be obtained from a variety of sources, including U.S. census reports, chamber of commerce studies, voter registration lists, school census statistics, and installation records from local telephone, electricity, gas, water, and sewer utilities. Future population trends depend on many factors, some known and some unknown. Known factors include

- Location with respect to transportation facilities for workers, raw materials, and manufactured products
- Possible expansion of present industries
- Availability of sites for residential, commercial, or industrial development

TABLE 6.14 Quantities of Sewage Flow (23)

Type of establishment	Liters per person per day	Gallons per person per day
Small dwellings and cottages with seasonal occupancy	190	50
Single-family dwellings	285	75
Multiple-family dwellings (apartments)	227	60
Rooming houses	150	40
Boarding houses	190	50
Additional kitchen wastes for nonresident boarders	38	10
Hotels without private baths	190	50
Hotels with private baths (2 persons per room)	227	60
Restaurants (toilet and kitchen wastes per patron)	26–38	7–10
Restaurants (kitchen wastes per meal served)	9–11	2½–3
Additional for bars and cocktail lounges	8	2
Tourist camps or trailer parks with central bathhouse	132	35
Tourist courts or mobile home parks with individual bath units	190	50
Resort camps (night and day) with limited plumbing	190	50
Luxury camps	380–570	100–150
Work or construction camps (semipermanent)	190	50
Day camps (no meals served)	57	15
Day schools without cafeterias, gymnasiums, or showers	57	15
Day schools with cafeterias, but no gymnasiums or showers	75	20
Day schools with cafeterias, gyms, and showers	95	25
Boarding schools	285–380	75–100
Day workers at schools and offices (per shift)	57	15
Hospitals	570–945+	150–250+
Institutions other than hospitals	285–475	75–125
Factories (gallons per person per shift, exclusive of industrial wastes)	57–132	15–35
Picnic parks (toilet wastes only, per picnicker)	19	5
Picnic parks with bathhouses, showers, and flush toilets	38	10
Swimming pools and bathhouses	38	10
Luxury residences and estates	380–570	100–150
Country clubs (per resident member)	380	100
Country clubs (per nonresident member present)	95	25
Motels (per bed space)	150	40
Motels with bath, toilet, and kitchen wastes	190	50
Drive-in theaters (per car space)	19	5
Movie theaters (per auditorium seat)	19	5
Airports (per passenger)	11–19	3–5
Self-service laundries (per wash, i.e., per customer)	190	50
Stores (per toilet room)	1500	400
Service stations (per vehicle served)	38	10

- Civic interest in community growth
- Availability of other utility services at reasonable rates
- Real estate values

Nevertheless, population estimates based on all known factors can be upset by extraordinary events such as the discovery of some new natural resource in the vicinity or the decision by a large manufacturer to locate in the community (21).

Detailed information regarding population projection methods and procedures is presented in a number of textbooks on sewer design. A typical plot of past census data for a small city is shown in Figure 6.10, which also includes estimates of future population made from the number of electric meters and water meters for each year subsequent to the last regular census year. The probable future population growth is indicated by two limiting lines, as shown, because of the difficulty of predicting future population with any great degree of accuracy. The records of the average daily and per capita water consumption and projected trends for both are also shown (21).

Flow Variation. The variation in domestic wastewater flow rates occurs on an hourly, daily, and monthly basis. Hourly changes in flow over a 24-h period are generally predictable in that they follow the daily cycle of life, with minimum flows occurring in the early morning around 4 A.M., and peak flows occurring in the late morning and early evening. This phenomenon is referred to as the diurnal cycle, and is shown graphically in Figure 6.11. Approximately one-half of the sewage generated in the course of a day arrives at the treatment plant over an 8-h period (16).

These peak volumes are generally assumed to be at least twice the average flow, but for hydraulic design purposes, a factor of 250% is frequently used. For lateral sewers, design factors of three to four times average flow rates are not uncommon. This relationship depends on the character of the community, in terms of its true sanitary sewage flow, without taking into consideration any stormwater inflow that could occur during periods of heavy rain or thaw. The smaller the community sewer system, the greater the fluctuation of flow from hour to hour and from day to day. The larger the system, the greater the ironing-out effect of the longer and larger conduits and the flows from "stable" sewage contributors in the larger service area (20).

Certain factors of community life tend to magnify peak flow conditions, such as the uniformity of living habits in a suburban residential area where everyone arises and leaves for work at the same time, where day-time home habits are uniform, and dinner becomes a universally timed event in most homes. There are, however, compensating factors which minimize peak flows, such as the frequent and randomly timed washing in home laundry equipment (20).

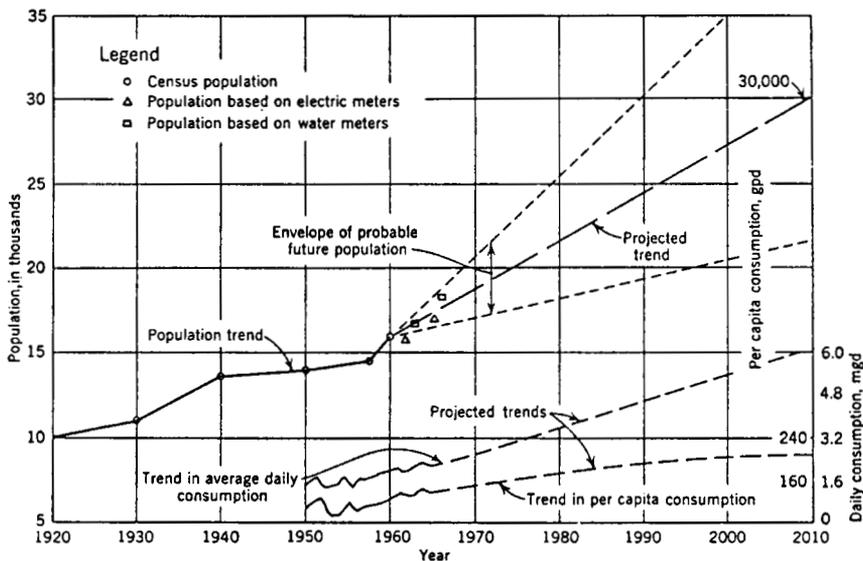


FIGURE 6.10 Population and water consumption trends (Gpd per capita $\times 3.8 =$ L/day per capita; $\text{mgd} \times 3785 = \text{m}^3/\text{day}$) (21).

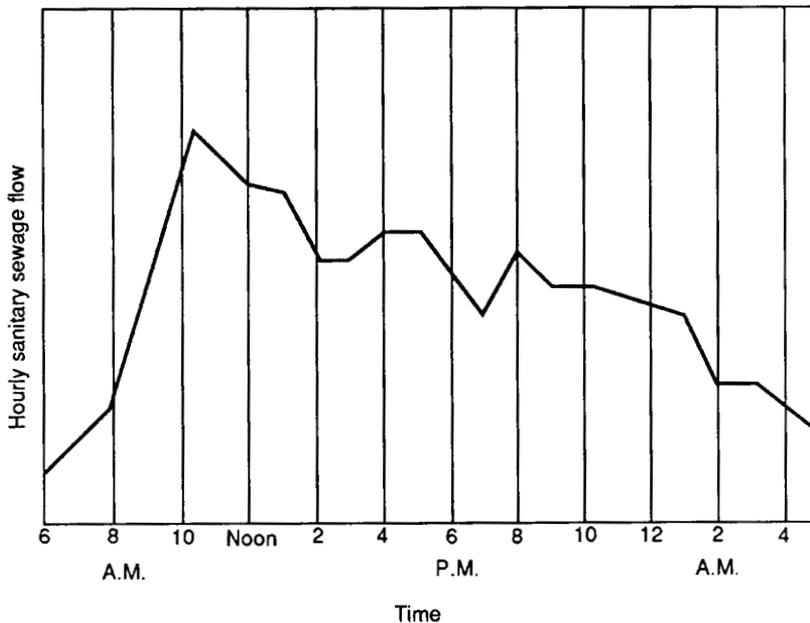


FIGURE 6.11 Typical hourly variations in sewage flow (20).

Variation in domestic wastewater flow from day to day will largely be observed as differences between weekday and weekend flow rates. These differences will again be dependent on the life-style of the community in question. Seasonal variations are common at resort areas, in small communities with college campuses, and other similar special cases. The effect of climate can be substantial, particularly with regard to inflow and infiltration. In areas where rainfall patterns tend to be cyclic, the seasonal effect of inflow and infiltration is particularly evident.

The variability of total flow within a community's sewer system is especially affected by industrial wastewaters discharged into the system. The flow rate of industrial effluents is related to a number of factors which can cause extreme variations in flow as discussed below.

Industrial Wastewater

Industrial wastewater emanates from the myriad of industrial processes that utilize water for a variety of purposes. The U.S. Department of Commerce estimates that major industrial water users discharge approximately 285 billion gallons (1080 billion liters) of wastewater daily (24). A graphic illustration of the relative discharges from various segments of society, showing the significance of the quantity of wastewater discharged by the industrial sector, is shown in Figure 6.12. About two-thirds of the total wastewater generated by U.S. manufacturing plants results from cooling operations. 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 (17).

In addition to the large quantities used for cooling purposes, water is used extensively throughout industry in process operations. This water is usually altered considerably in the industrial process and may contain contaminants that degrade the water quality such as nutrients, suspended sediments, bacteria, oxygen-

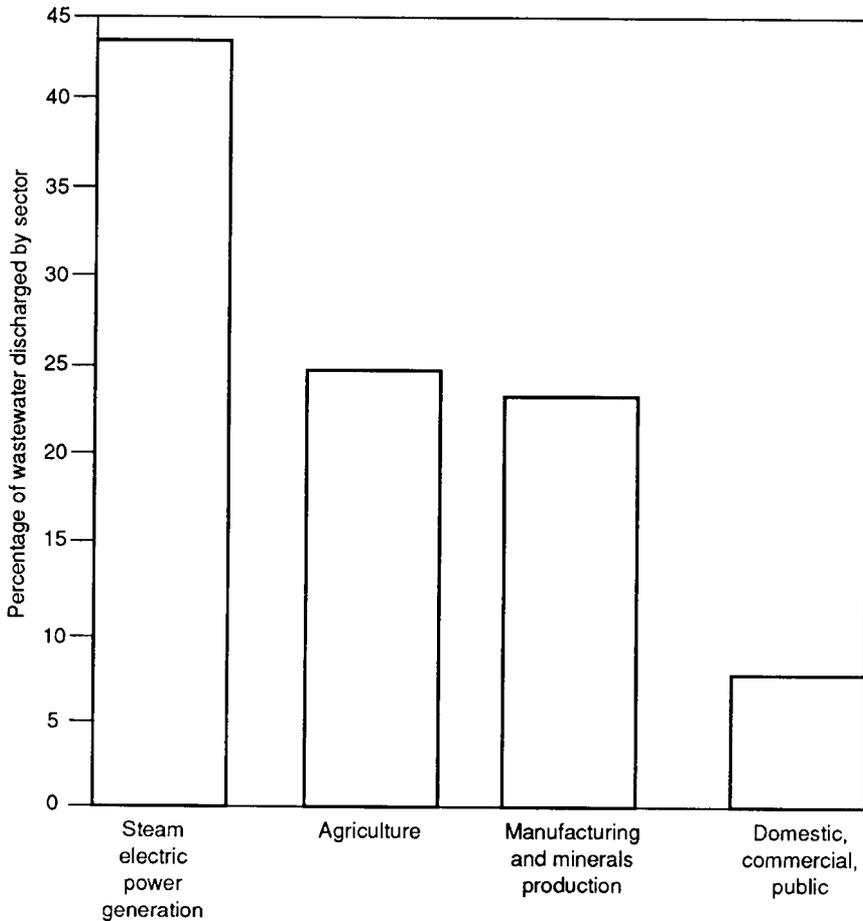


FIGURE 6.12 U.S. wastewater discharges (24).

demanding matter, and perhaps toxic substances as summarized in Table 6.9 of the previous section. A flow diagram for a hypothetical plant complex, illustrating water flows and some of the more typical treatment and processing steps that might be used is presented in Figure 6.13. In this case, raw water is taken in from a river, and treated effluent returns to the same river, with several possibilities for raw-water and effluent treatment being shown (25). The diversity of manufacturing processes employed by industry makes any general characterization of industrial wastewater impossible. The quantity of water used by industrial facilities varies widely, with some plants utilizing in excess of 50 mgd (189,000 m³/day) while others use no more than comparably sized mercantile establishments (17). In addition, variability in industrial wastewater flow rates can be extreme, since there are a great many factors within a given facility (batch versus continuous operation, for example) that affect this characteristic.

The composition of industrial effluents is equally as variable as flow rates and dependent to a large degree on the particular industry in question. Summary type data for the significant pollutant parameters typically found in the discharge from major industries are presented in Tables 6.15 and 6.16. However, it must be

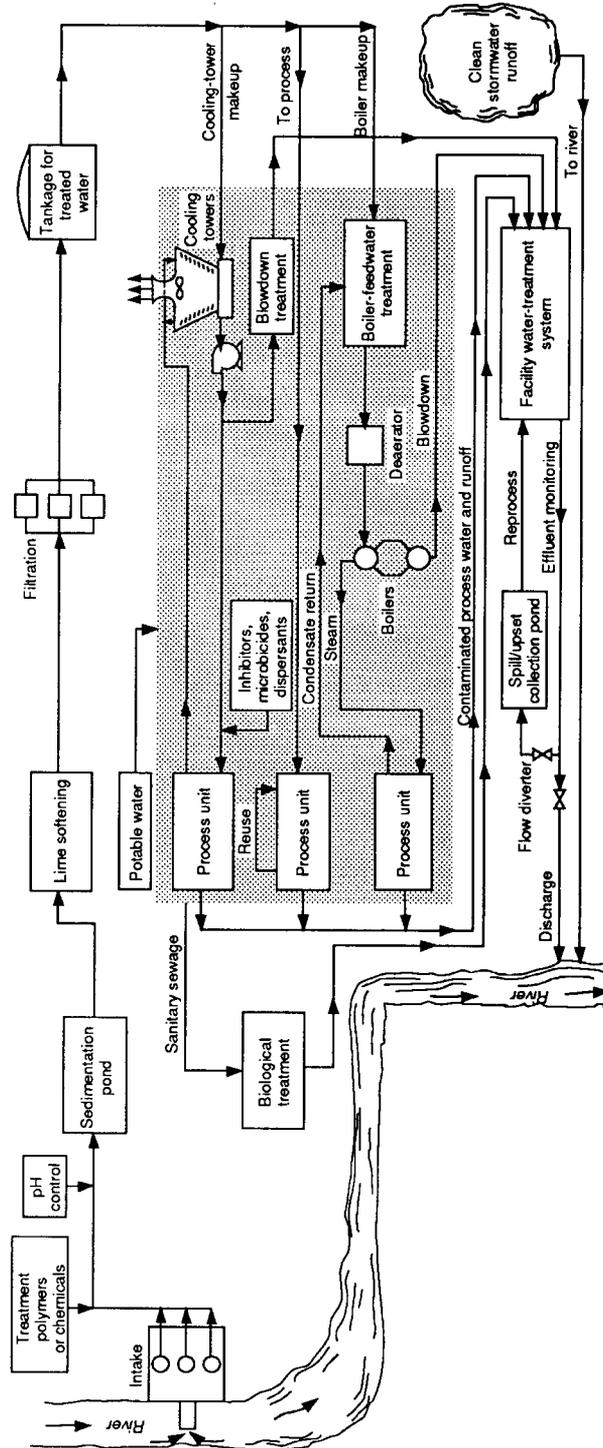


FIGURE 6.13 Water use flow diagram for a hypothetical industrial plant (2.5).

TABLE 6.15 Parameters Setting the Standards for the Discharge of Industrial Wastewater (43)

Parameter	Industry													
	Auto- mobile	Bever- age	Can- ning	Fertil- izer	Inor- ganic chemi- cals	Organic chemi- cals	Meat prod- ucts	Metal finish- ing	Plas- tics and syn- thetics	Pulp and paper	Petro- leum and refin- ing	Steel	Textiles	Dairy
BOD ₅	x	x	x		x	x	x		x	x	x	x	x	x
COD	x		x	x	x	x		x	x	x	x	x	x	x
TOC										x	x			x
TOD					x									
pH		x	x	x	x	x	x		x	x	x	x	x	x
Total solids	x				x	x						x		
Suspended solids	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Settleable solids		x												
Total dissolved solids		x	x	x	x	x	x	x	x	x	x	x	x	x
Volatile suspended solids														
Oil and grease	x	x		x		x	x	x	x	x	x	x	x	x
Heavy metals, general						x		x		x	x	x	x	x
Chromium	x			x	x						x	x	x	x
Copper														
Nickel	x													
Iron	x			x	x								x	x
Zinc	x			x					x					x
Arsenic														
Mercury				x	x									
Lead	x				x									x

TABLE 6.16 Major Water Contaminants from Some Industrial Sources (44)

Industry	Origin of contaminants	Components and characteristics of contaminants
Food		
Canning	Fruit and vegetable preparation	Colloidal, dissolved organic matter, suspended solids
Dairy	Whole milk dilutions, buttermilk	Dissolved organic matter (protein, fat, lactose)
Brewing, distilling	Grain, distillation	Dissolved organics, nitrogen fermented starches
Meat, poultry	Slaughtering, rendering of bones and fats, plucking	Dissolved organics, blood, proteins, fats, feathers
Sugar beet	Handling juices, condensates	Dissolved sugar and protein
Yeast	Yeast filtration	Solid organics
Pickles	Lime water, seeds, syrup	Suspended solids, dissolved organics, variable pH
Coffee	Pulping and fermenting beans	Suspended solids
Fish	Pressed fish, wash water	Organic solids, odor
Rice	Soaking, cooking, washing	Suspended and dissolved carbohydrates
Soft drinks	Cleaning, spillage, washing	Suspended and dissolved carbohydrates
Pharmaceutical		
Antibiotics	Mycelium, filtrate, washing	Suspended and dissolved organics
Clothing		
Textiles	Desizing of fabric	Suspended solids, dyes, alkaline
Leather	Cleaning, soaking, bathing	Solids, sulfite, chromium, lime, sodium chloride
Laundry	Washing fabrics	Turbid, alkaline, organic solids
Chemical		
Acids	Wash waters, spillage	Low pH
Detergents	Purifying surfactants	Surfactants
Starch	Evaporation, washing,	Starch
Explosives	Purifying and washing TNT, cartridges	TNT, organic acids, alcohol, acid, oil soaps
Insecticides	Washing, purification	Organics, benzene, acid highly toxic
Phosphate	Washing, condenser wastes	Suspended solids, phosphorus, silica, fluoride, clays, oils, low pH
Formaldehyde	Residues from synthetic resin production and dyeing synthetic fibers	Formaldehyde
Materials		
Pulp and paper	Refining, washing, screening of pulp	High solids, extremes of pH
Photographic products	Spent developer and fixer	Organic and inorganic reducing agents alkaline
Steel	Coking, washing blast furnace, flue gases	Acid, cyanogen, phenol, coke, oil
Metal plating	Cleaning and plating	Metals, acid
Iron foundry	Various discharges	Sand, clay, coal
Oil	Drilling, refining	Sodium chloride, sulfur, phenol, oil
Rubber	Washing, extracting impurities	Suspended solids, chloride, odor, variable pH
Glass	Polishing, cleaning	Suspended solids

emphasized that pollutant loading will vary extensively within a given industry, and consequently, industrial wastewater characterization must be done on a plant-by-plant basis. In order to accomplish this, a wastewater characterization study must be performed for each particular industrial facility under consideration. Accordingly, this section on industrial wastewater characterization will be devoted to an explanation of how to conduct such a plant wastewater survey.

Plant Wastewater Survey. In addition to characterizing plant wastewaters, a properly conducted survey of plant waste streams is a vital first step in developing an efficient plant wastewater management program. Data obtained from a comprehensive survey serve as the starting point for effective water pollution control plans. As such, the plant survey is necessary in order to

- Develop a design basis for wastewater control and treatment requirements
- Achieve compliance with applicable federal, state, and local regulations and standards
- Conserve resources through reduction of water usage and pollutant loading wherever possible
- Segregate incompatible wastewater streams and sewer systems

There are two basic elements in any plant wastewater survey: (1) definition of the physical characteristics of the plant's sewer systems, and (2) development of individual wastewater stream profiles. Generally, completion of many of the tasks necessary to define the physical systems is required before information on the flow characteristics and composition of individual waste streams can be obtained. Consequently, the two major components of the survey are usually handled consecutively rather than simultaneously, although some overlap is inevitable. A graphic illustration of the various components in a plant wastewater survey is provided in Figure 6.14 (40).

Physical Characteristics. The most common starting point of a plant wastewater survey is the drawing file, to determine the extent of available information. All layout and schematic drawings that relate to the plant's sewer systems and water supply should be compiled and carefully reviewed. If overall sewer maps and schematic flow diagrams are not available, they must be prepared. If these drawings do exist, they must be updated. The end product should be a complete set of up-to-date sewer maps and a comprehensive flow diagram of the plant's wastewater systems.

In gathering data, attention should be given to available information on the sanitary sewage system and stormwater collection and disposal facilities, as well as the process drainage system. In many plants, concern with the sanitary sewage system will focus on segregating sanitary wastes from other plant wastes and ensuring that there are no cross-connections between the separate systems. Generally, this phase of the survey should confirm that clean stormwater runoff is truly uncontaminated and is segregated prior to discharge and that all contaminated runoff is directed to process sewers. Conditions will obviously vary from plant to plant, but in most instances, the greatest amount of time and effort will be expended in gathering data to fully define the process sewer system.

In reviewing drawings and other records, the plant's water supply system should be analyzed to delineate all processes using water. Each operation should be studied to determine how water is used and in turn discharged. Some processes may actually produce water by synthesis as a by-product of a chemical reaction, while others consume water in various ways. This type of review will establish the actual mechanism of wastewater production within the plant, which will serve as a basis for the additional investigative work which must be performed.

The next step in the survey involves field verification of data gathered thus far, and exploration of areas of uncertainty. This phase of the study is extremely important and requires a dedication to detail, as well as a determination to uncover all necessary information. The overall objective of this portion of the fieldwork is to visually confirm information shown on the drawings and to fill any data gaps by firsthand observation. Specific objectives include

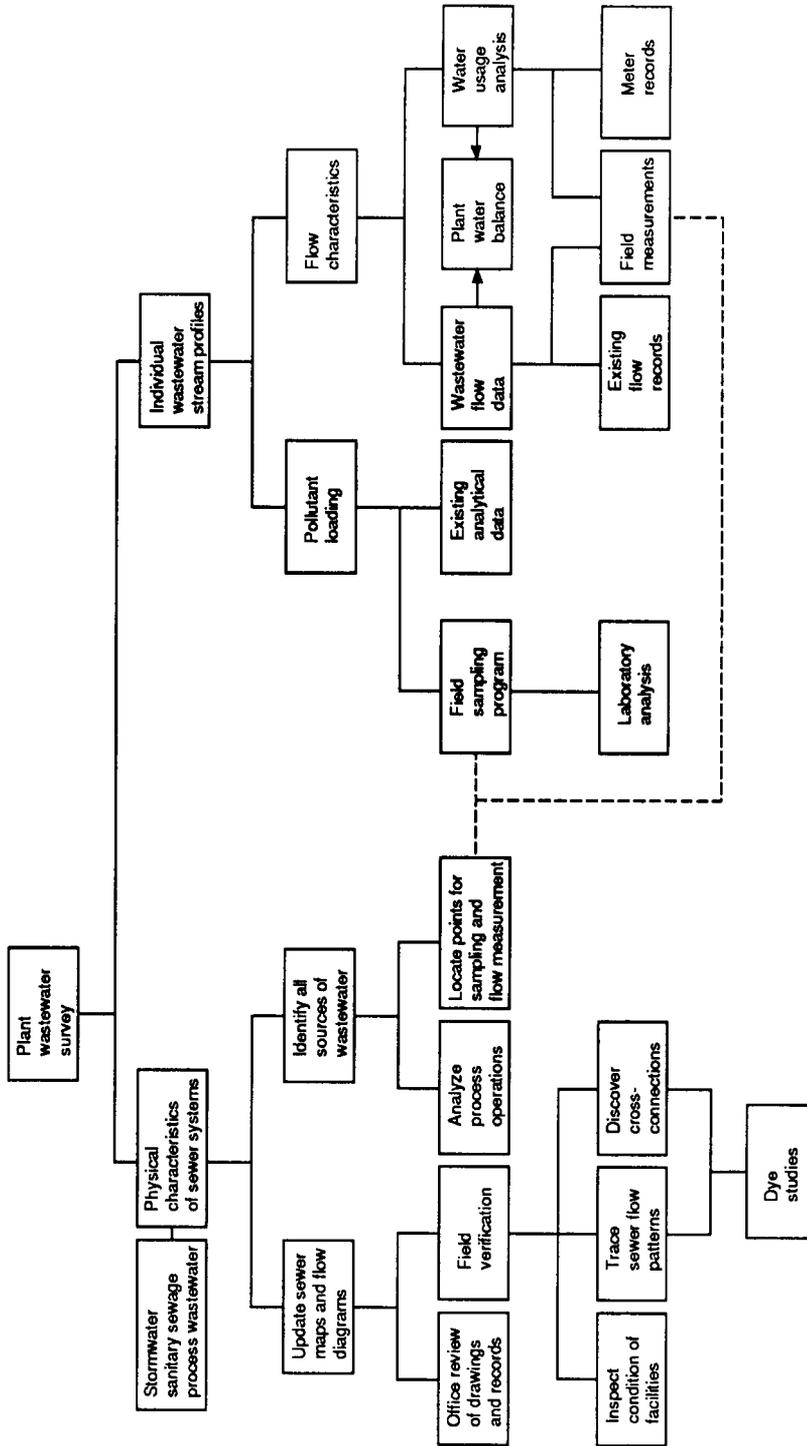


FIGURE 6.14 Components of a wastewater survey (40).

- Complete definition of physical characteristics to permit updating of sewer maps and flow diagrams
- Identification of all sources of waste within the plant
- Location of sampling points for individual and composite waste streams
- Discovery of cross-connections between storm, process, and/or sanitary sewer systems
- Familiarization with existing collection and disposal systems for subsequent work

In performing the fieldwork, all wastewater streams should be traced from one end to the other, or in other words, from the waste source to the point of ultimate disposition. This involves inspection of all man-holes, not only to verify location and condition, but primarily to confirm or determine flow direction in the sewer. Frequently, dye studies are necessary to establish beyond question the pattern of sewer flow within each system. Several chemicals may be used for tracing of sewer lines with dye, although sodium fluorescein and rhodamine are probably the most common. Sodium fluorescein is a hygroscopic orange-red powder that is freely soluble in water, perceptible at a concentration of 0.02 mg/L, and has no toxic effect on fish (38). These properties, combined with its brilliant yellowish-green fluorescence, make tracing the flow in sewers a relatively simple task. Rhodamine has a reddish color which is easily distinguishable and is sometimes preferred for this property in specific situations.

The field investigation should include interviews with key operating and maintenance personnel throughout the plant. Frequently, their knowledge of details and field changes not shown on the drawings can be invaluable in establishing sewer location and flow patterns. In old plants, long-term employees often are the best source of information concerning sewer lines. All data obtained in this manner should be verified by visual inspection and dye studies, if necessary.

Completion of the first phase of the survey will fully establish the physical characteristics of the plant's sewer systems and set the stage for the development of individual waste stream profiles, which constitute the plant wastewater characterization. It will also produce the data required to update sewer maps and flow diagrams or prepare new drawings in the drafting room, at the same time that the remainder of the survey proceeds. It is generally advantageous to make all drawing revisions promptly after completion of the field investigations. In this way, the information is still current, and questions may be resolved in the field conveniently as the ensuing portion of the survey is being completed (40).

Waste Stream Profiles—Composition. The establishment of individual waste stream profiles, which characterize the composition and flow of all wastewater sources, is the objective of any plant wastewater survey. Consequently, considerable care must be exercised in this phase of the survey to ensure that all data obtained are accurate and representative of actual plant conditions. In developing representative composition characteristics and flow values for individual waste streams, previous records or sampling results should first be examined. If such data are available in sufficient detail, a field monitoring program may only be necessary to update and verify the existing information. However, if data are unavailable or insufficient, a full program should be integrated with flow measurements, as it is most convenient and cost effective to perform these two functions in the field at the same time.

Careful consideration must be given to the location, type, frequency, and duration of sampling to be done in the field. A working knowledge of each process is necessary to ensure that all samples obtained are representative of actual conditions, taking into account such factors as the effect of changes in production on wastewater characteristics, diurnal variations, and seasonal effects.

An important consideration is to establish which pollutant parameters to analyze for. One approach in complex operations is to monitor the plant effluent for several days and analyze these composite samples for the full spectrum of pollutant parameters. This procedure will confirm the presence, or document the absence, of each pollutant. Those present in the plant effluent can then be checked in the individual waste stream analyses.

Field sampling can be accomplished either manually or through the use of automatic devices. Three distinct types of sampling procedures can be utilized:

- Grab sampling, in which a single volume of wastewater is obtained at a given point in time and then analyzed. This type of sample will not always provide an accurate measure of wastewater characteristics, especially when the flow is heterogeneous, or varies with time.
- Simple composite sampling, in which a timed sequential collection of equal-volume grab samples are combined in a single reservoir. This procedure can give a partial evaluation of the variability of wastewater composition with time.
- Flow-proportioned composite sampling, in which incremental samples are collected, with volumes proportional to flow. This type of sample provides the most accurate measure of wastewater quality and pollutant loading.

Automatic devices that can obtain all three sampling types are commercially available. These automatic samplers vary over a wide range of cost, applicability, and reliability and can be extremely helpful in conducting the wastewater survey. Automatic samplers are widely available on either a rental or lease-purchase basis, if a permanent installation is contemplated in the future. Special attention should be paid to the accuracy of sampling activities, since sampling errors can range up to 200% of the true value. The basic problem results from the fact that the typical industrial waste may have a large proportion of its pollutants in the form of suspended solids. Consequently, the quantity of suspended solids entrained during sampling should be proportional to the suspended solids content of the wastewater stream. Common practice is to simply place a suction tube in the wastewater flow or to immerse an open sampling bottle in the stream. Since solids entrainment is a velocity-controlled process, an attempt should be made to obtain samples isokinetically. Accomplishing this type of sampling is a difficult procedure, but it can be done with automatic samplers by aligning the sample tube so that it is facing upstream and is secured rigidly in place.

For manual sampling in inaccessible locations, what is frequently needed is an extendable pole, which can be easily fabricated, with a stoppered bottle attached to the end. The sample bottle should be hinged so that it can be tilted to align parallel to the wastewater flow to obtain an accurate sample. In addition, this orientation allows for sampling from very shallow streams. The bottle stopper should be attached to a string so that it can be removed while the sample bottle is submerged. Because of the potential for large errors associated with sampling, it is essential that extreme care be exercised in selecting sampling devices and procedures.

The volume of sample that must be collected for a particular analysis, the preservative that must be added to the sample, and the allowable holding time are all important considerations in the overall field sampling procedure. In this regard, it is imperative that all sample collection, preservation, and analysis be done in accordance with the precise and recognized techniques established for the sampling and analysis of wastewaters. A summary of recommended sample containers, preservatives, and maximum holding times is provided in Table 6.17 (page 6.34). Similar information for soil/sediment and waste samples is provided in Chapter 9 (45, 95).

The analytical aspects of the sampling program also involve a number of detailed considerations. Many plants have their own laboratory facilities or have direct access to a corporate analytical laboratory. In these cases, a decision must be made whether to utilize the company laboratory or contract the work to an independent laboratory. Factors to consider include the capability of the company laboratory in terms of personnel and equipment, and the proximity, capability and cost of an independent laboratory. Another important consideration is the question of objectivity, in that it may be preferable to use an independent laboratory for all data that will be reported to a regulatory agency.

If the option of an independent laboratory is chosen, the actual selection of a firm is extremely important. Quality control within the laboratory must be exercised to ensure the accuracy and reproducibility of the results. A technique for checking the accuracy of laboratory results is to submit preanalyzed samples for analysis by the lab. Such "spiked samples" are made available to interested parties by the Regional Analytical Quality Control Coordinator in each of the ten EPA regional offices.

Analytical results must be carefully reviewed and evaluated to ensure that the data are accurate and truly

representative. Variability in results can usually be attributed either to abnormal circumstances in the plant (temporary changes in a process, upset conditions, poor housekeeping, spills, etc.) or to errors in the sampling, analysis, or calculation of numerical values.

Questionable results should be brought to the attention of the laboratory for reexamination and confirmation. If necessary, additional samples should be taken at this time in order to check previous values. When this has been accomplished and there is reasonable certainty that the data are accurate, they should be summarized in a clear and systematic manner (40).

Waste Stream Profiles—Flow Characteristics. The accurate determination of individual waste stream flows involves a broad overall evaluation of water usage within the plant. This can best be accomplished through a water budget analysis or water balance, which accounts for all individual water demands and compares water usage to the field measurement of individual wastewater streams.

Generally, a good starting point for a water balance evaluation is a review of plant records on water usage and consumption. If water is obtained from a municipality or private water company, then the monthly or quarterly water bills will provide a basis for establishing total plant consumption. If water is obtained directly from wells or a surface source, generally it is metered as it enters the plant. These records should be carefully examined for a period of several years, not only to establish an average daily usage, but to determine trends, seasonal variations, or other significant factors.

Many industrial facilities meter the water supplied to major process units or other significant users within the plant. Such records can be extremely helpful in determining the pattern of water distribution within the facility. Water supply system drawings should be examined and field-verified to obtain an overview of water usage. Most often, field measurements and estimates will be necessary to determine individual flow values.

Many methods are available to estimate flows where records and flow instruments are unobtainable. Frequently, equipment ratings, tank gauges, process analyses, and visual observations can be used to estimate specific flows. Attention should be paid to periods of operation so that total consumption values can be determined for intermittent water users. Care must be exercised to ensure that values established for each water usage are representative of conditions that usually prevail within the plant.

In most instances, field measurement of incoming flow is desirable to either determine a specific usage value or confirm a previous estimate. Since water is supplied in pressure conduits, conventional water meters are the most applicable for measuring freshwater flow. In larger lines, orifices, nozzles, and venturi meters can be utilized, and nozzles and orifices can also serve to compute the flow from freely discharging pipes. Frequently, water supply lines will have to be broken to install a measuring device. Although this procedure may be troublesome and time consuming, it nevertheless is invaluable in obtaining accurate water usage data.

When this phase of the water balance has been completed, a summary such as the one shown in Table 6.18 (page 6.36) should be prepared. The summary must be concise, but still provide sufficient detail to fully document each entry. The total plant water usage obtained in this manner should be in reasonably good agreement (preferably less than 10% difference) with the consumption value for the entire plant obtained from meter records or water bills. If a significant disparity results from this comparison, further fieldwork is in order to refine the individual usage figures.

The next task in the water budget analysis is the determination of flow values for all individual waste streams entering the plant's sewer systems. This includes those flows normally directed to the sanitary and stormwater sewers, as well as the process sewer system. Flow data on individual waste streams will usually not be available, so that field measurements and estimates will be necessary. In some plants, particularly those discharging to publicly owned treatment works, the entire plant effluent may be monitored for flow prior to discharge. In those cases, the records should be reviewed to determine representative average and peak values that can serve to verify the cumulative total obtained from the individual waste stream flow determinations.

As with establishing individual water supply flows, the task of determining wastewater flow requires at-

TABLE 6.17 Recommended Containers, Preservation, and Holding Times for Water/Wastewater Samples (95)

Analytical group	Water/Wastewater ¹		
	Container	Preservation	Holding time (days)
Biological			
Bacteriological ²	B	I	6 hr
Toxicity, Acute	CU	I	2
Toxicity, Chronic	CU	I	2
Inorganics			
Residual Chlorine ²	SM	NA	I
Turbidity	SM	I	2
Conductivity	SM	I	28 ¹⁴
Temperature ²	SM	NA	I
BOD ₅	HP ⁵	I	2
Solids Series	HP	I	7
Settleable Solids	HP	I	2
Nutrients (N, P)	HP	S/I	28
Chloride	LP	NA	28
Ortho-P	LP	I ⁷	2
Dissolved P	LP	S ⁷ /I	28
COD	LP	S/I	28
Alkalinity	LP	S/I	28
Color	GP	I	2
Oil and Grease ²	LG	S/I	28
Metals	LP	N	180
Mercury	LP	N	28
Metals—TCLP	LP	I	360 ¹⁵
Metals—EP	LP	I	360 ¹⁵
Chromium VI	LP	I	1
Cyanide	LP	A ⁸ /C ⁹ /I	14
Sulfides	LP	Z/C ¹⁰ /I	7
Sulfates	LP	I	28
Nitrite	LP	I	2
Nitrate	HP	I	2
Hardness	LP	N	180
Fluoride	LP	NA	28
Organics			
VOCs ²	V	B ¹¹ /I	14/7 ¹⁶
VOCs-TCLP ²	V	I	28 ¹⁷
Extractables ³	GG	I ¹²	47 ¹⁸
Extractables—TCLP	GG	I	61 ¹⁹
Dioxins ⁴	LA ⁶	I ¹³	75 ²⁰
Percent alcohol	GG	I	NS
Phenols	LA	S/I	28
Organic halide (TOX)	LA	S/I	28

¹Consult 40 CFR, Part 136, Table II—Required Containers, Preservation Techniques, and Holding Times for latest requirements.

²Grab sample only; unless indicated a grab or composite is acceptable.

³Including pesticides, herbicides, and PCBs.

⁴Consult local laboratory for most recent dioxin container and preservation requirements.

⁵Use GP for BOD with multiple parameters

⁶Collect 2 sample containers (LA) per sample plus 4 at one location for matrix spike

⁷Filter on-site

⁸Only with residual CL_2

⁹To pH>12.0 S.U.

¹⁰To pH>9.0 S.U.

¹¹With residual CL_2 mix sample in 8 oz glass container with 8 drops 25% ascorbic acid

¹²With residual CL_2 mix sample with 0.008% sodium thiosulfate

¹³With residual CL_2 mix sample with 80 mg of sodium thiosulfate per liter

¹⁴Determine on-site if possible

¹⁵360 days: 180 days to extraction plus 180 days to analysis

¹⁶7 days if not preserved

¹⁷28 days: 14 days to TCLP extraction plus 14 days to analysis (7 days if not preserved following extraction)

¹⁸47 days: 7 days to extraction, 40 days to analysis

¹⁹61 days: 14 days to TCLP extraction, 7 days to solvent extraction, 40 days to analysis

²⁰Method 8290 specifies 30 days to extraction plus 45 days to analysis

Containers:

B—Bacteriological container

CU—Cubitainer: 1 gallon or 2 gallon

LP—1 liter polyethylene

GO—1 gallon amber glass (Teflon lid)

V—40 ml glass (Teflon septum lid)

SM—Stormore 500 ml polyethylene

LG—1 liter widemouth glass (Teflon lid)

GP—1 gallon polyethylene

HP—Half-gallon polyethylene

LA—1 liter amber glass (Teflon lid)

Preservatives:

A—Ascorbic acid

B—Sodium bisulfite

C—NaOH

H—HCl

I—Ice (4°C)

N—50% HNO_3 (pH<2.0 S.U.)

NA—Not applicable

S—50% H_2SO_4 (pH<2.0 S.U.)

Z—Zinc acetate

Holding Times: in days unless noted otherwise:

NS—Not specified

I—Immediate (within 15 minutes: 40 CFR 136 Table II)

C—NaOH

H—HCl

I—Ice (4°C)

N—50% HNO_3 (pH < 2.0 S.U.)

NA—Not applicable

S—50% H_2SO_4 (pH < 2.0S.U.)

Z—Zinc acetate

Shipping Note: When samples are to be shipped by common carrier or sent through the United States mail, it must comply with the Department of Transportation Hazardous Materials Regulations (49 CFR 172). The person offering such material for transportation is responsible for ensuring such compliance. For the preservation requirements of 40 CFR, Part 136, Table II, the Office of Hazardous Materials, Materials Transportation Bureau, Department of Transportation has determined that the Hazardous Materials Regulations do not apply to the following materials: hydrochloric acid (HCl) in water solutions at concentrations of 0.04% by weight or less (pH about 1.96 or greater); nitric acid (HNO_3) in water solutions at concentrations of 0.15% by weight or less (pH about 1.62 or greater); sulfuric acid (H_2SO_4) in water solutions at concentration of 0.35% by weight or less (pH about 1.15 or greater); and sodium hydroxide (NaOH) in water solutions at concentrations of 0.08% by weight or less (pH about 12.30 or less). This footnote is wholly reproduced from 40 CFR 136.3, which is definitive.

TABLE 6.18 Plant Water Usage (40)

Water use	Average consumption, gpd	Comments
1. Domestic (sanitary sewage)	8,600	Estimated 20 gal/capita/day per accepted plumbing practices.
2. Utilities		
a. Cooling tower makeup	28,400	Average of water meter records for past two years.
b. Steam boiler makeup	18,000	Average of direct measurements made over a two-week period.
c. Water treatment	800	Estimated from equipment manufacturer's data for backwash; 80 gpm for 10 minutes once per day.
3. Unit A		
a. Water used in product	60,000	Average of production records for past two years.
b. Laboratory	4,800	Ten sinks running continuously during an 8-h day; estimated at 1 gpm by bucket and stopwatch.
c. Pump and compressor cooling	20,200	Direct single measurement—14 gpm-24 h/day.
d. Floor wash	600	Estimated from hose nozzle size, water pressure, and time of operation.
e. Reactor wash	500	Reactors 1, 2, 3; five times/year 4, 5—twice yearly. Estimated average daily flow.
4. Unit B		
a. Vacuum seal water	7,200	Rated at 10 gpm while operating; average operation—12 h/day.
b. Compressor cooling	9,800	Two compressors: (1) Unit 1 rated at 1 gal/100 ft ³ . (2) Unit 2 rated at 3 gpm when operating.
c. Venturi scrubbers	16,800	Single measurement of 12.5 gpm. Plant operators estimated an average of 13 to 15 gpm. Assumed 14 gpm for 20 h/day.
d. Floor wash	500	Same as unit A.
e. Vessel rinse water	1,200	Internal spray rinse—1000 gpd; Hand rinse—200 gpd; per operating personnel.
5. Pilot plant	4,000	Pump cooling measured at 15 gpm. Seal water measured at 7 gpm.
Total	181,400	

tention to detail and ingenuity. Prior to initiating fieldwork, a program based on the earlier fieldwork should be developed that delineates the location, frequency, and type of flow-measuring device or estimating method to be utilized for each determination. The format for the program should be established so that entries can be made when data are obtained, and a summary of wastewater flow information such as Table 6.19 is automatically produced.

Actual wastewater flow determinations in the field involve consideration of a number of factors concerning the process itself. Variability of flow caused by process conditions is extremely important. Consequently, details concerning batch versus continuous operation, seasonal or diurnal variations, and duration and nature of the process require judgments regarding the type, frequency, and extent of the monitoring program. In order to avoid extensive piping modifications for the installation of monitoring equipment, innovative techniques should be utilized wherever possible. A number of references and technical articles are available that deal with this subject in great detail. There are several commercially available automatic flow-measuring de-

TABLE 6.19 Wastewater Flows (40)

Wastewater source	Process sewer system		Sanitary sewer system		Stormwater sewer system		Discharge frequency	Comments
	Average flow, gpd	Peak flow, gpd	Average flow, gpd	Peak flow, gpd	Average flow, gpd	Peak flow, gpd		
1. Domestic (sanitary sewage)			8,200	12,400			8 h/day	Average and peak values of flow monitoring records for past two years.
2. Utilities								
a. Cooling tower blowdown	5,400	8,200					Continuous	Average and peak values of direct readings by temporary flume over a two week period.
b. Boiler blowdown	2,000	—					Continuous	Estimated by open pipe method.
c. Water treatment backwash	900	—					10 min once/day	Single measurement by bucket and stopwatch.
3. Unit A								
a. Laboratory	4,800	—					Continuous for 8 h	Same measurement as plant water usage (Table 16.18).
b. Pump and compressor cooling	20,200	—					Continuous for 8 h	Same measurement as plant water usage (Table 16.18).
c. Floor wash	600	—					Once/day	Daily 10-min hose down.
d. Reactor wash	500	—					19 washes/yr	Estimated average daily flow.
e. Roof and area drainage							intermittent	Volume, frequency, and duration dependent upon weather conditions.
f. Steam condensate	3,000	—	6,500	—			Continuous	Condensate is only partially returned to the boiler, with the remaining volume being discharged at 18 individual points. Each flow was estimated by bucket and stopwatch.
g. Reactor upset and spillage								Emergency condition of a random nature.

(continues)

TABLE 6.19 Wastewater Flows (40) (continued)

Wastewater source	Process sewer system		Sanitary sewer system		Stormwater sewer system		Discharge frequency	Comments
	Average flow, gpd	Peak flow, gpd	Average flow, gpd	Peak flow, gpd	Average flow, gpd	Peak flow, gpd		
4. Unit B								
a. Vacuum seal water	7,200	—	—	—	—	—	3 times/day	Same estimate as plant water usage (Table 6.18)
b. Compressor cooling	11,600	—	—	—	—	—	Continuous	Determined by measuring depth of flow in the sewer.
c. Venturi scrubbers	16,800	—	—	—	—	—	20 h/day	Same estimate as plant water usage (Table 6.18).
d. Floor wash	500	—	—	—	—	—	Once/day	Daily 10-min hose down.
e. Vessel rinse water	1,200	—	—	—	—	—	3 tanks/day	Same estimate as plant water usage (Table 6.18).
f. Roof and area drainage	—	—	—	—	intermittent	—	—	Volume, frequency, and duration dependent upon weather conditions.
g. Steam condensate	4,200	—	—	—	3,700	—	Continuous	Condensate is only partially returned to the boiler, with the remaining volume being discharged at 14 individual points. Each flow was estimated by bucket and stopwatch.
5. Pilot plant	4,200	—	—	—	—	—	Intermittent	Determined from observation of sump pump operation; pump rated at 30 gpm and operates for an average of 6 min per hour.
System totals	83,100	—	—	—	8,200	—	—	—
Total to sewer	—	—	—	—	101,500	—	—	—

vices that can provide reasonably accurate flow measurement data and which would be useful in a plant wastewater survey.

Methods that are available for use in determining wastewater flows are summarized in Table 6.20. Generally, the choice of technique will depend primarily upon accessibility, equipment, and piping configuration and the hydraulic characteristics (i.e., flow from freely discharging pipes versus flow in open channels, etc.). For relatively small flows from the open end of a pipe, the bucket and stop-watch method can be a convenient procedure. It relies upon the use of a container of known volume and notation of the time required to fill it. Pipes that are flowing full when freely discharging may be monitored by the use of orifices and nozzles in a manner similar to pressure conduits. The flow from pipes that are partly full when discharging can be estimated by either the open pipe or California pipe method, or measured directly by use of an open flow nozzle.

The most common devices used for measuring the flow in sewers or open channels are weirs or flumes, which create an obstruction to the free flow of liquid. A direct measurement of water level at a specified distance upstream of the weir or flume in turn establishes the flow, which is proportional to the depth of liquid. Use of these devices in a plant survey may require temporary installations, where drainage lines are cut and directed to a measuring flume or weir especially rigged for the purpose. Flow-recording instruments are readily connected to these devices and can be extremely helpful in establishing flow values and patterns. Other techniques for flow determination in sewers include the use of current meters and pitot tubes, the measurement of liquid depth and velocity, and the chemical dilution method. Related observations, such as measuring level changes in tanks, pumping rates, and duration can frequently be employed to estimate the flow in sewer lines.

In addition to determining the flow for individual waste streams, the total plant effluent should be monitored if this is not routinely done. The average value established for the plant effluent should be compared to the total obtained from summarizing the individual wastewater flows in Table 6.19. As with the water supply figures, agreement within 10% should be considered acceptable. If a larger disparity results, judgment is necessary to select the areas requiring reexamination in the field.

When all field measurements and estimates have been completed, the last step in the water budget analysis is the comparison of water supply and wastewater flow values as illustrated in Table 6.21. In this procedure, allowances must be made for all water consumed or lost in the plant in order to effect a true balance. Typical usage within a plant resulting in consumption would include water used in the product, evaporation and windage losses from a cooling tower, makeup to a steam boiler, lawn sprinkling, and leaks in the water supply system. In older facilities, infiltration into sewer lines should be taken into account in reconciling sewer flows with water supply figures. The final water balance should provide an accurate summary and accounting of all water used in the plant, which in turn should serve as a key element in fulfilling the goals of the plant wastewater survey.

The last step in the development of individual waste stream profiles is the application of statistical analysis to the data. Statistical correlation will permit a proper choice of average values and will also provide a correct method for determining the range of values for a specific parameter. Generally, average values and variability as expressed by the standard deviation will be of most interest in developing a wastewater management plan. This information should be sufficient to provide the basis for judgments regarding the control of individual wastewater streams, process changes, recycling, segregation of wastewaters, and the evaluation of alternative programs. However, consideration of peak values and probability determinations such as the 50% and 90% values will undoubtedly be necessary for the design of wastewater treatment facilities. One of the many available books or technical articles on statistics should be consulted for this type of data analysis (40).

Utilizing Survey Results. In broad terms, the survey results should lead to a plan for wastewater management within the plant, and the development of a continuing program for monitoring and control of plant wastewaters, as shown in Figure 6.15 (page 6.42). As indicated at the outset, an in-depth plant survey frequently will bring to light information not previously realized regarding water usage and loss of product or

TABLE 6.20 Comparison of Flow Measurement Techniques (12)

Method	Application*	Comments
Venturi meter	A	Useful where high pressure recovery is necessary
Orifice meter	A,B	Relatively inexpensive; easy to install; large head loss
Flow nozzle	A,B	Greater head loss and less cost than venturi meter; less head loss and more cost than orifice
Magnetic flow meter	A	Good for fluids containing suspended solids; no greater head loss than for equivalent length of pipe
Pitot tube	A,D	Economical for large pipes; can be used to measure velocity flow in open channel
Rotameter	A	Simple, inexpensive device for measuring relatively low flow rates of clear, clean liquids
Open-end pipe method	B,C	Approximate, inexpensive procedure for estimating flow from open pipes by measuring two dimensions of stream freely discharging into air
California pipe method	C	Requires horizontal freely discharging pipe of known diameter and measurement of distance from top of pipe to water surface
Open flow nozzle	C	Two common types are Kennison nozzle and parabolic flume; useful for accurate and continuous measurement
Weirs	D	Commonly used; easy to install; inexpensive; V-notch, Cipolletti or rectangular design; sediment can accumulate behind a weir and affect accuracy
Flumes	D	More accurate than weir; self-cleaning; Parshall or Palmer–Bowlus design; minimum upstream effects; subject to flooding
Depth of flow measurement	D	Utilization of Manning formula to estimate flow by measuring depth, when slope and coefficient of roughness are known
Continuity equation	D	Involves determination of velocity by a current meter or timing a float or tracer dye between two points, and measuring depth of flow to determine wet cross-sectional area
Bucket and stop-watch	B,C	Estimating procedure using container of known volume and noting time required to fill; good for small flows where physically convenient
Pumping rates	A,B,C	Flow determination by recording the time of pumping and pump capacity at the discharge pressure using manufacturer's pump curves
Tank level change	A,B,C	Useful for batch operations to determine waste flow by monitoring change in level with time in tank of known volume
Dilution method	A,D	Addition of dye, salt, or radium to liquid stream with corresponding measurement of concentration to determine flow; monitoring apparatus is expensive

*A = pressure pipe; B = freely discharging pipe flowing full; C = freely discharging pipe flowing partially full; D = open channel flow.

TABLE 6.21 Typical Plant Water Balance (40, 41)

Water usage	Average volume supplied, gal/day	Average wastewater flow, gal/day			Consumptive use, gal/day	Comments
		Process sewer	Sanitary sewer	Storm sewer		
1. Domestic	8600		8200			
2. Utilities						
a. Cooling tower	28,400	5400			24,700	Calculated evaporation and windage loss based upon circulating rate and location.
b. Steam boiler	18,000	2000				
Unreturned condensate		7200		10,200		
c. Water treatment	800	900				
d. Miscellaneous leaks in water supply system					5000	Based on estimates in the literature for systems in this age category.
3. Unit A						
a. Water used in product	60,000					
b. Laboratory	4800					
c. Pump and compressor cooling	20,200	20,200				
d. Floor wash	600	600				
e. Reactor wash	500	500				
4. Unit B						
a. Vacuum seal water	7200	7200				
b. Compressor cooling	9800	11600				
c. Venturi scrubbers	16,800	16,800				
d. Floor wash	500	500				
e. Vessel rinse water	1200	1200				
5. Pilot plant	4000	4200				
System totals	181,400	83,100	8200	10,200	89,700	
Allowance for infiltration	—	4000	400	—	—	
Adjusted system totals	181,400	79,100	7800	10,200	89,700	
Water balance	181400		186,800			
Discrepancy		5400	Acceptable (within 3%)			

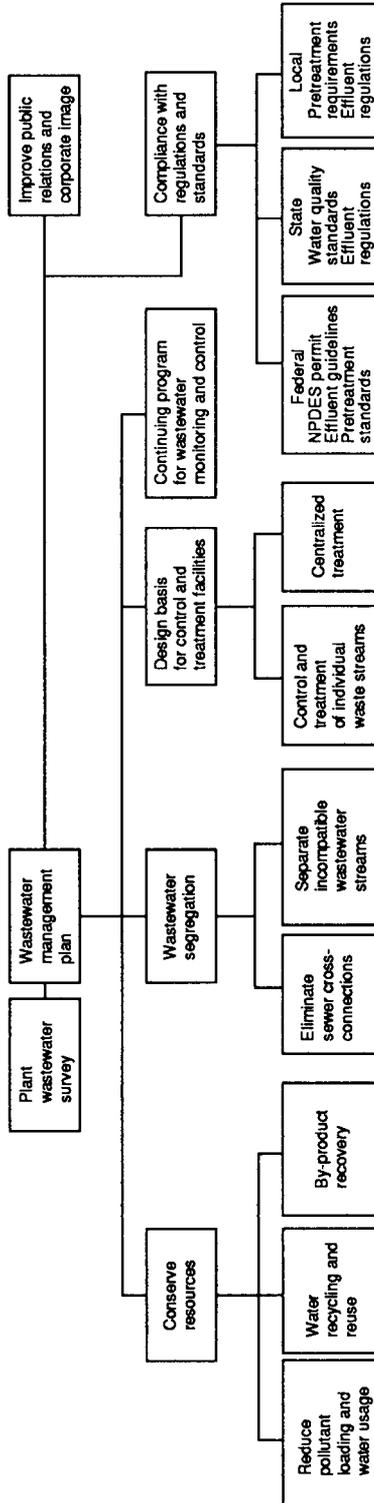


FIGURE 6.15 Wastewater survey utilization (41).

raw materials. These data can in turn be used for conservation purposes, which can result in considerable cost savings. The initial emphasis should be placed on a thorough appraisal of each waste stream to investigate the various possibilities available to reduce water usage and pollutant loading. Often, process changes can be instituted with a minimum of disruption, which will significantly reduce the quantity of water used or the amount of pollutants discharged into the waste stream. In many instances, water recycling schemes will become evident once the waste streams with the major pollutant loadings are isolated. Finally, the elimination of cross-connections between the sewer systems, resulting in wastewater segregation, can frequently effect sizable cost savings.

If wastewater treatment is necessary, the results of the survey will provide the data required to proceed in a systematic manner. The feasibility and advantages of treating individual waste streams, or groups of waste streams with compatible pollutants, can be explored and evaluated before considering centralized treatment. In many cases, this type of approach will be advantageous in that treatment would be performed prior to dilution in the sewer by other waste streams, and considerably smaller volumes would be involved in the treatment process. In any event, the data gathered during the plant survey will provide the basis for the evaluation of alternative treatment programs. Comparison of wastewater data with applicable effluent regulations should serve to provide a basis for a preliminary conceptual design of facilities. In most cases, a detailed design for treatment of industrial wastewater should not be undertaken without a pilot program to firmly establish all design parameters.

Once the wastewater situation within the plant has been fully defined and a wastewater management plan has been formulated, required information can be furnished to regulatory agencies and a cost estimate for implementation of the plan can be developed.

The final aspect of utilizing the results of the wastewater survey is the establishment of a continuing program for wastewater monitoring and control. The extensive effort expended in completing an in-depth survey should not be regarded as a one-time endeavor. Rather, it should be viewed as the beginning of a continuing plan to control wastewaters within the plant. Such a program requires emphasis from management to provide the necessary funds and to create an awareness and orientation in all employees toward environmental considerations. This can be accomplished by stressing the fact that waste monitoring and waste treatment systems are in reality an integral part of plant operations. Production and operating personnel should be encouraged to consider the variability of their waste load as a factor in evaluating the overall performance of their unit.

In the final analysis, wastewater control and pollution abatement depends not only upon the operation of treatment facilities and the performance of monitoring equipment, but equally upon the awareness, attitude, and concern of all plant personnel (40).

COLLECTION

Collector sewers collect wastewater from service lines and carry it by gravity to interceptor sewers. If topography dictates, the collector sewer may discharge to a pump station, which transports the wastewater via a force main to another collector sewer or interceptor sewer at a higher elevation. Finally, the collection system delivers the wastewater to the treatment plant site.

Groundwater infiltration and stormwater inflow utilize collection system capacity. During wet weather, system flows may increase significantly and result in pump station or treatment works overloads or may cause a sanitary sewer overflow. Infiltration/inflow studies are undertaken to identify problem areas and to define rehabilitation actions.

Typically, collector sewers are 8 to 12 in (20 to 30 cm) in diameter, follow streets, and are designed for peak flows of three to four times expected average daily flow. The interceptor sewers are generally larger, follow valleys and stream beds, and are designed with a flow peaking factor of 2.5 to 3. Pump stations and

force mains are designed for peak flow conditions. A representative layout of portions of a sanitary sewer system is shown in Figure 6.16.

Hydraulic Design

Following the location and topographic survey, the sewer line is plotted in plan, and a ground elevation along the line is plotted in profile. Additional elevation data are needed to define design restraints, such as bottom of stream beds, road and railroad structures, and sewer and service line elevations.

Gravity Sewer. Once the field data are available and design flows are established, manual or computer-assisted computations may begin to size 8 in (20 cm) minimum and vertically locate a gravity sewer in profile. Table 6.22 illustrates an approach to sewer line design computations. If the design flow computations are known to involve several predictable components, a separate computation may be desirable (see Table 6.23).

Slope. The slope of the gravity sewer should be sufficient to provide a minimum velocity of 1 ft/s (0.3 m/s) during average to low-flow conditions. Typical design practice is to provide a minimum velocity of 2 ft/s (0.6 m/s) when the pipe is flowing full. Maximum velocities should not exceed 10 ft/s (3 m/s) due to the development of hydraulic problems in the sewer.

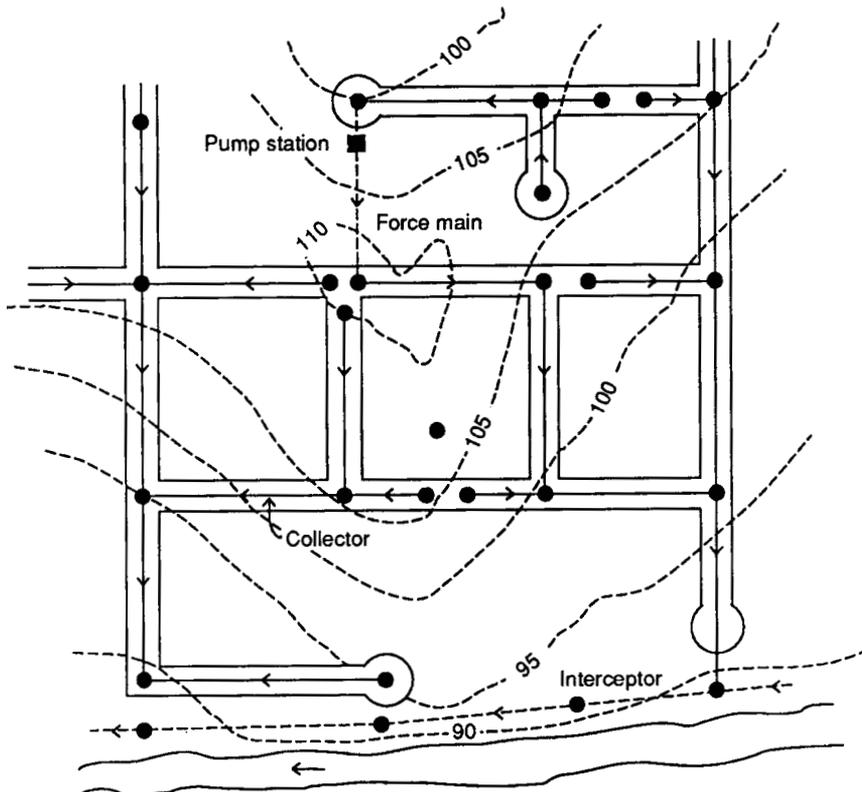


FIGURE 6.16 Sanitary sewer system layout.

TABLE 6.22 Sewer Design Computation Sheet

Reach	Manhole		Length	Design flow			Slope	Diameter	Full Capacity	Velocity			Downstream elevations		Upstream elevations		Depth of cut	
	From	To		Min	Avg	Max				Min	Avg	Max	Invert	Ground	Invert	Ground	Downstream	Upstream

Minimum slopes for 8 to 36 in (20 to 91 cm) sewers, when flowing full at 2 ft/s (0.6 m/s), are shown in Table 6.24. These values were computed using an n value of 0.013. When the slope is between sizes, the larger sewer size should be selected.

Flow. Kutter's formula has been the basis of calculating open channel flows in sewers for many years. In U.S. customary units, Kutter's formula is

$$V = \frac{(1.81/n + 41.67 + 0.0028/S)}{1 + n/\sqrt{R}(41.67 + 0.0028/S)} \sqrt{RS}$$

where V = mean velocity, ft/s

R = hydraulic radius

S = slope of the energy grade line

n = roughness coefficient

In practice, most engineers use the simpler Manning formula, since the n value is essentially the same as Kutter's n . The Manning formula is

$$V = \frac{1}{n} R^{2/3} S^{1/2} \quad (\text{metric units})$$

or

$$V = \frac{1.49}{n} R^{2/3} S^{1/2} \quad (\text{U.S. customary units})$$

The roughness coefficient n varies with respect to pipe material, size of pipe, depth of flow, smoothness of joints, root intrusion, and other factors. For design of new sewers, the pipe material, or its interior liner, is used to select an n value. Table 6.25 shows ranges of Manning's n for common sewer pipe materials. Typically, an n value of 0.013 is used in sewer design.

In lieu of calculations, an alignment chart or special slide rules are available for solving Manning's formula. Figure 6.17 is such an alignment chart for a circular pipe flowing full.

TABLE 6.24 Minimum Gravity Sewer Line Slopes

Sewer size		Minimum slope, ft/100 ft (m/100 m)
in	cm	
8	20	0.40
10	25	0.28
12	30	0.22
15	38	0.15
18	46	0.12
21	53	0.10
24	61	0.08
30	76	0.058*
36	91	0.046*

*Minimum practical slope for gravity sewer line construction is about 0.08.

TABLE 6.25 Ranges of Manning's n

Pipe material	Manning's n
Asbestos cement	0.011–0.015
Brick	0.013–0.017
Clay	0.010–0.015
Concrete	
New	0.011–0.015
Monolithic	0.012–0.017
Iron	
New	0.012–0.016
Cement-lined	0.011–0.014
Plastic	0.010–0.015
Steel	
Welded	0.010–0.014
Riveted	0.013–0.017

Force Main. For force mains (pressure sewers), velocity is the primary design factor, and it is dependent roughness associated with the pipe material. Typically, a design velocity of 2 ft/s (0.6 m/s) at average daily flow is used. Maximum velocities are considered in the range of 8 to 12 ft/s (4.8 to 7.2 m/s). The maximum velocity dictates the pressure condition for design of fittings and reaction blocking.

The Hazen–Williams formula is used commonly for force main design. The velocity equation form is

$$V = 0.85CR^{0.63}S^{0.54} \quad (\text{SI units})$$

or

$$V = 1.32CR^{0.63}S^{0.54} \quad (\text{U.S. customary units})$$

where V = mean velocity, m/s (ft/s)

C = roughness coefficient

R = hydraulic radius

S = slope of the energy grade line

The value of the Hazen–Williams C varies with the type of material as shown in Table 6.26 and is influenced by the type of construction and age of the pipe. Common design practice uses values of 100 for unlined and 120 for lined force mains as representative of conditions during normal service periods.

System Components

All sewer systems will include service connections and manholes. Other system components may include pump stations, stream and aerial crossings, and depressed sewers. Also, protection of drinking water may be required.

Service Connections. Service lines, generally on private property, are those lines from a house, commercial establishment, or industry that transport wastewater to the sanitary sewer system. These lines are typically installed on a 2% grade using 4 to 6 in (15 to 20 cm) or larger pipe. One percent is considered as minimum grade for house connections. Larger dischargers should conform with collection system design standards.

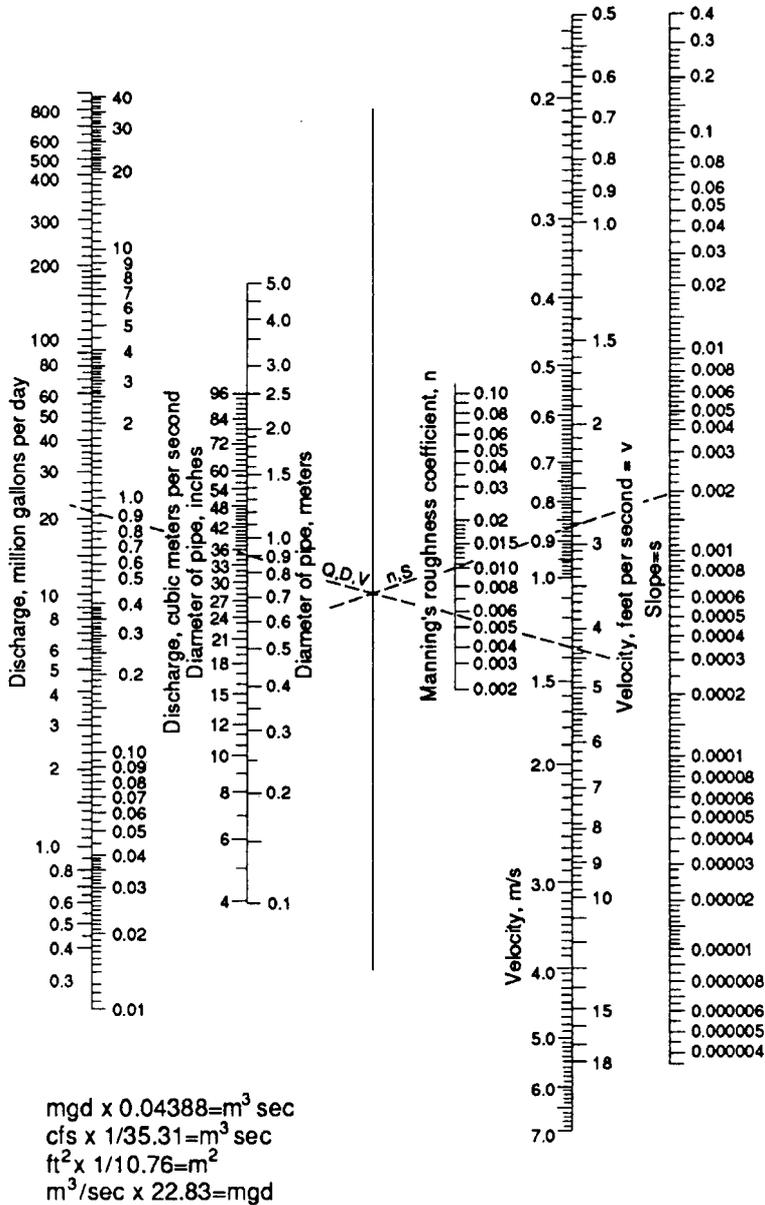


FIGURE 6.17 Alignment chart for Manning's formula.

TABLE 6.26 Ranges of Hazen–Williams *C*

Pipe material	Hazen–Williams <i>C</i>
Asbestos cement	120–140
Clay	100–130
Concrete*	100–140
Iron*	100–130
Plastic	130–140
Steel	110–120

*Upper range for lined pipe.

Wyes and tees are sometimes installed in smaller collection sewers and are the preferred method for connecting service lines. They are used when location of services can be predicted by existing development or are provided at assumed locations for future development. Also, wye or tee fittings can be used to replace existing sewer pipe, or a tee-saddle or tee-insert can be cut into the existing pipe. These are the several preferred methods to provide a good service connection and to thus avoid exfiltration or infiltration.

Other less effective service connections are much more highly dependent on workmanship and include breaking a hole in the pipe and grouting in a stub. Local plumbing and building codes should be studied to learn if such service connections are legally permissible.

Manholes. Properly designed and maintained manholes are essential to a functional sanitary sewer system. Manholes are installed at all changes in slope, pipe size, and alignment, at all intersections, and at the end of a sewer line. Minimum spacing for access varies with pipe size. Typical manhole design data are presented in Table 6.27.

TABLE 6.27 Manhole Design Data (21, 46)

Parameter	Typical value
Minimum distances	
Up to 15 in (38 cm)	400 ft (120 m)
18–30 in (48–76 cm)	500 ft (150 m)
36 in (92 cm) and larger	Varies
Minimum diameter	
Base	48 in (122 cm)
Access	22 in (56 cm)
Maximum turns/angles	
Up to 24 in (60 cm)	90°
27–48 in (70–120 cm)	45°
Larger pipe or turns	Special construction
Steps	
Width, minimum	12 in (30 cm)
Intervals	12–16 in (30–50 cm)
Frames	
Height	6–12 in (15–30 cm)
Weight	250–500 lb (115–250 kg)
Covers	
Weight	100–160 lb (45–70 kg)

Brick masonry was used almost exclusively until recent years, when the requirement for infiltration control and the cost of, and time for, installation all increased. Currently, precast concrete manholes are most commonly used. Poured-in-place concrete, concrete block, and large pipe sections are alternatives for special applications.

Half sections of standard and drop manholes are shown in Figure 6.18. Drop manholes are used when the difference in inverts is 24 in (60 cm) or more; outside drop connections are preferred. When the invert elevations differ by less than 24 in, the invert should be filleted to prevent solids deposition.

The manhole invert should conform in shape and slope to that of the sewer line. In fact, sometimes a section of pipe is laid through the manhole and the upper portion is sawed or broken off. More commonly, U-shaped channels are specified.

As shown by an example in Figure 6.19, a different type of manhole connection is required to dissipate energy from force main tie-ins. The termination connection should be no more than 2 ft (30 cm) above the gravity sewer flow line at the receiving manhole.

Pump Stations. In the wastewater collection system, pump stations (also known as lift stations) raise the wastewater to a higher elevation to continue gravity flow at reasonable slopes and depths and transfer wastewaters between gravity drainage basins. The pump station may be built in place or supplied as a package unit for smaller flows.

Pump station–force main systems typically use centrifugal pumps. Pneumatic ejectors are considered for smaller stations. Positive-displacement screw pumps are a viable alternative when only an on-site lift is required.

A variety of centrifugal pump types are used for wastewater pumping. The pump station is designed in a wet well–dry well configuration unless submersible pumps are used. The wet well receives the wastewater flow and is sized for 10 to 30 min detention with bottom slopes of 2:1. The dry well is the pump and motor area. Ventilation, humidity control (dry well), and standby power supply are important design considerations.

For smaller stations—less than 1 mgd (3800 m³/d)—at least two pumps should be provided. Three or more pumps are used in larger pump stations. In both cases, the pump capacities should be sufficient to pump the maximum flow rate if any one pump is out of service.

To avoid clogging, the associated pump suction and discharge piping should be at least 4 in (10.2 cm) in diameter. Also, the pump should be capable of passing 3-in (7.6-cm) diameter solids. Especially for layer influent sewers, bar screens or comminutors are used to protect the pumps from clogging or damage.

Stream Crossings. In order to maximize gravity flow, many sewers parallel streams and rivers. When a gravity sewer (or force main) crossing is required, it should be made as perpendicular as possible while maintaining grade. The construction may be performed subaqueously or dry through the use of cofferdams.

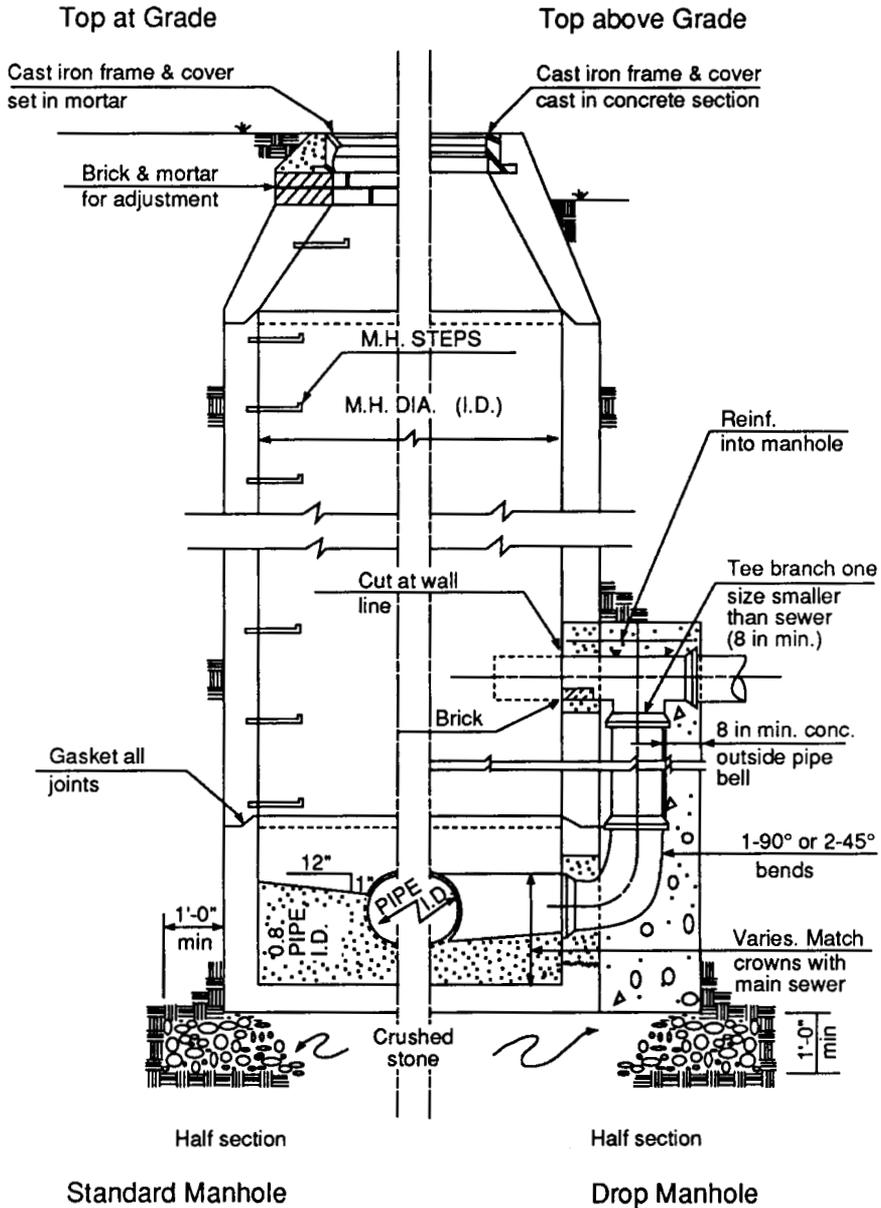
The design of the sewer should be at a sufficient depth below the stream bottom to protect the sewer line. Some guidelines for cover requirements are (46):

- 1-ft (0.3-in) cover when located in rock
- 3 ft (0.9 in) or more when not a rock bottom
- Below pavement in paved stream channels

Cast or ductile iron pipe with mechanical joints is used frequently.

When aerial stream crossings are used, the pipe should be above frequent flood elevations, and the collection of debris should be considered. Inverted siphons should be reviewed as an alternative.

Aerial Crossings. In some cases, local topography and good design practice make it desirable to use aerial crossings of streams or of the ground surface at gullies and valleys. This applies to gravity sewers and force mains.



Scale: None

FIGURE 6.18 Typical manhole sections.

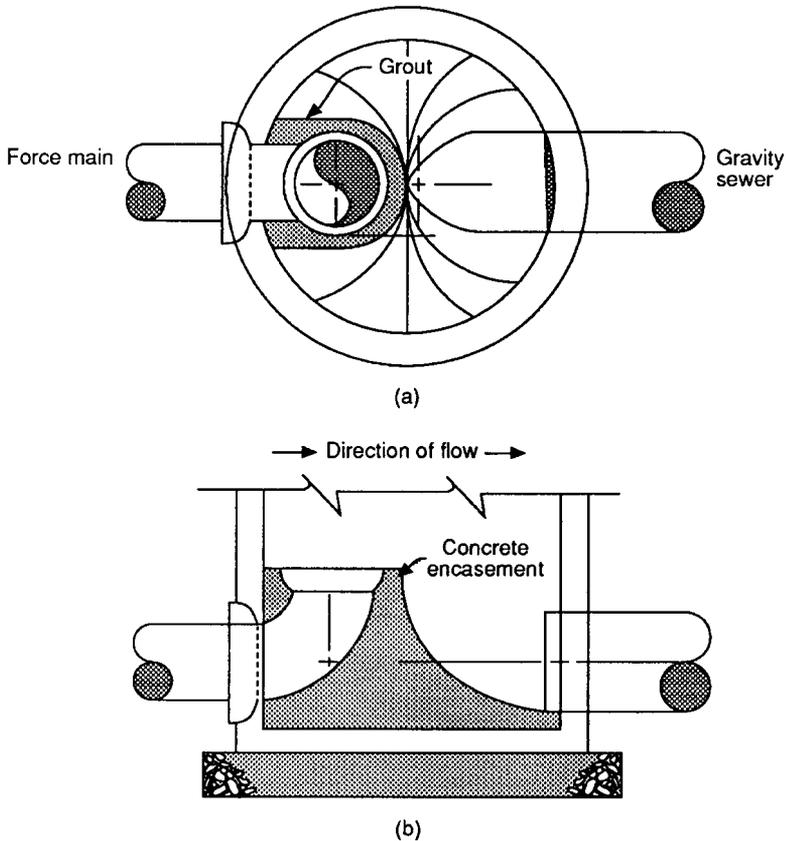


FIGURE 6.19 Typical force main connection to manhole. (a) Plan; (b) section.

Piers are designed to support each pipe joint with provisions to prevent flood damage, frost heave, overturning, and settlement. Figure 6.20 shows typical concrete pier details.

Pipe insulation and/or increased slope are used to protect against freezing. Expansion joints are used between above- and below-ground sewers. If attached to a structure, i.e., bridge, with expansion joints, the pipe expansion joints should match those on the structure.

Depressed Sewers. In order to regain elevation, depressed sewers (also called inverted siphons) may be used to carry flow under obstructions, such as streams or subways. The sewer line (referred to as barrel) is below the hydraulic grade line and is thus always filled with wastewater. Because of maintenance and cleaning difficulties, depressed sewers are avoided as much as possible.

It is common practice to have at least two barrels between manholes on each side of the obstruction. Normally, in the case of two barrels, average daily flow can be carried by each barrel. To minimize the need for cleaning, a minimum velocity at average daily flow is 3 ft/s (0.9 m/s). The inlet and outlet features should permit interchanging for cleaning and maintenance, as well as staging of the barrels for peak conditions. If more than two barrels are used, at least one, the primary barrel should be capable of carrying the average flow.

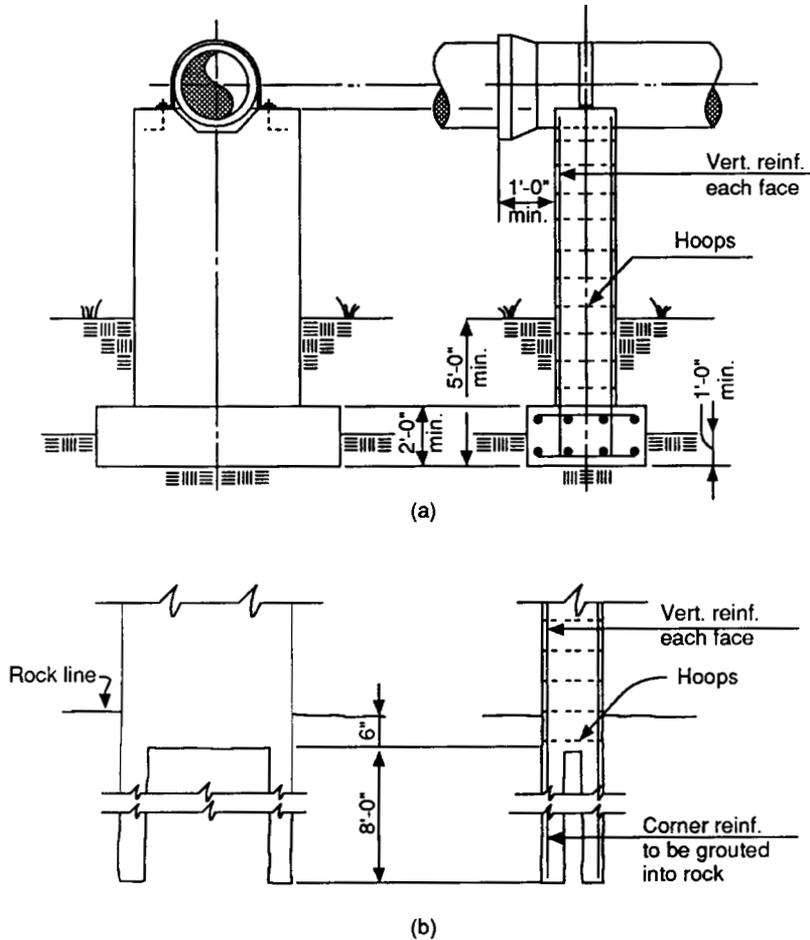


FIGURE 6.20 Reinforced concrete pipe pier details. (a) Footing in earth; (b) foundation in rock.

Drinking Water Protection. Obviously, sewer lines should be located away from wells and other drinking water supply sources as well as water transmission mains. Local and state agencies may have specific requirements with respect to drinking water protection.

When proximity to water supplies or mains cannot be reasonably avoided, the following guidelines for gravity sewers and force mains should apply.

- Use pressure-type sewer pipe.
- Do not use the same trench for sewers and water mains.
- Provide at least 10 ft (3 m) horizontal separation between lines.
- Install sewer at least 1.5 ft (0.5 m) vertical separation below the water main.
- Cross lines as near perpendicularly as possible.

- Encase the sewer in concrete to give additional protection when poor vertical or horizontal separation cannot otherwise be avoided.
- Provide identification markings when the sewer line material or appearance may be confused with a water main.

Sewer Materials

An assessment of pipe and joint materials is available for gravity and pressure sewer lines. In some cases, special materials are required for internal and/or external corrosion protection.

Pipe Materials. A wide variety of materials have and are being used for sewer pipe. Some of the more common pipe materials are discussed below. Other materials not presented include brick masonry and corrugated or welded steel.

Asbestos Cement Pipe. Gravity and pressure sewers are available in asbestos cement pipe with diameters of 4 to 42 in (10 to 107 cm). The pipe's relatively light weight, ease of handling, long laying lengths, tight joints, and wide range of strength classifications are favorable factors in pipe selection. Since asbestos cement pipe is subject to corrosion by acids, protective linings may be required.

Clay Pipe. Vitrified clay pipe is available in diameters of 4 to 42 in (10 to 107 cm) and lengths of 3 to 6 ft (0.9 to 1.8 in). It is widely used for small- to medium-sized gravity sewers. Since it is chemically inert, clay pipe resists internal and external corrosion from acids and alkalis. Use of clay pipe is limited by its low strength and its brittle nature.

Concrete Pipe. Depending on its intended use, concrete pipe can be produced in a variety of shapes—e.g., circular, elliptical, or arch—and strengths. Variables affecting strength include concrete strength, wall thickness, use of coatings, reinforcing materials, and/or prestressed elements. The more typical concrete pipes are prestressed concrete and reinforced concrete pipes, which are circular with diameters of 40 to 360 cm (16 to 144 in) and 30 to 360 cm (12 to 144 in), respectively. Also, concrete pipe may be produced to satisfy requirements for pressure as well as gravity lines. The major drawback to concrete pipe is that internal and/or external corrosion protection is required in the presence of acidic conditions, including hydrogen sulfide.

Iron Pipe. For force mains or gravity sewers subject to damage, such as aerial sections and stream crossings, iron pipe is commonly used. Pipe diameters range from 2 to 48 in (5 to 122 cm). Iron pipe has special advantages of tight joints, long laying lengths, external load-bearing capacity, and compact resistance. Even though coatings and/or liners are required in highly acidic conditions, iron pipe does have more relative resistance to corrosion than concrete pipe. Also, iron pipe is available in a number of lengths, thicknesses, classes, and strengths. For the most part, ductile iron, with its higher ductility, strength, and impact resistance, is replacing cast iron in new sewer line designs.

Plastic Pipe. Plastic pipe, typically polyvinyl chloride (PVC), is available in diameters of 4 to 15 in (10 to 37.5 cm) and in lengths of 10 ft (3.2 in). Its light weight, tight joints, long lengths, and corrosion resistance make it an alternative mostly to asbestos cement and vitrified clay pipe. However, its lateral support requirements and thin walls are unfavorable characteristics. Truss pipe, constructed by a thermal process, has characteristics similar to the PVC solid-wall plastic pipe.

Joints. The joint between pipe sections is important to the maintenance of the hydraulic continuity and structural integrity of the sewer line as well as for inhibition of infiltration and root intrusion. The primary factors affecting performance are the characteristics of the joint and the quality of installation. As with the pipe itself, corrosion can affect gasket or packing compounds. A wide variety of joints exist, but only the more commonly used are discussed below.

For asbestos cement pipe, a collar joint is used in which two rubber rings provide seals around two inserted spigots. The joint is very watertight and is suitable for force mains.

Compression, or push-on, joints are used for vitrified clay pipe. A plastic material is cast onto the spigot and into the bell and a rubber compression ring (O-ring) is used to make the seal. This can provide a very tight line and still exhibit some flexibility and resistance to shear loading. Though less common, collar joints can be used.

Some of the types of joints used for concrete pipe include:

- *Cement mortar or mastic packing:* These joints are rigid and watertightness is totally dependent on workmanship.
- *Compression-type rubber gasket with or without opposing shoulders:* These joints are suitable for gravity sewers.
- *O-ring-type gasket with opposing shoulders:* These joints are suitable for gravity sewers and force mains. When the spigot is grooved for the O-ring gasket, a high-pressure joint is provided for force mains, or it becomes a factor in gravity sewer design when infiltration must be significantly controlled.

For ductile iron pipe, there are a variety of joints available, including:

- *Push-on (compression O-ring) type:* These joints are useful for gravity systems and are the cheapest type to install.
- *Mechanical type:* These joints are mostly used for force mains.
- *Ball type:* These joints are used for submerged piping and provide freer turning deflection.

Welding or bonding techniques are used for jointing plastic pipes. As with other pipes, special provisions are needed at manhole connections and when the pipe material changes, such as at stream crossings.

Corrosion Protection. Sewer lines may be subject to internal corrosion by acidic or alkaline industrial wastewaters or hydrogen sulfide gas and/or to external corrosion by nonneutral soils. Clay and plastic pipe materials are highly resistant to corrosion, as is ductile iron pipe, but to a much lesser extent. Thus, these pipe materials should receive first consideration when a corrosive environment exists.

The alternative is to provide internal and/or external protection for asbestos cement, concrete, iron, and steel pipe. Examples of protection techniques include application of

- Bituminous and coal tar products
- Vinyl and epoxy resins
- Plasticized PVC sheets
- Cement lining
- Polyethylene lining

For concrete pipes, other alternatives are to use type II versus type I Portland cement; to provide extra (sacrificial) wall thickness; or to use limestone or dolomite aggregate. The latter approach requires specific tests on the reaction of the specific aggregate supply to be used by the manufacturer.

Sanitary Sewer Overflows

A sanitary sewer overflow (SSO) is the discharge of untreated or partially treated sewage from a sanitary sewer system and occurs when flow exceeds the collection system capacity. A SSO may result from a variety of causes, such as sewer line backups due to localized blockages, surcharging through manhole covers, and overflows through constructed outlets. Overflows may occur during dry weather, but they commonly are as-

sociated with wet weather conditions, when infiltration and/or inflow overload the sewer system. Infiltration/inflow is addressed in the next section of this chapter (96).

Addressing the problem of SSOs requires a range of identification and evaluation efforts, design and construction of wastewater facilities, rehabilitation of existing facilities, and operations and maintenance programs. Potential work can include water quality impact assessment; collection system monitoring and modeling; infiltration/inflow studies; sewer system evaluation surveys; sewer system rehabilitation design and construction services; design of relief facilities; including pipelines, pump stations, and additional treatment facilities; and design of wet weather storage and treatment facilities.

SSO Issues. SSOs are a concern due to a number of potential adverse impacts on health, water quality, and economic interests. The discharge of untreated or partially treated sewage represents a health risk due to the presence of pathogenic organisms. SSOs often occur in locations where public access potential is high and contact with the raw sewage may occur. For example, a SSO may result in basement flooding or other locations where human contact may occur. Other high-exposure areas include streets and small drainage streams in residential areas.

SSOs also can cause adverse impacts on water quality in receiving waters. In addition to pathogens, raw sewage contains pollutants that exert oxygen demand and may contain toxins, which in turn can affect water quality and the aquatic environment. Other potential impacts on receiving waters include use impairment, such as beach and shellfish bed closings in coastal areas and swimming prohibitions in inland areas. Nutrient loading may accelerate eutrophication. Because raw sewage contains other pollutants, such as oil and grease and other materials classified as floatables, SSOs can cause aesthetic degradation.

A third area of impact of SSOs is property damage and loss of economic activity. Surcharged collection systems may back up into basements through fixtures or open cleanouts, causing damage. Also, pump stations may flood, damaging mechanical and electrical equipment when flows exceed pumping capacity. Beach closings and aesthetic degradation of water bodies may result in loss of economic activity, including tourism and convention business. Enforcement action by regulatory authorities can lead to fines and penalties as well as sewer connection moratoriums, limiting economic growth and development. Finally, significant legal and financial resources may be required to deal with regulatory enforcement actions or citizen lawsuits.

SSO Sources. A sanitary sewage collection and conveyance system will surcharge when the flow exceeds the capacity of the pipe flowing full. Surcharging will result in a SSO when the hydraulic grade line rises to a level where the system is open to the environment. Locations where a surcharged sewer can overflow include fixtures or open cleanouts in homes, manhole covers in streets or easements, or in constructed overflows to a receiving water or storm sewer. Constructed overflows exist in separate sanitary sewer systems mainly to mitigate property damage and health risks. A surcharged sewer can also exfiltrate raw sewage through open joints to the groundwater.

A sanitary sewer system also can overflow at a pump station or treatment facility if flows exceed the pumping capacity of the station or the hydraulic capacity of the headworks or downstream facilities. Constructed overflows may also be located at pump stations and at treatment plant headworks to mitigate damage to the facility.

A constriction in the sanitary sewer system can be due to an undersized pipe but may also occur when the capacity of the pipe has been reduced. Reductions in pipe capacity can occur through obstructions including roots, grease, debris, broken pipe, or joint failure. Debris can enter the sewer from construction activities, by vandalism, and even from sewer maintenance work.

Obstructions can exacerbate surcharging and backups by causing a buildup of solidified grease, sticks, rags, plastic bags, brick, rocks, sand, eggshells, and silt. Constrictions also may occur due to deposition of materials, such as grit during low velocities associated with minimum flows and in reaches with very flat slopes.

Infiltration and inflow (I/I) are extraneous flows entering the sanitary sewer system and are associated

with wet weather. A SSOs will occur when I/I flows produce a peak flow in the sewer that exceeds its capacity. The ratio of peak wet weather flow rate to the average dry weather flow can be used as a parameter to determine if a particular area of the sewer system will overflow. Typically, as the peaking factor approaches 4 to 5, the likelihood of surcharging and overflows increases. The peak flow ratio at which overflows occur varies depending on the specific characteristics of the system.

SSO Abatement. SSOs can be reduced in a number of ways, including:

- Sewer system maintenance
- Reducing peak flows by removing I/I
- Wet weather storage and treatment facilities
- Increasing conveyance and treatment capacity

Sewer System Maintenance. Many sewer systems receive little or no preventive maintenance and need enormous rehabilitation programs. Most municipalities only provide maintenance when an emergency, such as clearing a blockage or repairing a collapse, necessitates action. Factors that contribute to the high level of needed rehabilitation include:

- Poorly planned and staged systems with no provision for long-term growth
- Poor installation, causing unnecessary maintenance problems
- Materials that do not withstand settling of structures, earth movement, or traffic
- Joints that were not designed to prevent root intrusion

A preventive maintenance program to reduce SSOs would address areas that become plugged by regular hydraulic, mechanical, or chemical cleaning and root removal, as well as line replacement to eliminate hydraulic restrictions. Often during an emergency repair, the repair crew will discover a damaged pipe nearby that can be repaired as part of the preventive maintenance program. A collection system preventive maintenance program should also include pump stations. Procedures would be based both on recommendations of equipment manufacturers and knowledge gained from experience.

Reducing Peak Flow by Reducing I/I. Many I/I reduction programs have had limited success. One reason is that I/I from building laterals on private property often are excluded from rehabilitation programs. An estimated 50 to 70% of I/I may originate from sources on private property, and local governments generally are unable or reluctant to address problem sources on private property. A building lateral can serve as a connection point for area drains, roof leaders, and foundation drains. If these connections, which are usually illegal, are removed, alternative outlets often are not readily available.

Wet Weather Treatment Facilities. A wet weather storage and treatment facility can provide hydraulic relief to the collection system. The objective of such a facility is to provide storage of flow for smaller wet weather events, and treatment with a controlled discharge during larger wet weather events. The sizing of the facility could be based on providing a minimum treatment level, meeting water quality standards, and/or reducing the number of overflow events. Typically, wet weather treatment facilities are designed to provide for sedimentation, disinfection, and removal of floatables. This is analogous to combined sewer overflow (CSO) treatment facilities; combined sewers were designed and constructed years ago in metropolitan areas for the purpose of conveying both stormwater and sewage.

Increasing Conveyance and Treatment Capacity. In order to increase conveyance capacity, an evaluation of the collection system must be made to identify and eliminate restrictions. This can be accomplished by inspection, monitoring, and modeling of the system to characterize its features and behavior. To eliminate restrictions, a relief sewer can be constructed, or undersized pipe sections can be enlarged. Another method is to transfer flow from one drainage basin to another. If a SSO occurs due to a capacity limitation at a pump station, pumping capacity can be increased.

A number of “in-system” technologies or strategies can be used for in-line storage. These are generally most effective with large-diameter pipelines and require careful analysis to ensure that flooding of service connections will not occur. Disadvantages include the opportunity for sediment deposition and increased maintenance costs.

Infiltration/Inflow

The extraneous water that enters a sanitary sewer system utilizes the design capacity and increases the cost of wastewater collection and treatment facilities. The flows are generally attributed to infiltration and inflow, which are illustrated in Figure 6.21.

Infiltration is the groundwater that leaks into the sewer system, generally through deteriorated and/or defective pipes, joints, connections, and manholes. Usually, this flow fluctuates seasonally with the movement of groundwater.

Inflow is the stormwater that directly enters the sewer system after rainfall from such sources as flooded manhole covers, yard and roof drain connections, and cross-connections with a storm sewer. Generally, inflow causes sudden increases in the wastewater flow rate.

Systematic studies of the sanitary sewer system may be used to locate defects that are cost-effective to repair. These procedures are summarized in Figure 6.22.

Infiltration/Inflow Analysis. The initial phase of a sewer study is the infiltration/inflow (I/I) analysis. This study uses historic data and in-system flow measurements to establish the nonexistence or possible existence of excessive extraneous flows on a system and subsystem basis and to justify any further study by a more in-depth sewer system evaluation survey.

Background information and field studies are analyzed to determine the extent of infiltration/inflow in the system. Historic and current data include flow records; environmental factors; sewer maps; engineering

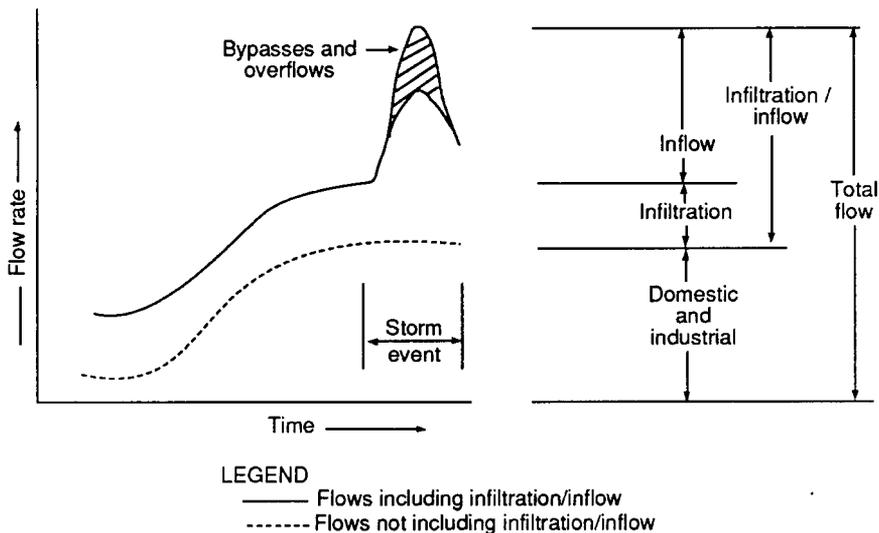


FIGURE 6.21 Characteristic infiltration and inflow hydrograph (47).

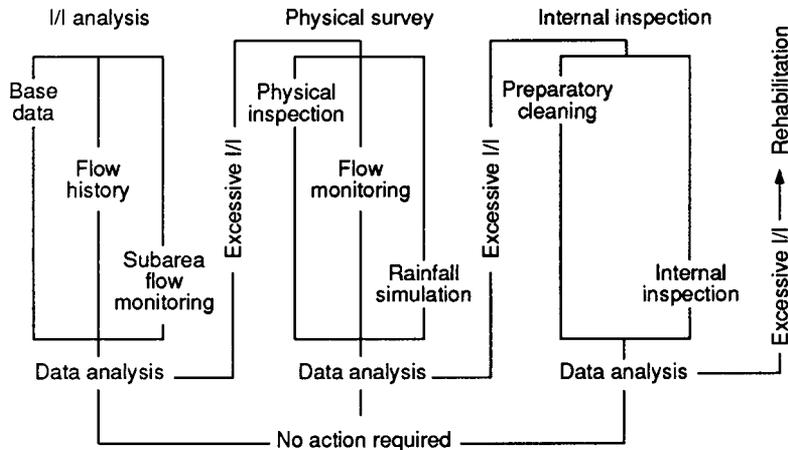


FIGURE 6.22 Sewer system study procedures.

plans, specifications, and reports; construction practices; observed and reoccurring problems; and applicable local ordinances. Employee interviews are an important source of data.

Field studies include some verification of documented problems and a general overview of the system. The major effort is directed to collection and analysis of wastewater flow rates at the treatment plant and selected manholes and/or pump stations in the sewer systems. Means are needed to measure and record rainfall and fluctuations in the level of groundwater.

The flow-monitoring data should span at least one storm event and provide sufficient data to produce a sewage flow (and rainfall) hydrograph. Bypasses and overflows should be identified and described, and the quantity, duration, frequency, etc., of discharge should be included as much as practical.

Continuous recording devices are available for measurement of in-system flows at selected manholes. The devices work as a function of depth of flow or in conjunction with specific weir or flumes. Manual measurements and dye-dilution are other methods that have found application.

The field studies provide an estimate, by service area, of the infiltration/inflow present. Available cost information is used to estimate and compare the cost of transporting and treating the infiltration/inflow and of an appropriate elimination measure by individual areas. All flows that are more expensive to transport and treat than to find and rehabilitate are designated as excessive infiltration or inflow.

The final step of the I/I analysis is to propose action to eliminate the excessive infiltration/inflow. For areas where the origin of the infiltration/inflow are clearly identified, design and construction of appropriate corrective measures should be proposed. For areas where the source is unknown, a more thorough sewer system evaluation survey should be proposed, after which rehabilitation will be defined.

Evaluation Survey. A sewer system evaluation survey (SSES) is a two-phased, systematic study to determine the exact location, flow rate, and rehabilitation costs of an I/I problem. The first phase uses physical inspection along with rainfall simulation to define specific problems and avoid unnecessary further study. The second phase uses internal inspection to pinpoint problems.

Physical Survey. The physical survey phase of an SSES combines physical inspection of manholes, lamping of lines, flow monitoring, flow isolation, and rainfall simulation to isolate sources or areas of excessive infiltration/inflow. Each manhole and reach of sewer line is included in the survey; typical inspection forms are shown in Figures 6.23 and 6.24, respectively.

The inspection of manholes is used to identify the type and condition of materials. The inspector looks

MH# _____ Location _____ Drainage Area _____
 Project _____ Project No. _____
 Client _____
 Date ____ / ____ / ____ Time _____ a.m.
 _____ p.m. Inspector _____
 Weather: No rain _____ rain _____ snow _____

A. INVENTORY

Item	Construction material	Condition		
		Good	Fair	Poor
Cover				
Ring (frame)				
Steps				
Corbel work				
Walls				
Aprons				
Troughs				

B. OBSERVATIONS

- Diameter of manhole opening _____ in
- Holes in cover ____ number ____ size _____ in
- Visible infiltration through joints, cracks, lines, etc., height above sewer invert, estimated flow rates _____

- Evidence of leaks in manhole, height above sewer invert

- High-water marks in manhole, height above sewer invert, possible cause _____

- Manhole has ____ does not have ____ ground water level gauge
 ground water level above sewer invert _____
- Debris: no ____, yes ____, type _____, depth ____ in
- Special problems, conditions, sources of inflow, overflows, bypasses.

FIGURE 6.23 Manhole inspection form (48).

WASTEWATER DISPOSAL

CHAPTER SIX

6.62

MH# _____ Location _____ Drainage Area _____

Project _____ Project No. _____

Client _____

Date ____ / ____ / ____ Time _____ a.m.
 _____ p.m. Inspector _____

Weather: No rain _____ rain _____ snow _____

Pipe Data	Incoming Lines			Outgoing Lines
	From MH# ____ Distance _____ ft	From MH# ____ Distance _____ ft	From MH# ____ Distance _____ ft	To MH# ____ Distance _____ ft
Pipe sizes, in				
Type of pipe				
Depth from MH top to invert				
Depth of flow, in				
Temperature of flow, °F				
Conductivity of sewage, mho/cm				
Sulfate concentration, ppm				
Fluoride concentration, ppm				
Root growth in pipe				
Type of deposition				
Depth of deposition, in				
Recommended cleaning method				
Visible infiltration/inflow sources in pipe				
Structural conditions of pipe				
Special problems and conditions				

Sketch of Manholes and Sewers:

FIGURE 6.24 Sewer line inspection form (48).

for visible infiltration, evidence of leaks, or other indicators of infiltration or inflow, such as high water marks and perforated manhole covers. Similar data are collected for the connected sewer lines. Typically, a single form is developed and used throughout the system.

Flow monitoring is used to better define areas requiring study and to quantify infiltration by individual sewer line reach. Continuous flow monitoring by subarea, as more broadly defined in the I/I analysis, helps to identify smaller areas for infiltration and/or inflow study. Using weirs or other instantaneous flow measurements in the early morning (low-base-flow) period, an estimate of infiltration can be made on a reach-by-reach basis.

Using results of the physical survey, reaches and subareas are better defined for more study by use of rainfall simulation techniques.

Rainfall Simulation. Smoke testing is the most frequently used rainfall simulation technique and provides a means to locate mostly inflow sources. Infiltration sources for shallow lines (mostly house service connections) are identified by the appearance of ground smoke.

Flooding with dyed water is used also to locate inflow sources. Dyed water is added to storm sewers or streams that parallel or cross a sanitary sewer; if inflow is present, the dye will be evidenced at the downstream sanitary sewer manhole. Infiltration may be located by surface flooding or injection of dyed water over the suspect reach of sewer.

Air testing of reaches is another technique for locating sources of infiltration. Air testing of individual joints is used during test-and-seal rehabilitation.

Preparatory Cleaning. The analysis of data from the physical survey and rainfall simulation will identify areas for further study by internal inspection. Preparatory cleaning is necessary to provide the camera with an unobstructed view of the pipe wall to permit location of structural defects, misalignment, and other problems.

The cleaning phase requires the dislodging of grease, roots, and other debris; transporting the material to a point of access, e.g., a manhole; removing the material from the sewer; and disposing of the material at an approved site, such as a landfill.

The cleaning equipment for dislodging material is categorized by four basic types:

1. Bedding machines, which are good for roots and other blockages
2. Bucket machines, which are good for moving sand, gravel, roots, and similar debris
3. High-velocity water machines, which are good for transporting sludge, mud, and sand
4. Hydraulically propelled machines, which are good for moving sludge, mud, and sand.

All of these devices are used for small, 8-in (20-cm) pipes; bucket and hydraulically propelled machines are used more commonly for larger—over 15-in (38 cm)—pipes. The type and quantity of debris, access, structural condition of pipe and manholes, and cleanliness required are some of the important considerations in selection of cleaning techniques.

Once the debris is moved to a point of access, it may be removed by bucket and shoveling, a vacuum device, or a trash-type pump. Bucket (cleaning) machines can also be used to remove debris from the sewer. A variety of vehicles or trailers are available to transport the debris to a point of off-site disposal.

Internal Inspection. Television inspection with a closed-circuit television camera is the most common internal inspection technique. Documentation is achieved with an inspection log (Figure 6.25) and photographs of the monitor and/or video recording. Figure 6.26 shows a typical arrangement for television inspection. Other techniques include photographic inspection, physical inspection, and air testing.

The purpose of the final study phase is to determine the exact location, condition, and estimated flow rate for each source of infiltration/inflow. When possible, it is best to conduct internal inspections during periods of high groundwater. In some cases, it may be desirable to use surface flooding with or without dyed water to simulate high groundwater or rainfall runoff conditions.

At the conclusion of the internal inspection phase, a cost-effectiveness analysis of applicable rehabilita-

WASTEWATER DISPOSAL

6.64

CHAPTER SIX

Area _____ Sheet No. _____

Project _____ Project No _____ Date _____

Client _____ Inspector _____

Section on _____

From Manhole # _____ To Manhole # _____

Pipe Size _____ in, Pipe Type _____ Pipe Length _____ ft Type of Joints _____

Ref. no.	Footage	Observations	Infiltration/ inflow, gpm	Recommended corrective action	Photo no.
1					
2					
3					
4					
5					
6					
7					
8					
9					
10					
11					
12					
13					
14					
15					
16					
17					
18					
19					
20					
21					
22					
23					
24					
25					

SPECIAL NOTES _____

MH# _____ MH# _____

○ _____ ○ 

Direction of flow 

Direction of measurement 

FIGURE 6.25 Internal inspection log form (48).

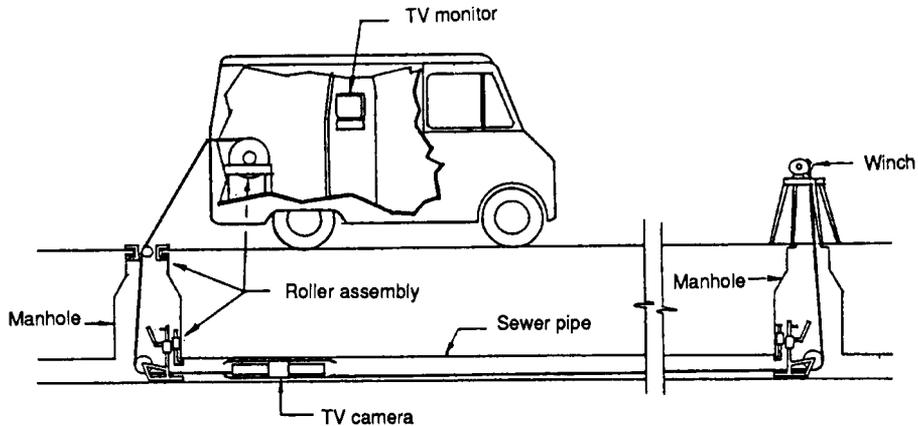


FIGURE 6.26 Typical arrangement for television inspection (48).

tion methods is made. The results appear in a final SSES report specifying the cost, location, and type of rehabilitation needed to correct problems of excessive infiltration/inflow.

Rehabilitation

Sewer system rehabilitation may occur after the simple observation of a failure, such as a collapsed pipe, or after an in-depth SSES. The selection of a rehabilitation method is first dependent on the sources, infiltration or inflow, and extent of the problem. Cost analyses are used to select between appropriate means.

Infiltration Correction. The most frequently used methods for rehabilitating infiltration sources are

- Replacement
- Grouting
- Lining

These and other techniques are shown in Table 6.28, which summarizes possible corrective methods with respect to types of infiltration sources.

Replacement. The excavation and replacement of pipes, connections, or manholes is an extremely effective rehabilitation. The principal disadvantages are the length of time required for correction and the cost. Table 6.29 shows important factors to be considered in a cost-effective analysis of the replacement alternative.

The application of replacement is particularly appropriate when the pipe or manhole has lost its structural integrity due to internal (e.g., hydrogen sulfide) or external (e.g., soil consolidation) actions. In some cases, there may be need to increase capacity while eliminating an infiltration source.

Grouting. Chemical grouting is widely considered for sealing leaking pipe joints and manholes when they are structurally sound. The success of this method is largely dependent on the type of problem, selection of grout, and techniques of application. Criteria for use in a rehabilitation cost-effectiveness analysis are shown in Table 6.30.

Different grouts are available. The basic types are one that mixes with the soil to decrease permeability

TABLE 6.28 Infiltration Sources and Rehabilitation Methods (48)

Infiltration sources	Rehabilitation methods
Collapsed pipe	Replacement; slip-lining
Broken or crushed pipe	Replacement; slip-lining
Cracked pipe	Slip-lining; lining with cement mortar or epoxy mortar; replacement
Deteriorated pipe joints	Chemical grouting
Leaking offset joints	Slip-lining; chemical grouting; replacement
Open joints	Slip-lining; chemical grouting; replacement
Deteriorated mortar joints in brick pipes	Brick mortar replacement
Leaking house service connections	Chemical grouting; slip-lining; replacement
Faulty taps between sewers and manholes	Excavation and repair
Faulty taps between service connections and main sewers	Excavation and repair
Collapsed manhole and wet wells	Replacement
Deteriorated manhole walls, bases and troughs	Lining with cement mortar or epoxy mortar; chemical grouting; cement grouting
Deteriorated mortar joints in brick manholes	Brick mortar replacement
Deteriorated wet wells and pumping station structures	Lining with cement or epoxy mortar
Other sources such as deteriorated regulators and tide gates	Repair according to situation

and one that actually solidifies in the leaking crack. Each can provide good results for correcting infiltration problems.

Grouting rehabilitation is often referred to as test and seal, since individual joints are air-tested to determine if sealing or grouting is required. The process is accomplished by pulling a television camera and packer through the sewer line; Figure 6.27 illustrates a typical arrangement of equipment used in grouting a sewer line.

The television camera is used to position the packer at each joint. It is emphasized that each joint in a

TABLE 6.29 Pipe Replacement Evaluation Criteria (48)

Size of pipe
Depth of pipe
Type of service
Type of pipe
Removal of existing pipe
Number of service connections to be made
Groundwater elevation
Proximity to other utilities
Pipe transportation requirements
Infiltration allowance requirements
Access to site work
Availability of storage area for pipe materials and equipment
Availability of storage area for excavated materials
Weather conditions
Availability and cost of labor

TABLE 6.30 Pipe Grouting Evaluation Criteria (48)

Mobilization distance	Pipe grade
Weather conditions	Pipe cleanliness
Terrain	Depth of flow
Type of soil	Flow rate
Access to manholes	Ability to plug
Manhole opening	Type of joint
Manhole size	Joint spacing
Manhole cleanliness	Offset joints
Manhole depth	Intruding service connections
Hazardous gases in manhole	Structurally damaged pipe
Type of pipe	Random versus successive manhole sections
Pipe size	Availability and cost of labor
Pipe alignment	

sewer reach be included. If only leaking joints are repaired, groundwater will migrate along the pipe until it begins to leak through another joint, which was not tight but was not leaking due to the initial absence of surrounding groundwater.

The packer is first used to air-test the joint and, as necessary, to pump the chemical grout. Wiering and water testing are preferred to air testing of grouted joints.

Lining. In structurally sound reaches, a polyethylene plastic pipe can be pulled through the sewer. This slip-lining process offers numerous advantages, due to the characteristics of the liner, and has less roughness (greater flow-carrying capacity) than most original pipes.

This approach is especially well suited for reaches with no or minimal connections or in site-specific cases where other techniques are not practical. Evaluation criteria for a rehabilitation alternative cost-effectiveness analysis appear in Table 6.31.

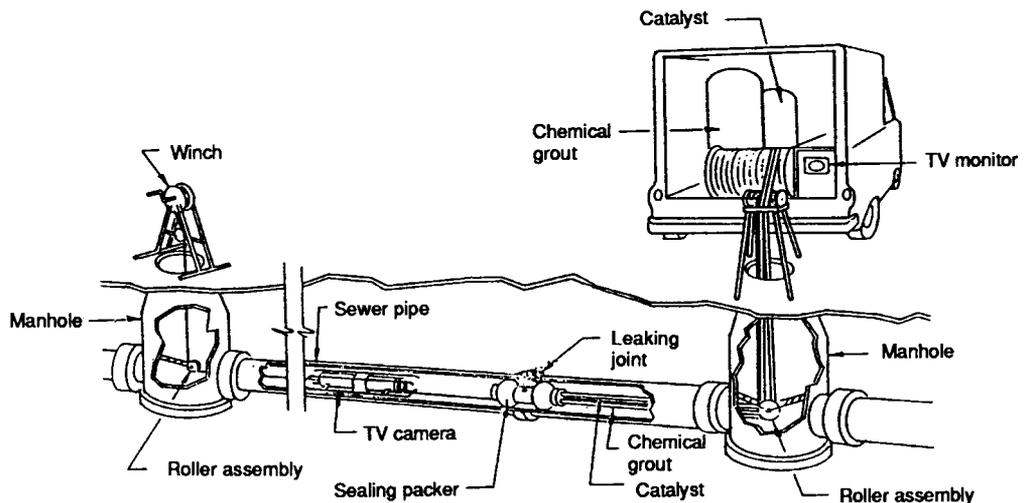
**FIGURE 6.27** Typical arrangements for chemical grouting (48).

TABLE 6.31 Pipe Lining Evaluation Criteria (48)

Size of sewer	Extent of sewer cleaning required
Length of sewer	Technique to “prove” or preinspect sewer lines
Depth of sewer	Excavation requirements
Grade and direction change of sewer	Groundwater elevation
Depth of flow in sewer	Access to site of work
Size of liner pipe	Availability of electrical power for fusing
Liner pipe wall thickness required	Availability of storage area for pipe materials and equipment
Annulus grouting requirements	Availability of storage area for excavated materials
Number of service connections to be made	Mobilization distance
Type of surface restoration required	Availability and cost of labor
Pipe transportation requirements	
Type of manhole “seals” required	

Inflow Correction. As shown in Table 6.32, plugging and disconnections are the most common methods of correcting inflow problems. For manholes, provisions must be made to keep surface waters out.

SUSPENDED SOLIDS TREATMENT

The origin and composition of wastewater solids dictates the complexity of suspended solids treatment processes and the alternatives available for individual wastewater treatment works. The suspended solids requiring treatment or removal include

1. The largely inorganic solids contributed by the wastewater source and the variety of other solids that invariably enter both municipal and industrial sewer systems
2. The solids containing high fractions of organics from the originating waste source
3. The inorganic and organic solids produced in the treatment works as a result of chemical and biological treatment process

TABLE 6.32 Inflow Sources and Rehabilitation Methods (48)

Inflow sources	Rehabilitation methods
Low-lying manholes	Manhole raising
Perforated manhole covers	Replace with watertight covers
Cross-connections	Plugging
Roof leader drains	Disconnection
Foundation drains	Disconnection
Cellar drains	Disconnection
Yard drains	Plugging
Area drains	Plugging
Cooling water discharge	Disconnection
Drains from springs and swampy areas	Plugging

The importance of suspended solids treatment cannot be overemphasized, since the selected processes are fundamental to the capital and operating cost of the treatment works as well as its operability and consistent capability to meet effluent limitations. The selected process must also have the flexibility to sustain a predetermined range of hydraulic and suspended solids loadings. Finally, the removal of floating solids and proper handling and disposal of sludges influence the aesthetics of the treatment facility.

Due to the diversity of suspended solids treatment requirements, the process selection must be intergrated with the overall treatment works' processes and treatment goals. This is illustrated by the following functions of suspended solids treatment.

1. Remove large objects in the influent stream in order to protect the treatment plant's equipment and to avoid obstructions in the downstream pipes and channels.
2. Reduce the solids loading on subsequent treatment processes. This also includes the damping of shock hydraulic and waste loads.
3. Collect active biomass after a biological process, e.g., activated sludge, for return to the process as seed stock for the incoming waste stream.
4. Collect and remove, for separate treatment, the excessive organic or biomass solids, e.g., waste activated sludge. This function also applies to waste chemical sludges.
5. Polish the final effluent by removing fine solids before discharge from the treatment works.

The processes commonly used for suspended solids treatment include screening, gravity setting, flotation, filtration, and mechanical systems, e.g., centrifugation. As in the treatment of drinking water supplies, chemical additives are useful, or even sometimes required, to improve the efficiency of these treatment processes.

Screens

The use of screens for suspended solids removal is one of the oldest processes used in wastewater treatment. Initially, coarse screens were used to remove unsightly solids, and later, they became important as protection for equipment in downstream treatment works. Today, fine screens are used also to remove significant quantities of suspended solids from the wastewater. This is particularly true in industrial applications.

Coarse Screens. The most common types of coarse screens are bar screens. In small plants, comminutors are frequently used. Other types of coarse screens include stationary and drum screens, using woven wire or other media.

Bar Screens. Both manual and mechanically cleaned bar screens are used to intercept debris carried by the wastewater flow, which is too large to pass through downstream pumps and channels. Rocks, boards, rags, and other large objects are captured by the screen and removed by rakes. The screened material readily dewater for disposal on site or at a landfill.

A bar screen is a vertical or sloped set of equally spaced steel bars that completely cross a channel. For manually cleaned screens, the bars are sloped 30 to 45° from the horizontal; flatter slopes increase the ease of cleaning. Steeper slopes are used for mechanically cleaned screens.

When the bar screen is simply intended to protect a raw wastewater pump, such as in a sewer collection system or at the influent to a treatment plant, the bar spacing is usually 2 to 6 in (50 to 150 mm). At the treatment plant, more protection of the equipment is required, and the space between bars is typically 1 to 2 in (25 to 50 mm). State agency design criteria and spacing requirements of mechanical cleaning equipment must be considered.

The design of bar screens includes consideration of channel design. Under normal flow conditions, the

approach velocity should be between 2 and 4 ft/s (60 and 120 cm/s). This prevents solids from settling in the channel and from being flushed through the screen.

Other design considerations include

1. The screen should be cleaned frequently enough to prevent ponding and significant hydraulic head loss. Thus, except in small domestic systems or some industrial applications, mechanically cleaned bar screens are preferred for both design and operational reasons.
2. Two or more channels should be used with provisions made to dewater individual screening units for maintenance and repair.
3. When multiple screens are planned for simultaneous use, the entrance channel should be designed to divert proportional flow to each unit.
4. Adequate provisions should be made for the collection and disposal of the screenings. In some applications, it may be desirable to macerate the screenings and return them to the wastewater flow. This eliminates the need for separate disposal of screenings but requires that these solids be removed again from the wastewater by subsequent processes.
5. The channel behind the unit should be lowered 3 to 6 in (75 to 150 mm) to compensate for head loss through the bar screen.

The mechanically cleaned bar screens are equipped with rakes that are periodically pulled through the screen spacings by endless pullers. The screenings are deposited at the top of the unit and conveyed to a depository for later disposal. Both front- and back-cleaning screening are available.

Comminutors. When very large solids are not expected to enter the wastewater stream and other conditions favor retaining screenable solids in the wastewater, comminutors may be utilized in lieu of bar screens. These devices cut the larger objects into ones of a size 0.25 to 0.75 in (6 to 19 mm), which will not be detrimental to downstream plant operations. Some of the different types of cutting mechanisms are presented in Table 6.33.

In addition to review of manufacturers' data, the basis of comminutor design should consider

- The size of the largest particle that should be allowed to pass through the unit, considering downstream piping and equipment, must be determined.
- The characteristics of the solid particles, especially of industrial origin, should be considered, along with selection and reliability of cutting mechanisms.
- The unit should have adequate capacity to handle peak flow conditions.
- A bypass bar screen should be provided and provisions made to isolate the comminutor for maintenance and repair.
- If possible, the comminutor should be downstream of a grit chamber or, at least, have a trap to collect rocks and gravel-type material ahead of the unit and, thus, avoid excessive wear of the equipment.
- In some applications, a coarse bar screen, as used before pumps, should be located upstream to protect the mechanical equipment.

TABLE 6.33 Comminutor Cutting Mechanisms (16)

-
1. Coarse material is cut by cutting teeth and shear bars on a revolving drum that passes through a stationary cutting comb.
 2. Cutters on an oscillating arm contact fixed cutters on the semicircular drum.
 3. Stationary grid intercepts larger solids while smaller solids pass through to cutting teeth mounted on rotating disks. Cutting teeth are also mounted on the combs of the grid.
 4. Rotating cutters travel up and down a bar screen, cutting retained solids and permitting entry.
-

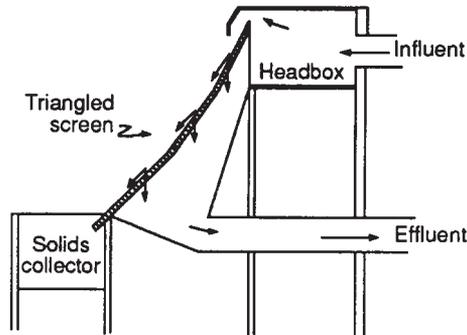


FIGURE 6.28 Static screen schematic.

Wedge Wire Screens. Using wedge wire as the screen medium, static and rotary screens are available for coarse as well as fine screening of wastewater. The units may be used by an industry for pretreatment or at a treatment plant for removal of large or fine solids. The fundamentals of screening using static and rotary screens are illustrated in Figures 6.28 and 6.29, respectively.

Design data for the coarse screening of wastewater by wedge wire screens are presented in Table 6.34. For applications requiring removal of finer particles, screens are available with smaller openings; the other design data change as a result of this modification.

Traveling Screen. Using stainless steel or another noncorrosive material as the medium, traveling screens are used mostly for coarse screening. The medium forms an endless belt, which may extend essentially perpendicular to the wastewater flow. Much like the rakes on mechanically cleaned bar screens, trays collect and remove solids captured by the screen. Depending on the space openings, traveling screens may be considered for influent screening or effluent polishing.

Vibrating Screens. Circular, rectangular, or square vibrating screens are commercially available for coarse or fine screening of wastewaters. Normal applications are for pretreatment, and associated by-

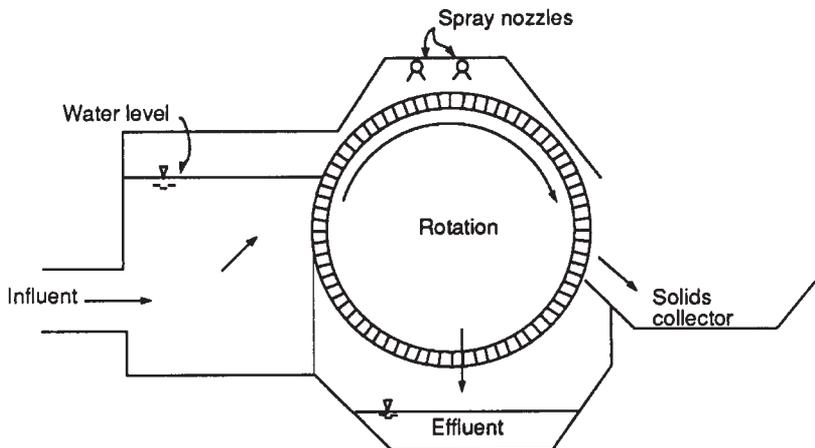


FIGURE 6.29 Rotary screen schematic.

TABLE 6.34 Wedge Wire Screen Design Data (49, 50)

Parameter	Static screen	Rotary screen
Suspended solids removal, %	5–25	5–25
Loading rate	4 to 16 gal/min/in (860 to 3400 m ³ /day/m ²) of screen width	15 to 112 gal/min/ft ² (880–6575 m ³ /day/m ²) of screen area
Screen opening	0.01 to 0.06 in (0.25 to 1.5 mm)	0.01 to 0.06 in (0.25 to 1.5 mm)
Head loss	4–7 ft (1.2–2.1 m)	2.5–4.5 ft (0.8–1.4 m)
Motor size	—	0.5 to 3 hp (0.4–2.2 kW)
Solids by weight, %	12–15	16–25

product recovery, by food and other industries. The wastewater should contain little or no grease to make vibrating screens an alternative to the static and rotary wedge wire screens. Typically, the solids collected by vibrating screens will have a lower moisture content than from other screening methods.

Fine Screens. To achieve removal of fine solids, the principles of static and rotary screens apply. The screen material may be stainless steel, fabric, or other medium with openings of only 0.01 to 0.06 mm.

The rotary microscreen (Figure 6.30) can be used for the removal of residual suspended solids from a biological treatment process. Typical design parameters are shown in Table 6.35. Also, to improve effectiveness, it may be necessary to provide flow equalization to ensure a relatively constant quality and quantity of wastewater to the microscreen. Pilot plant testing should be a part of the design considerations.

Grit Chambers

Grit chambers are utilized typically after bar screens to remove sand, rocks, and other heavy material from the wastewater. Gravity grit chambers allow quiescent settling of grit, and aerated grit chambers impart a rolling action to the wastewater, which allows the heavier grit material to settle out while the lighter organics

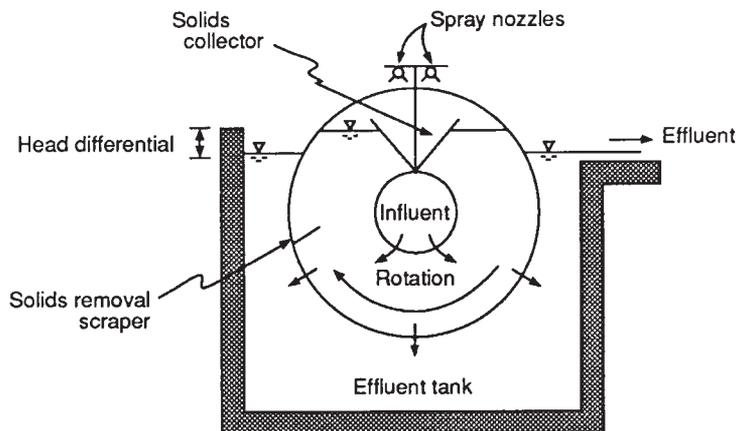
**FIGURE 6.30** Microscreen schematic.

TABLE 6.35 Microscreen Design Data (16, 49, 50)

Parameter	Typical value	Remarks
Suspended solids removal, %	50–70	Also reduces BOD
Submergence	75% of height, 66% of area	Range, 70–80%
Screen openings	20–25 μm	Range, 15–60 μm
Loading rate	5–10 gal/min/ft ² (295 to 585 m ³ /day/m ²) of submerged surface area	Higher for smaller screen openings
Head loss	3–6 in (75–150 mm)	Typical to provide overflow weir for partial bypass when head exceeds 6 to 8 in (150 to 225 mm)
Typical drum diameter	10 ft (3 m)	Wider drums increase backwash requirements
Drum speed	0.5–4.5 r/min	Directly proportional to head loss
Backwash flow, % of total	2–6	Continuous. Inversely proportional to pressure, which may range from 15 to 80 lb/in ² (100–550 kN/m ²)

remain in suspension. The extracted grit is composed of mostly inorganic material that readily dewater and normally requires no additional treatment before disposal in landfills.

Grit washing equipment may be necessary if excessive quantities of heavy organic particles are expected to be collected and/or the collected grit is not removed from the site frequently. This provision is mostly for health protection and odor (aesthetic) control.

Gravity Grit Chambers. When space permits, manually or mechanically cleaned gravity grit chambers can be used. The chamber is typically rectangular and the depth is fairly shallow. An inlet velocity control is necessary to provide a uniform cross-sectional velocity in the grit chamber. A Parshall flume is often used to provide the inlet control as well as measure plant flow.

Typical design parameters are controlled velocities near 1 ft/s (0.3 m/s) and detention times of about 60 s. The length and depth of the channel are governed by the settling velocity of the minimum-sized particles to be removed and by the selected storage volume. Duplicate units should be provided with provisions to isolate and dewater individual units for maintenance and repair.

The physical features of a (mechanically cleaned) gravity grit chamber are illustrated in Figure 6.31. The uniform proportioning (splitting) of flow and outlet flow controls are important features. The grit collection sump may be located near the inlet or outlet depending on the design of a specific unit.

Manual cleaning of a grit chamber using a shovel or mechanical equipment is not common except for smaller treatment works. However, manual cleaning of a grit collection basin for inorganic wastewater treatment may be practical. In such a case, one approach is to have a sloped entrance to the basin so that a front-end loader can actually enter the unit to remove the settled solids. This type of system should have sufficient storage capacity to minimize the frequency of cleaning.

Most conventional grit removal is performed by mechanical collectors equipped with bottom scrapers to move the grit to a collection sump. Bucket or screw conveyors are typically used to lift the grit from the collection sump to a container for transport to the point of ultimate disposal (burial).

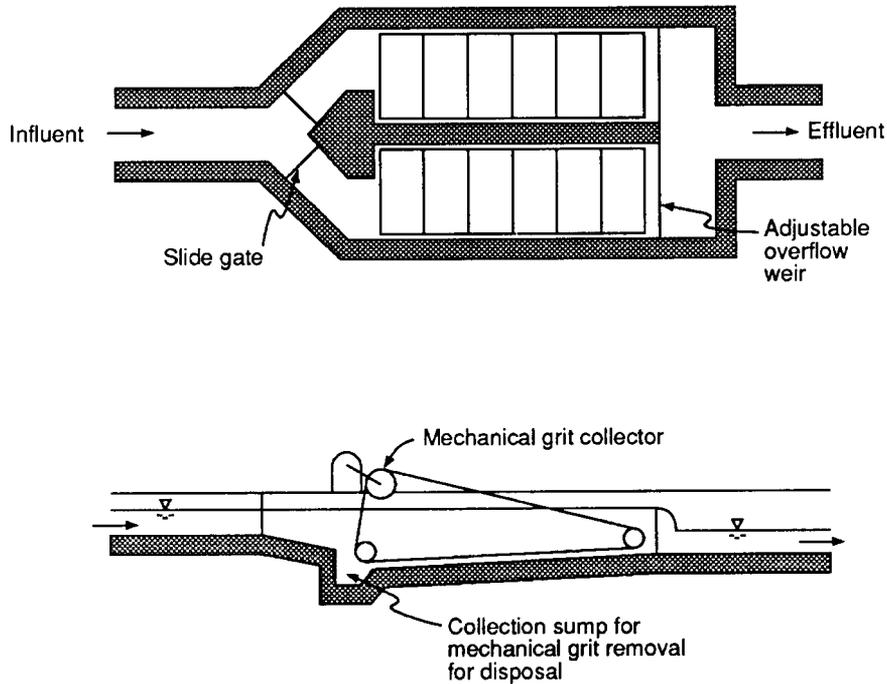


FIGURE 6.31 Gravity grit chamber schematic.

Aerated Grit Chambers. By providing air to impart a spiral flow pattern, grit can be removed in less space and with less organic content. Typical design data for aerated grit chambers are presented in Table 6.36.

The geometry of the chamber is important not only to avoid “dead areas” but also to ensure effective rolling action and to accommodate the selected grit collection equipment, which may be pumps or bucket or other types of conveyors. As with gravity chambers, the design should provide for multiple units and for isolation and dewatering of individual chambers for maintenance and repair.

TABLE 6.36 Aerated Grit Chamber Design Data (4, 16)

Parameter	Typical Value	Remarks
Detention time	3 min	Range, 1–5 min at peak flow
Air supply	3 ft ³ /min/ft (0.3 m ³ /min/m) of length	Range, (1–8 ft ³ /min/ft) (0.1–0.75 m ³ /min/m) of length
Height of air diffusers above tank bottom	2 ft (0.6 m)	Range, 1.5 to 3 ft (0.45 to 0.9 m) depending on grit collection equipment
Length-to-width ratio	2:5 or 5:1	With transverse baffle
Width-to-depth ratio	2:1 or 5:1	With longitudinal baffle

Sedimentation

Sedimentation processes for removal of particulate and colloidal solids and flocculent suspensions are an integral part of almost every wastewater treatment plant. This section deals with the process selection and the basis of design for suspended solids removal systems. Ancillary equipment is also discussed.

The principal design considerations are the settling basin's cross-sectional area, detention time, depth, and overflow rate. In addition, the efficiency of the settling process is affected by the wastewater and suspended solids characteristics, number of basins, variation in flow, and general operation and maintenance practice.

Sedimentation Processes. Depending on the characteristics of the suspended particles and the concentration of the suspension, the sedimentation process is described by four classifications:

1. Class I—Particles
2. Class II—Particles and suspensions
3. Class III—Suspensions
4. Class IV—Thickening

The segregation of these types of sedimentation is shown in Figure 6.32.

Class I Sedimentation. The settling of a dilute concentration of discrete suspended particles which do not tend to flocculate is termed Class I sedimentation. This process is applicable to design of some industrial

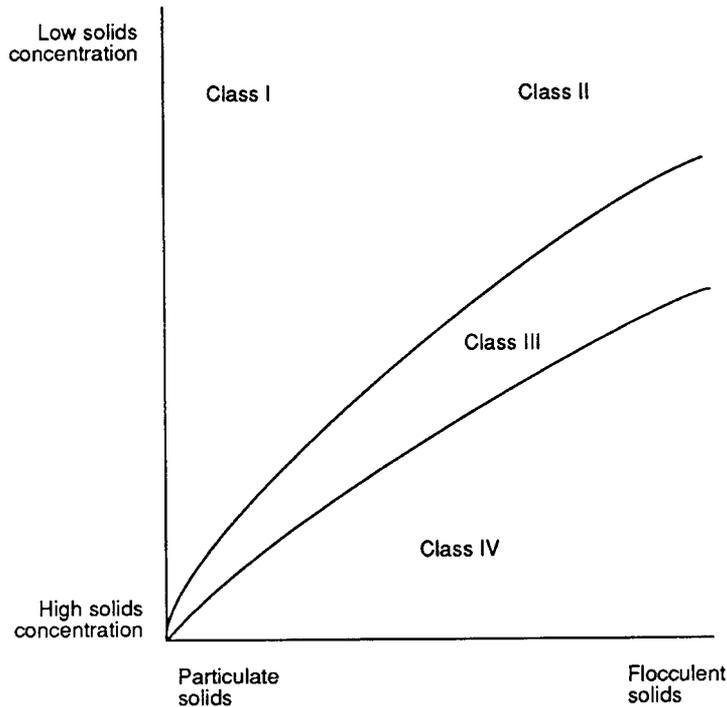


FIGURE 6.32 Sedimentation classifications.

wastewater treatment systems where the settling of heavy inert materials is influenced only by the characteristics of the individual particles.

The settling velocity is defined as the distance a particle will settle with time, and this value may be used to define a design overflow rate for a given settling tank depth. The overall settling velocity will be a function of the weights and configurations of particles in a given wastewater stream.

Settling column tests can be used to analyze the removal of suspended solids with respect to particle velocities.

Class II Sedimentation. Most industrial and domestic wastewaters contain a broad range of particles varying in size and surface characteristics. These types of suspended solids (common to the influent of a primary settling basin) and flocculating suspensions are part of Class II sedimentation.

In this process, the heavier particles will settle fastest and in the process may join together with lighter particles to form yet heavier and larger particles as the mass settles. Thus, the greater the depth of the settling basin, the greater the efficiency of the sedimentation process. This dependence on depth is the principal difference between Class I and Class II sedimentation.

Settling column tests are used to predict the performance of a settling basin under ideal (quiescent) conditions. The wastewater to be examined is placed in a column equipped with sampling ports. The column diameter should be great enough to minimize wall interferences with the settling process.

By knowing the initial suspended solids concentration and by collecting and analyzing samples from the ports over time, a plot of equal concentration can be prepared to illustrate the settling path for different removal rates. Results from a hypothetical test are present in Figure 6.33. This shows that a depth of d_i and a

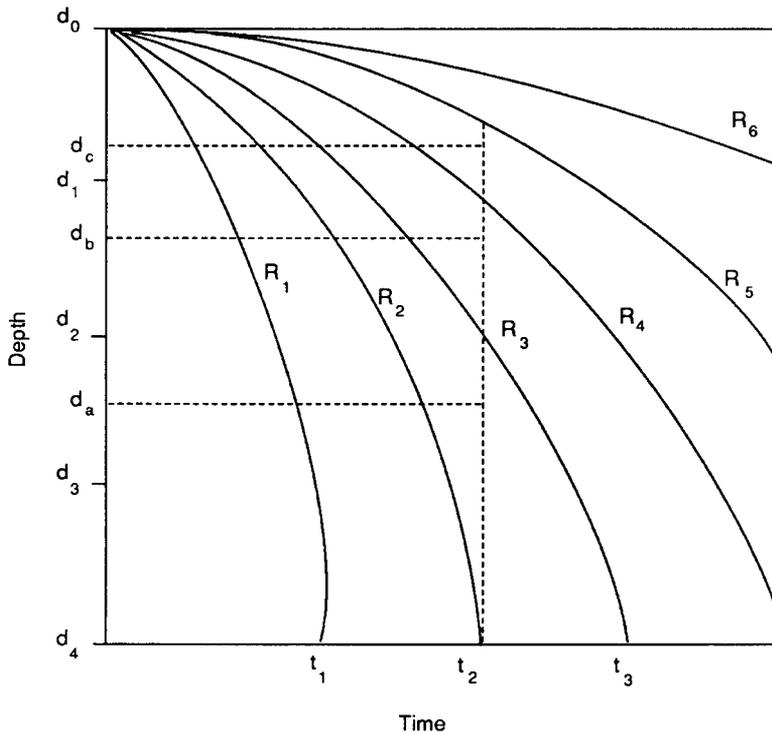


FIGURE 6.33 Class II sedimentation test.

time of t_n are required to remove the given percentage of suspended solids R_x . The average velocity of the removed solids (overflow rate) will be less than d_i/t_n .

Using the notation in Figure 6.33, the overall removal R for a depth of d_4 can be calculated as

$$R = R_2 + \frac{d_a}{d_4}(R_3 - R_2) + \frac{d_b}{d_4}(R_4 - R_3) + \frac{d_c}{d_4}(R_5 - R_4) + \dots$$

All R values are in percent. The d_a , d_b , and d_c depths are midway between the respective R lines at time t_2 . In practice, there will be little value in considering proportional removals from the higher percent removal ranges when the depth ratio becomes insignificant.

Class III Sedimentation. The settling of relatively high concentrations of suspended solids is referred to as Class III sedimentation or zone settling. In this process, the solids settle as a mass, producing a distinct interface between the solids and clarified water zone. Activated sludge and flocculated suspensions exhibit these characteristics.

More precisely, the settling solids are in three zones as shown by the settling column test in Figure 6.34. The upper zone is called hindered settling since the upward displacement of liquid "hinders" the settling ve-

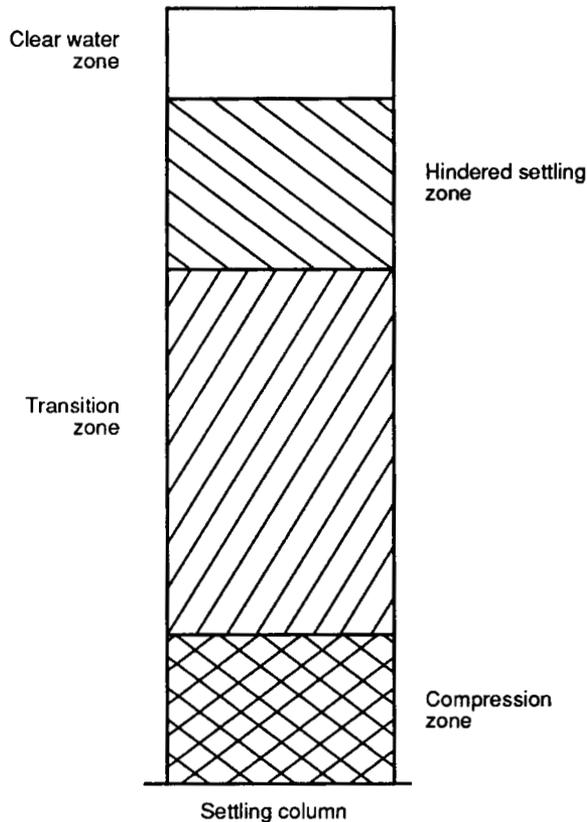


FIGURE 6.34 Class III sedimentation zones.

locity from being as great as it would be for discrete particles. The compression zone is where the solids are mutually supported.

The subsidence of the clear water-suspension interface is important in design of Class III sedimentation systems and may be determined by timing the movement in a graduated settling column. As illustrated in Figure 6.35, the subsidence rate, or velocity, during hindered settling is important in settling basin design; this is the slope of that portion of the curve. Also, the surface area of the settling basin must be large enough so that the liquid rise, or overflow, rate is less than the subsidence rate. Thus, the required surface area equals the overflow rate divided by the subsidence rate of hindered settling.

Class IV Sedimentation. The fourth class of sedimentation is used to describe the thickening or compression of the sludges from the other three sedimentation classifications. In this process, the particles are in physical contact with each other, thus reducing the rate of subsidence. Compaction may be increased by gentle agitation.

The thickening process is especially important when dealing with sludges of high solids concentrations, such as produced in Class III sedimentation. The cross-sectional area required for thickening may exceed that for suspended solids removal. A settling column test should be used to make this determination.

The area necessary for thickening is a function of the desired settled solids/sludge concentration C_d . This concentration requires a certain period of time t_d to occur; thus time and the associated depth of the column d and flow Q determine the required area A of the settling zone.

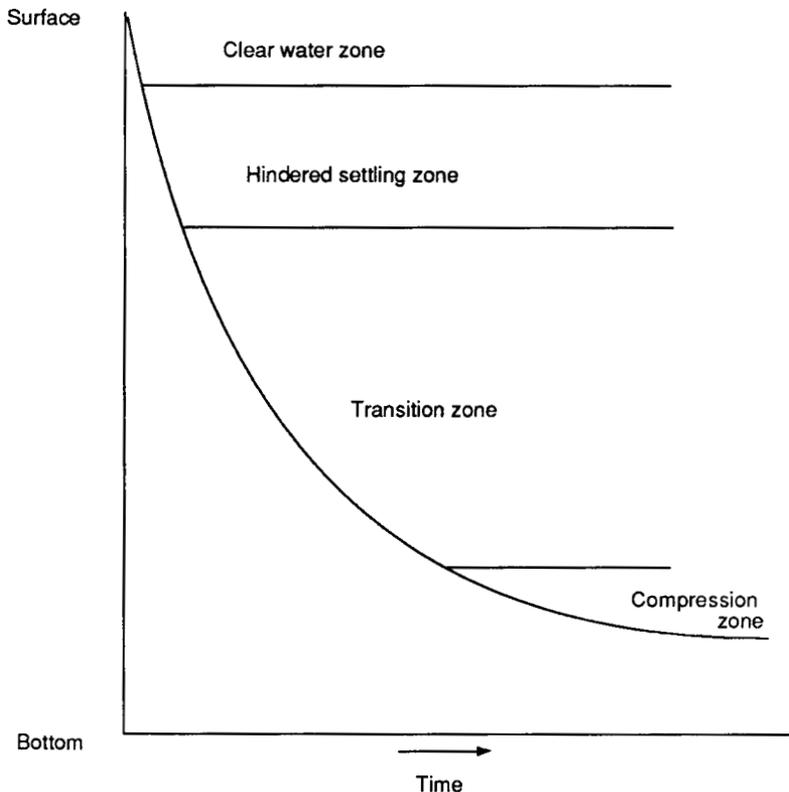


FIGURE 6.35 Class III sedimentation curve.

$$A = \frac{Qt_d}{d}$$

Since the wastewater flow rate and depth are known, the time for concentration must be determined. This is done graphically (see Figure 6.36) as follows:

1. Draw a line tangent to the hindered settling and compression portions of the curve.
2. Bisect the intersecting tangents and draw a line to the curve. This point on the settling curve is representative of the most critical concentration C_c in that to achieve this concentration would require the maximum area for any flow rate.
3. Extend a horizontal line through the point representing the desired sludge concentration C_d .
4. Project a line tangent to C_c to intersect with the horizontal line from C_d . The point of intersection establishes the amount of time t_d required for thickening.

The area required for thickening may now be calculated.

Scale-up. When applying the settling tests to an actual design, scale-up factors must be applied to account for departure from essentially optimum settling conditions. The full-scale settling basin will be sub-

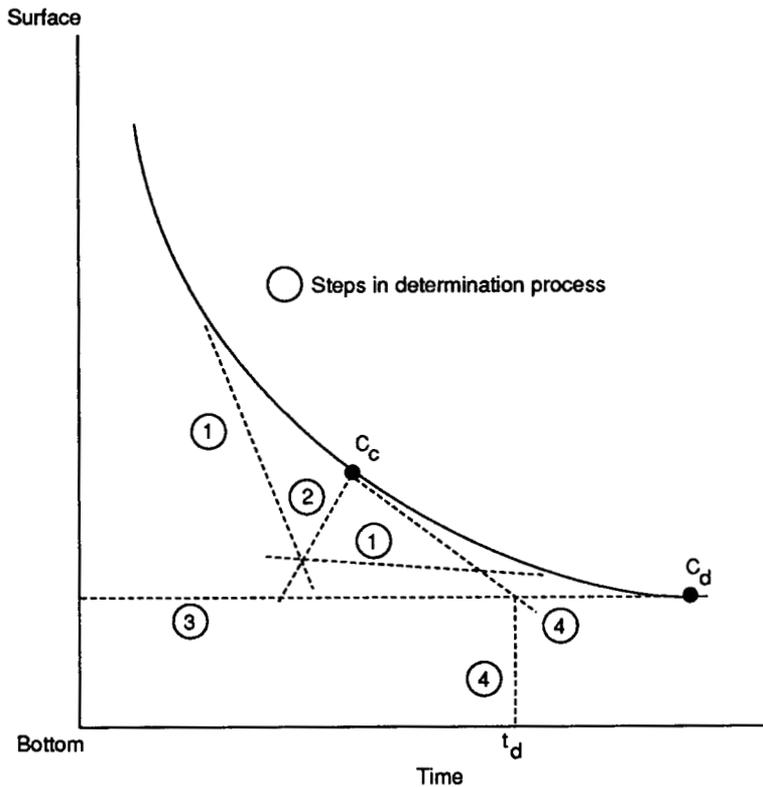


FIGURE 6.36 Thickening capacity determination.

ject to variations in quantity and turbulence of flow; inlet and outlet features; physical factors, such as wind and temperature; and requirements for sludge storage. Also, the scale-up factor tends to slightly decrease as a function of increasing test unit size.

To achieve effective scale-up, the full-scale settling basin will have a larger area, larger volume, and smaller overflow rate relationship than the test unit. Table 6.37 provides order-of-magnitude factors for scale-up; the previously mentioned influences as well as the specific wastewater's characteristics and the desired performance should be used to judge the best actual scale-up factor. Site conditions and commercially available sludge collection equipment will also influence the size of the full-scale settling basin.

Design Considerations. Settling basin designs must provide for

- Effective removal of suspended solids from the wastewater flow
- Collection and removal of settled solids (sludge) from the basin

In most applications, thickening or compression of the sludge is important.

The efficiency of the settling basin is directly related to the occurrence of short circuiting, and the removal of biological and chemical flocs is most significantly affected by short-circuiting. Thus, basin design should consider the following factors which cause or permit short-circuiting:

- High inlet and outlet velocities
- Turbulent flow
- Wind stresses
- Temperature gradients
- Mass density differences
- Basin geometry

Moreover, the four principal structural designs for settling basins are for the

1. *Inlet*: The influent flow should be evenly distributed with minimal inlet velocities to avoid turbulence and short-circuiting.
2. *Settling zone*: Quiescent conditions are needed for effective particle and suspension settling. Measures may be required to reduce wind, density, and temperature effects and to control flow-through velocities.
3. *Sludge zone*: Sufficient depth should be provided for sludge storage and to permit desired thickening.
4. *Outlet*: The effluent velocities should be minimized by limited weir loadings and proper weir placement.

Since design conditions are significantly affected by the use of chemicals, separate discussions follow for simple gravity sedimentation of Class II and III solids common to most wastewaters. This is termed *plain*

TABLE 6.37 Full-Scale to Test-Scale Factors

Sedimentation classification	Area	Volume	Overflow rate
I—Particles	1–1.25	1.25 ±	0.75–1
II—Particles and suspensions	1.25–2	1.75–2	0.5–0.75
III—Suspensions	1–2	1.25–2	0.5–1
IV—Thickening	1–1.25	1.25–1.5	0.75–1

sedimentation. *Chemical sedimentation* refers to settling influenced by chemical coagulant additions and conditions of Class III sedimentation.

Furthermore, there are two fundamental settling basin classifications: (1) horizontal-flow basins, which are circular or rectangular units with a flow pattern across the basin, and (2) vertical-flow basins, which are annular or rectangular units with an upward flow pattern from bottom to top. Both types are used for plain and chemical sedimentation; however, the vertical-flow basins should receive special consideration for chemical flocs and for biological flocs when the design overflow rates are low, e.g., less than 500 gpd/ft² (20 m³/day/m²). The horizontal- and vertical-flow basins are most widely used in the United States and Europe, respectively.

More specific design considerations for plain and chemical sedimentation are presented in the following sections.

Plain Sedimentation. Circular and rectangular horizontal-flow settling basins are most widely used for plain (without chemical conditioning) sedimentation.

Circular Settling Basins. The principal features of a circular settling basin are illustrated in Figure 6.37. Circular units can be used for a wide range of suspended solids removal applications; their configuration is also well suited for sludge thickening.

Rectangular Settling Basins. The use of rectangular settling basins is widespread. However, as the length increases and/or solids loadings increase, the application of rectangular units becomes limited by their ability to adequately remove sludge. (Figure 6.38 shows the physical features of a rectangular settling basin.)

Dimensions. The dimensions of the settling basin are important considerations to ensure proper inlet and outlet designs and effective settling conditions in the basin. Table 6.38 summarizes typical dimensional information for circular and rectangular settling basins.

Inlet Design. The inlet to the settling basin should

- Dissipate inlet velocities
- Equally distribute flow horizontally and vertically
- Prevent short-circuiting and turbulence

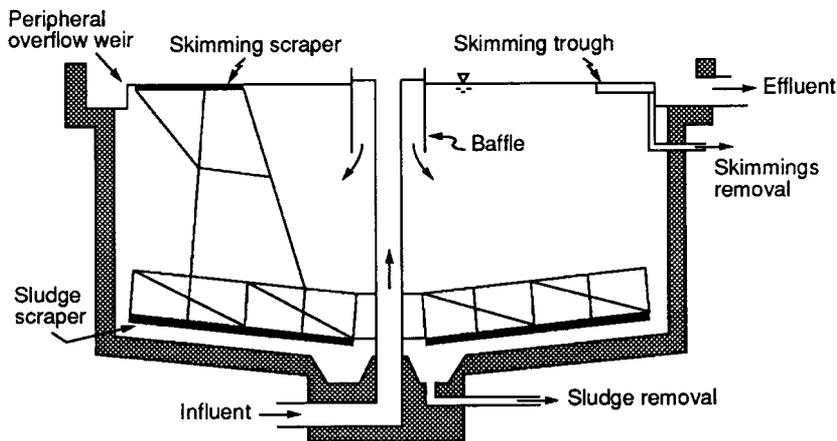


FIGURE 6.37 Circular settling basin.

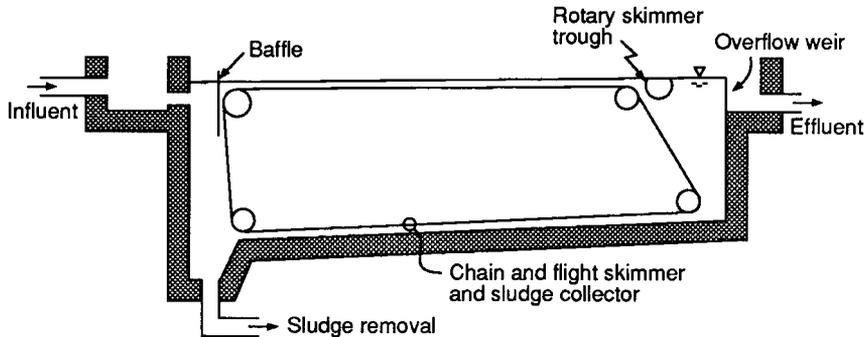


FIGURE 6.38 Rectangular settling basin.

The distance between the basin's inlet and outlet should be at least 10 ft (3 m) unless special measures are taken to prevent short-circuiting. As the distance between the inlet and outlet increases, the inlet design becomes less critical to the basin's performance.

For circular basins, the inlet flow may be either center or peripheral feed. Design of the center feed units must consider uniform radial distribution of the flow. The inlet velocities should be high enough to maintain particle suspension, 1 to 2 ft/s (0.3 to 0.6 m/s), at design flow, but small enough not to produce excessive inlet energy, 3 ft/s (1 m/s). The controlled discharge velocity should be less than 0.25 ft/s (0.08 m/s). For peripheral feed, the inlet velocity should be maintained to avoid settling of solids before the flow is distributed through orifices or other types of openings around the entire circumference.

The inlet to a rectangular basin is normally distributed across the entire width of the basin by means of equally spaced ports. The inlet channel design should maintain solid suspensions and be proportioned to provide equal flow to each inlet port. Velocities through the inlet ports range from 0.25 to 0.5 ft/s (0.08 to 0.16 m/s).

Baffles are used with both circular and rectangular tank inlets to improve the distribution of flow. Velocities are further dissipated, and thus the possibilities of short-circuiting are reduced.

In circular settling basins with center feed, the inlet well should not be less than 20% of the basin diameter and should have a depth of 55 to 65% of the basin. Baffles in rectangular settling basins are usually installed 2 to 3 ft (0.6 to 1 m) in front of the inlet ports, submerged 1.5 to 2 ft (0.45 to 0.6 m), with the top off the baffle 2 in (5 cm) below the water surface.

TABLE 6.38 Settling Basin Dimensions

Parameter	Typical Value	Remarks
Depth	10–15 ft (3–4.5 m)	Range, 7–15 ft (2.4–4.5 m)
Circular basin		
Diameter	40–150 ft (12–45 m)	Range, 10–200 ft (3–60 m)
Rectangular basin		
Length	80–120 ft (25–36 m)	Range, Up to 300 ft (91 m)
Width	10–20 ft (3–6 m)	Multiple tanks for greater width
Length-to-width ratio	3:1	Minimum
Width-to-depth ratio	1:1	Up to 2.25:1

Outlet Design. The design of settling basin outlets focuses on the overflow or effluent weiring system. The weir (usually V-notch) should be vertically adjustable to control detention time and minimize short-circuiting. Baffles are used to prevent the overflow of floating solids; skimming troughs or mechanical skimmers are thus required.

The principal outlet design criterion is the weir loading rate, which is defined by the flow per length of weir, in gallons per day per foot (cubic meters per day per meter). The weir must be sufficiently long that the loading rate will not cause updrafts or other turbulences in the settling basin. Typical values for weir loading rates of 10,000 to 15,000 gpd/ft (125 to 185 m³/day/m) are used by regulatory agencies. Higher rates may be satisfactory for larger basins and specific suspended solids. Class III solids are most (negatively) sensitive to increased weir loading rates.

Depth. The depth of the settling basin defines the volume associated with the required area for settling and thus is used to determine the detention time of the wastewater flow. The basin depth should be sufficient to prevent scouring velocities along the bottom of the basin and uplifting of solids at the outlet end of the basin. Also, the design volume must include storage for settled solids.

Typically, settling basin depths range from 8 to 15 ft (2.4 to 4.5 m), and detention times range from 2 to 4 h. Both parameters are a function of the suspended solids characteristics and degree of removal required.

Overflow Rate. Overflow rate is a critical parameter for settling basin design. It is defined as the settling velocity of the particles and/or suspensions to be removed and is expressed as a flow rate per unit area, gallons per day per square foot (cubic meters per day per square meter). Techniques for determining the settling velocity of Class II and Class III solids were presented earlier in this section. Table 6.39 shows typical design overflow rates for different sedimentation classifications.

Solids Loading Rate. The solids loading rate is defined as the quantity of solids per day per unit area, pounds per day per square foot (kilograms per day per square meter) and is derived from the sludge-thickening characteristics (see Class IV sedimentation). For activated sludge systems, the typical solids loading rates have upper ranges of 20 to 35 lb/day/ft² (4 to 7 kg/day/m²).

Solids Collection. Collection of skimmings and sludge is accomplished by scraper or suction units. Scraper units are best suited for settled Class I and II solids, and suction units are used frequently for settled Class III suspensions. The removal of large quantities of sludge and a faster sludge removal rate are best achieved with circular settling basins equipped with suction units.

In circular settling basins, the surface skimmer and sludge scraper arms (generally two) are driven from the center support column. A skimmings trough is provided at a point on the periphery. The sludge is scraped along the sloped bottom to a sludge collection and draw-off well. The tip speed of the scraper ranges from 2 to 15 ft/min (0.6 to 4.6 in/min) for the lower limit of Class III sludges and to the upper limit of Class II sludges.

Suction sludge removal units also are driven from the center of the basin, but sludge is removed along the slope of the bottom rather than from a collection well. A tapered header is used and is sized for a uniform velocity. Suction ports (orifices) are sized to remove sludge in proportion to the affected basin settling area.

Chain and flight (see Figure 6.38) and traveling bridge scrapers are used in rectangular settling basins for

TABLE 6.39 Settling Basin Design Overflow Rates

Sedimentation classification	Overflow Rate, gpd/ft ² (m ³ /day/m ²)	
	Typical value	Range
I—Particles	800–1000 (33–41)	200–4000 (8–163)
II—Particles and suspensions	600–1200 (24–49)	600–2000 (24–81)
III—Biological suspensions	200–800 (8–33)	200–800 (8–49)
III—Chemical suspensions	800–1000 (33–41)	400–1800 (16–73)

skimming and sludge collection. A typical application will use a horizontal scraper speed of 2 to 4 ft/min (0.6 to 1.2 in/min). Suction mechanisms may be added to traveling bridge units.

Tube Settlers. Designs for new, or modification of existing, settling basins may incorporate tube settlers for plain or chemical sedimentation. Manufacturers provide modular units with a minimum hydraulic radius and length of about 1 in (2.5 cm) and 24 in (61 cm), respectively. For gravity drainage of settled solids, the angle of the tube settlers should be 45 to 60°.

The flow pattern in the tube settler module is countercurrent with the liquid flow upward to an effluent overflow zone as the solids move down the tubes and then to a sludge collection zone beneath the tube settler. An example of this flow pattern in a modified, rectangular settling basin is shown in Figure 6.39.

The principal design considerations as well as the reasons for successful utilization of tube settlers are

1. The multiple tubes provide a significant increase in the effective settling area.
2. The detention time may be reduced to fractions of an hour rather than several hours.
3. The small tube openings promote and protect quiescent conditions, laminar flow, and uniform flow distribution.
4. The countercurrent flow pattern allows the settled solids to agglomerate (Class II sedimentation).

Conservative designs will incorporate conventional settling tank design criteria; however, manufacturers should be consulted for updates on performance data and design criteria on this relatively new technology in wastewater treatment.

Since tube settlers are believed to be especially applicable to chemical sedimentation, they have wider application in water treatment. In the wastewater treatment area, tube settlers are an alternative means to upgrade existing settling basins and to provide chemical sedimentation of effluents requiring further reductions of suspended solids.

Wedge Wire Settlers. In addition to use as a screening device, wedge wire has been successfully mounted in settling basins, as illustrated in Figure 6.40. The wire mesh, with openings of 0.125 to 0.25 mm, is mount-

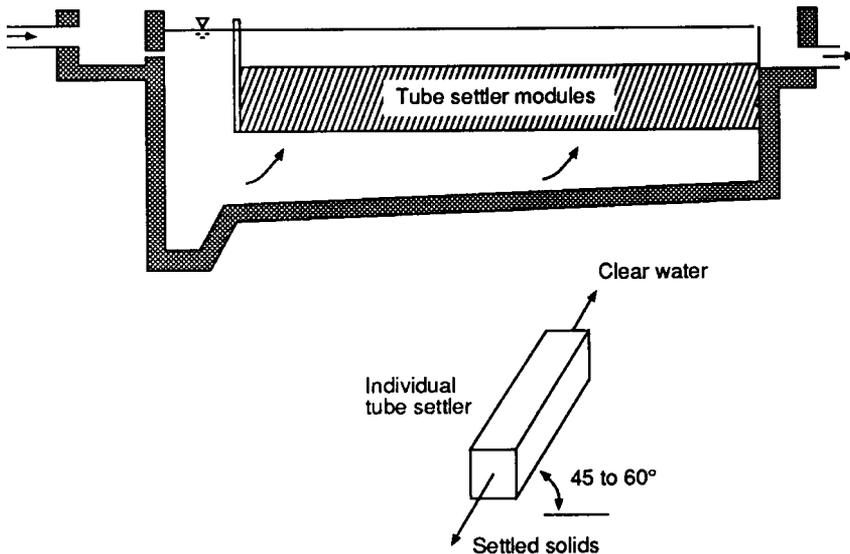


FIGURE 6.39 Tube settler installation.

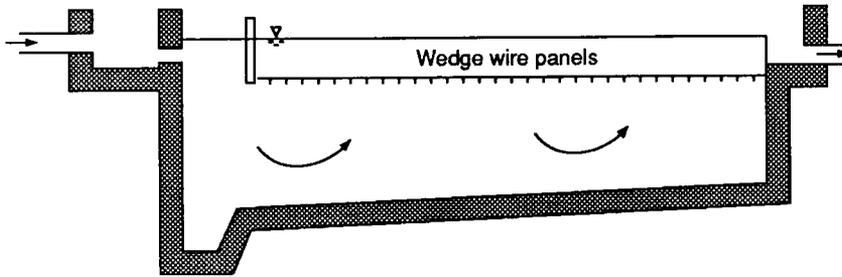


FIGURE 6.40 Wedge wire settler installation.

ed parallel to the water surface in a manner similar to tube settlers. Applications are best suited for upgrading the performance of existing settling basins for the removal of biological flocs or similar suspended solids.

As the flow moves upward through the wire mesh, larger, unsettled solids are blocked from the overflow. The approach velocity to the settler should be less than what would break up flocculant suspensions impacting the wire mesh.

A special feature of the wedge wire settler is that it effectively distributes the flow across the cross-sectional area of the basin. Thus, conditions are very favorable for the settling and agglomeration of the fine particles which passed through the wedge wire settler. This action improves the efficiency of the process but requires that the unit be taken out of service periodically (every several days) to lower the basin's water level to permit cleaning these settled (fine) solids from the settler.

Depending on the performance history of wedge wire settlers with regard to the plant's wastewater characteristics, pilot plant studies should be considered. The desired suspended solids removal may require a hydraulic loading of 50 to 100% of that for simple gravity settling. Thus, the settlers are most applicable to modifying settling basins in plants that are underloaded hydraulically but are not meeting effluent suspended solids limits.

Chemical Sedimentation. Chemical sedimentation describes the addition of chemicals to improve the settling of suspended solids. The process is quite similar to that of chemical treatment for (raw) drinking water supplies.

Chemical Treatment. The addition of chemical coagulants is an effective way to improve the settling characteristics of suspended solids. This process may also be a feasible alternative to improve the efficiency of existing and/or overloaded settling basins.

As in treatment of water supplies, a wide variety of polymers and other chemicals may be utilized for coagulation, i.e., formation of a floc, of wastewaters. Some of the more commonly used chemicals are alum, ferric chloride, ferric sulfate, ferrous chloride, ferrous sulfate, lime, and sodium aluminate. Coagulant aids, such as activated silica and bentonite clay, may be helpful. A comparison of advantages and disadvantages of the more common coagulants is presented in Table 6.40.

Predesign tests should be performed to identify the best chemical (and coagulant aid) for treatment of a specific wastewater. Jar tests and/or zeta potentials are generally used to determine the best coagulant and its dosage requirements. Settling columns are used to define the settling characteristics of the floc and, thus, the criteria for settling basin design. The settling process is Class III sedimentation.

Sludge production in chemical sedimentation includes gravity-settled suspended solids (plain sedimentation) plus increased agglomeration of suspended solids and formation of insoluble reaction products, both caused by the coagulant. The quantity of sludge may be estimated using jar test data.

TABLE 6.40 Comparison of Common Coagulants (51)

Coagulant	Advantage	Disadvantage
Alum	Easy to handle and apply; most commonly used; produces less sludge than lime; most effective between pH 6.8–7.5	Adds dissolved solids (salts) to water; effective over a limited pH range
Ferric chloride	Effective between pH 4–11; also makes sludge dewatering easier	Adds dissolved solids (salts) to water
Ferric sulfate	Effective between pH 4–6 and 8.8–9.2	Adds dissolved solids (salts) to water; usually need to add alkalinity
Ferrous sulfate	Not as pH-sensitive as lime	Adds dissolved solids (salts) to water; usually need to add alkalinity
Lime	Commonly used, very effective; may not increase TDS; sludge dewateres easily	Very pH dependent; produces large quantities of sludges; overdose can result in poor effluent quality
Polymer	Small dosage usually needed; easy to handle and feed	Improper dose results in poor floc formation

The coagulants may be delivered to the wastewater treatment plant in a liquid or dry form. Required plant features for feeding a dry form include a storage area for the concentrated form, a solution mix tank for preparation of the feed concentration, a solution storage tank, and a metering system to ensure the proper dosage. Often, the liquid coagulations may be fed directly from a storage tank by means of a metering system.

In both the water supply and wastewater treatment plant flow path, a rapid-mix tank is used to ensure a thorough dispersion of the coagulant for a more complete reaction with the suspended solids. Next, flocculation (a gentle mixing process) is required for the agglomeration of the coagulants and suspended solids. Finally, the treated wastewater is discharged to a settling basin.

Solids Contact Settling Basins. The horizontal-flow settling basins, previously discussed for plain sedimentation, have application for chemical sedimentation. However, this section focuses on the vertical-flow or solids contact settling basin, which combines mixing, flocculation, and sedimentation in a single unit. With this system, a large volume of “old” floc remains in the system, which increases the rate of “new” floc formation.

There are two general types of solids contact settling basins. The slurry recirculation type with a rectangular design maintains the high floc concentration by means of recirculation. This is illustrated in Figure 6.41.

The sludge blanket type of solids contact settling basin is shown in Figure 6.42. In this unit, the wastewater flow is upward through a suspended blanket of sludge. The sludge blanket serves as a screen and impedes the upward flow of larger particles so that they settle more quickly.

The retention of floc limits the application of the solids contact process if there are sufficient biological solids present to cause anaerobic or septic conditions to develop and upset the system. A wide operating range of hydraulic and solids loadings may also adversely affect performance of the system. Thus, solids contact units are most applicable to water supply treatment, inorganic wastewater treatment, and advanced wastewater treatment.

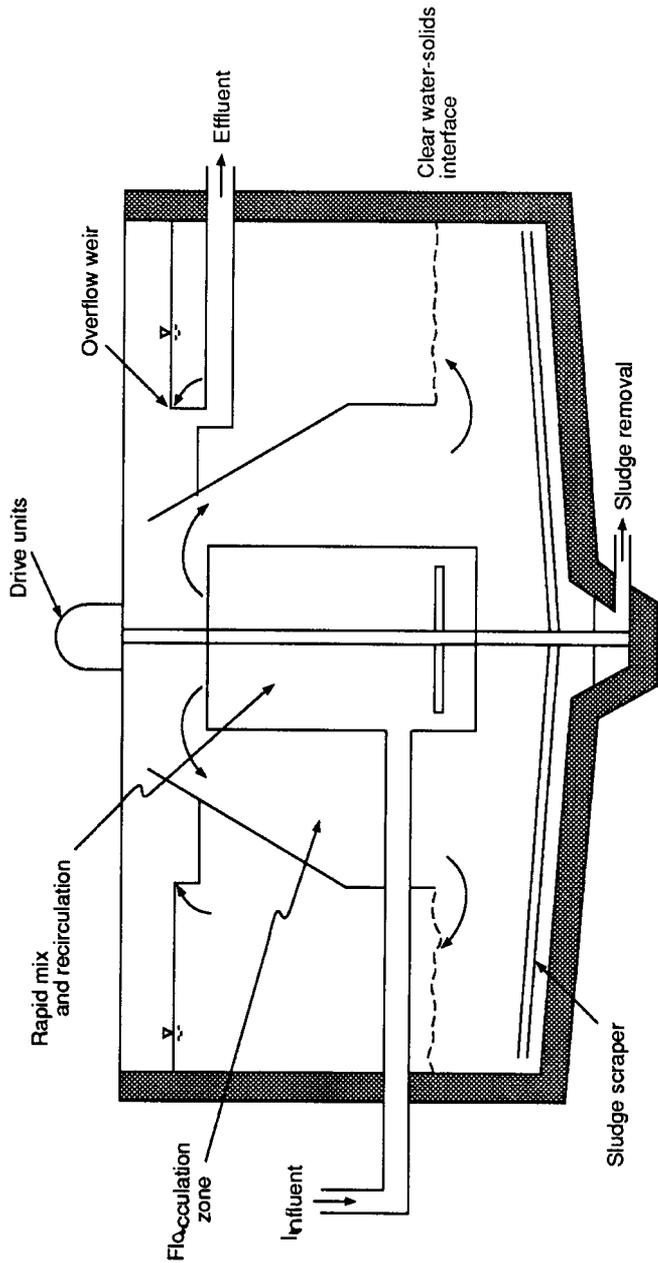


FIGURE 6.41 Solids contact settling basin—slurry recirculation type.

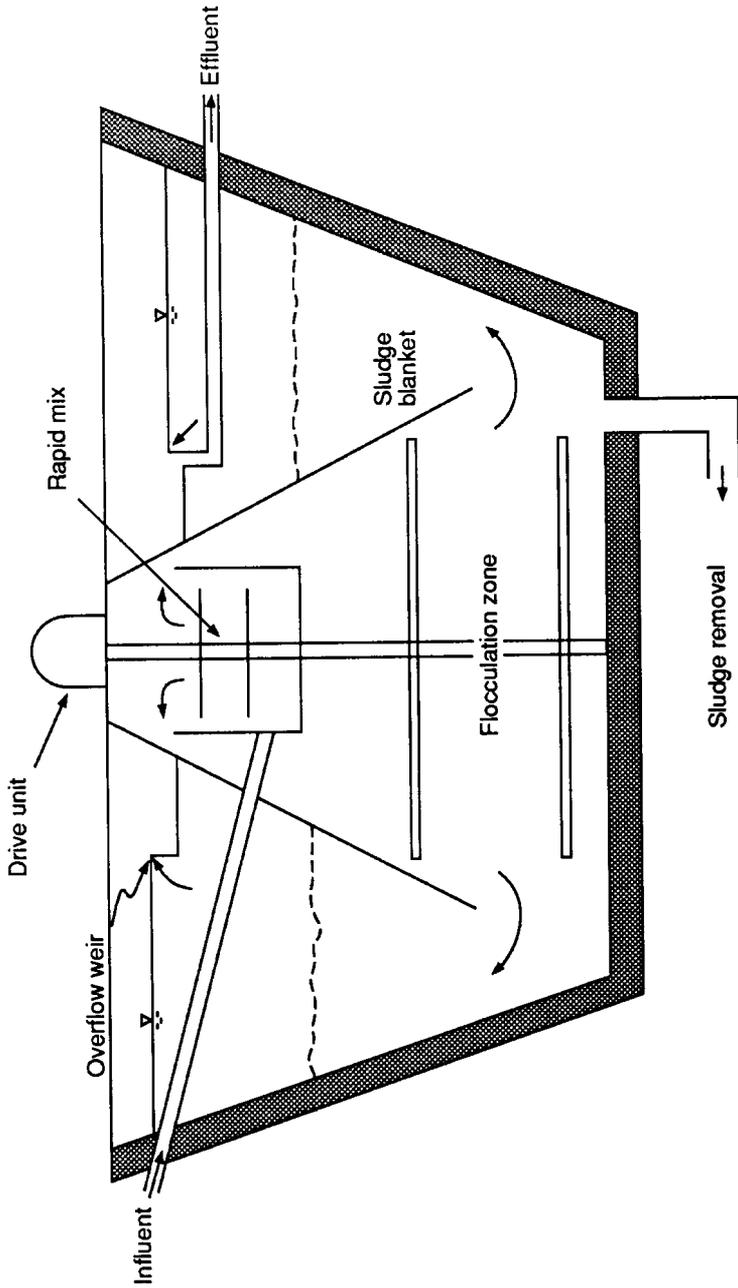


FIGURE 6.42 Solids contact settling basin—sludge blanket type.

The design of solids contact settling basins must be developed for a specific application after predesign studies and tests. Performance data on existing systems should be evaluated during the study phase. Equipment manufacturers will be a major reference source.

Flotation

When fine gas bubbles are introduced into wastewater, the bubbles will attach to the suspended solids, reducing their specific gravity and, thus, permitting their rise to the surface where the air-entrained solids may be collected and removed.

Flotation processes have numerous wastewater treatment applications, including

1. Removal of solids that have poor gravity settling characteristics
2. Replacement of more space-consuming alternative suspended solids treatment units
3. Industrial processes and pretreatment for reduction of pollutants and/or by-product recovery
4. Removal of a wide variety of materials categorized as oil and/or grease
5. Special applications, such as treatment of peak loads from industrial operations or from stormwater flows
6. Concentration of wastewater treatment sludges prior to dewatering processes

Usually, chemicals are used to enhance the adhesion, entrapment, or sorption of gas bubbles by the suspended or flocculant solids. Polymer, alum, ferric chloride, and other chemicals are used in this process. Pilot or bench scale testing should be performed to determine if chemicals are required to achieve the desired results. If chemicals are needed, these tests should be used to select the additive and its dosage requirements.

In wastewater treatment, air is the gas used in flotation, and the method of air supply is used to define the process. The three most common flotation processes are

1. *Dissolved air flotation (DAF)*: This occurs when air is injected while the wastewater is under pressure. Fine bubbles are released when the pressure is reduced in the flotation tank.
2. *Air flotation*: This occurs by means of aeration, typically by diffuser, at atmospheric pressure.
3. *Vacuum flotation*: This occurs when the wastewater is saturated with air before application of a vacuum.

DAF is most widely used because of its effectiveness for removing a wide range of solids. Conversely, simple air flotation has limited application. Vacuum flotation is not used frequently because of its comparatively high costs and increased operation and maintenance requirements. However, all flotation systems have higher operating costs for power and chemicals and require more skilled operators than alternative gravity settling systems.

Dissolved Air Flotation. The equipment and flow pattern for circular and rectangular DAF units are shown in Figures 6.43 and 6.44, respectively.

The design of a DAF unit must be developed for individual applications through predesign studies and tests. Table 6.41 lists typical data for design parameters and serves to illustrate the variability of design requirements. The selection of chemical additives must also be made in the predesign phase.

Air Flotation. For air flotation, air is applied near the bottom of an open aeration tank equipped with surface skimming equipment. Especially because of its limited effectiveness with most wastewater suspended solids, the design of the air flotation system should be developed through pilot or bench scale tests.

Vacuum Flotation. The commercially available vacuum flotation unit is a covered cylindrical tank plus an aeration tank, detention tank, vacuum pumps, and scum and sludge removal equipment.

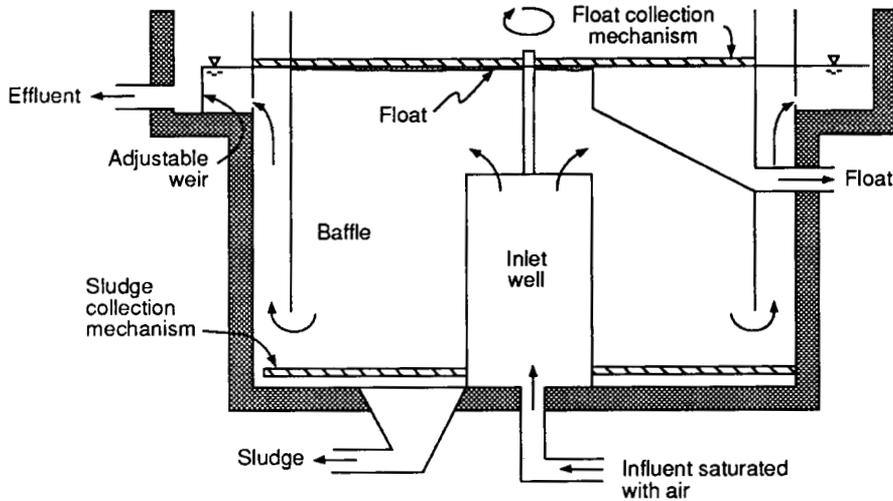


FIGURE 6.43 Circular dissolved air flotation unit.

Design data for vacuum flotation units are presented in Table 6.42. These units are generally less effective for removal of suspended solids than the DAF units.

Centrifugation

The practical use of centrifuges for suspended solids removal is limited to possible industrial pretreatment applications. In these cases, the wastewater would have a very high concentration of suspended solids. The more common application of centrifugation is in the concentrating and dewatering of wastewater treatment sludges, which is presented in a later section.

The common types of centrifuges are basket, disk, and solid bowl. The perforated basket type is well suited for removal of coarse solids. The imperforated basket and solid bowl types have application for removal of fine and coarse solids. The disk type is used in the removal of fine solids. With all types, the use of chemical additives increases the performance of the centrifuge.

TABLE 6.41 Dissolved Air Flotation Design Data (16, 49)

Parameter	Typical value	Remarks
Pressure	40–50 psig (270–340 kN/m ²)	Range, 25–70 psig (170–475 kN/m ²)
Air-to-solids ratio	0.03:0.05	Range, 0.01:0.20
Water depth	3–10 ft (1–3 m)	Commercial units
Surface loading rate	500–8000 gpd/ft ² (20–325 m ³ /day/m ²)	
Retention tank, detention	0.5–3 min	
Float detention	20–60 min	
Recycle, %	5–120	When utilized

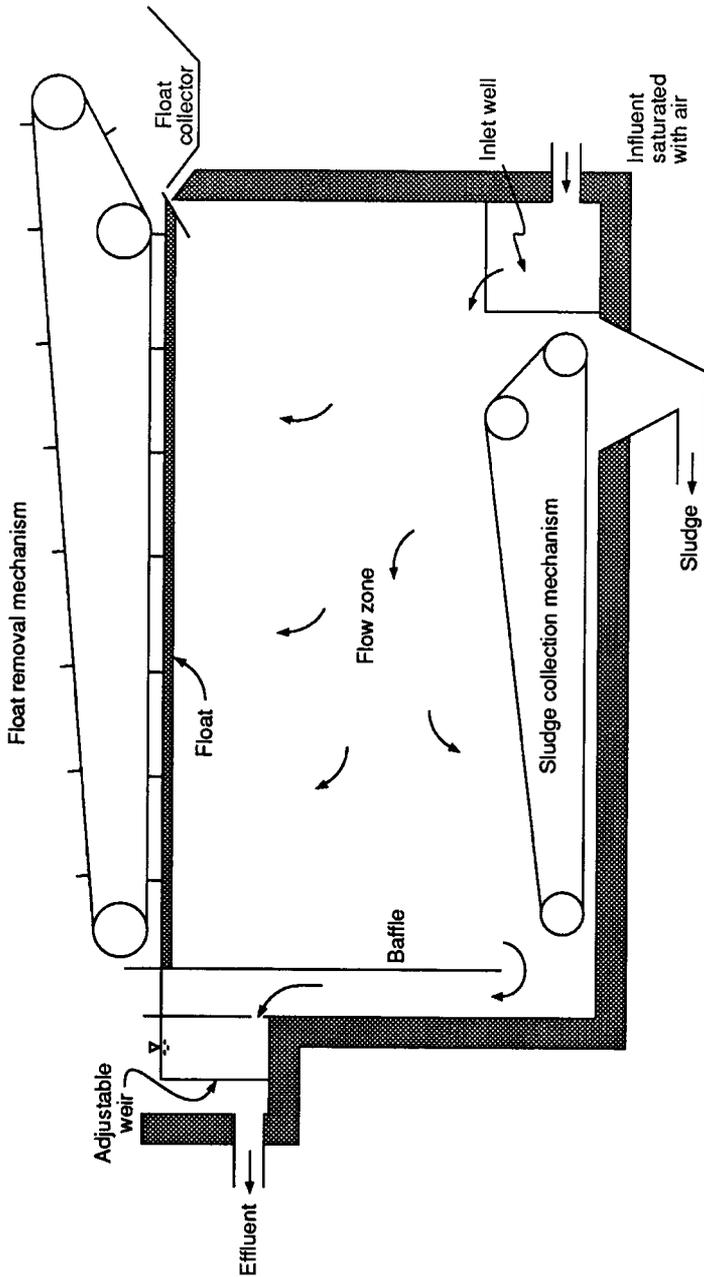


FIGURE 6.44 Rectangular dissolved air flotation unit.

TABLE 6.42 Vacuum Flotation Design Data (16)

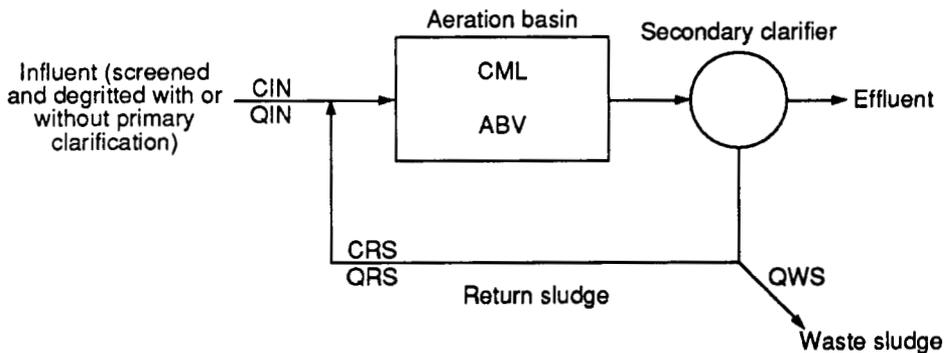
Parameter	Typical value
Tank diameter	12–60 ft (3.7–18.2 m)
Water depth	10 ft (3 m)
Vacuum equivalent	230 min Hg
Air requirements	0.025–0.05 ft ³ /gal (0.18–0.37 m ³ /m ³) of wastewater
Overflow rates	5000–10,000 gpd/ft ² (200–400 m ³ /min/m ²)

The design of centrifuge units is based on the solids feed rate, particle size, solids density, feed consistency, temperature, and chemical additives. The expected performance is measured by the percent of solids removed and the moisture content of the recovered solids cake.

Filtration

The efficient removal of suspended solids by filtration requires an influent with relatively low solids concentrations, such as the effluent from a secondary equivalent treatment process. The common alternative systems include granular-media filtration, diatomaceous earth filtration, and ultrafiltration. Fine screens, e.g., microscreens, are an alternative unit process; these were discussed earlier in this section.

Since granular-media filters can better accept even these reduced suspended solids loadings, they are by far the most widely used filtration system in wastewater treatment plants. This filtration process is presented in detail in the subsequent section on advanced wastewater treatment techniques.



CIN - Influent BOD₅ concentration

QIN - Influent flow per day

CML - Suspended solids concentration in aeration basin

ABV - Aeration basin volume

QWS - Waste sludge flow per day

CRS - Suspended solids concentration in return sludge

QRS - Return sludge flow rate

FIGURE 6.45 Activated sludge design flow schematic.

AEROBIC BIOLOGICAL TREATMENT

Aerobic biological wastewater treatment is the process by which microorganisms use the waste's organic component, in the presence of oxygen, to produce cell growth and end products of carbon dioxide and water. Most designs are developed to achieve secondary wastewater treatment levels. The primary processes are characterized as activated sludge, pond systems, trickling filters, and rotating biological contactors.

Activated Sludge

The activated sludge process is a continuous-flow, aerobic biological process for the treatment of domestic and biodegradable industrial wastewaters (16, 46, 50, 52, 53). The process provides a high-quality effluent and is characterized by the suspension of microorganisms, which are maintained in a relatively homogeneous state with the wastewater by mixing induced by the aeration system. The mixture is referred to as mixed liquor. Fundamentally, the microorganisms oxidize the soluble and suspended organics to carbon dioxide and water in the presence of oxygen. Some of the organic material is synthesized into new cells or used to support growth of existing cells; the surplus is comprised of waste and excess sludge.

The overall treatment process will include preliminary, and often primary, treatment before the aeration basin(s). The mixed liquor is discharged to a secondary clarifier where the microorganisms settle out and are recycled to the aeration basin. Excess sludge is piped to separate sludge-handling processes. The clarifier overflow proceeds to disinfection and final discharge or to supplemental treatment, if required.

The companion review and selection of the activated sludge process and aeration system are the primary design considerations.

Activated Sludge Processes. Discussions follow on the conventional activated sludge process and its more common modifications: extended aeration, contact stabilization, high-rate modified aeration, step aeration, and oxygen-activated sludge.

Other processes, some patented, seek to improve the state of the art and should be considered by the designer on a case-by-case basis. For example, sequencing batch reactor (SBR) systems combine biological treatment and sedimentation in a single basin. The decant goes to additional treatment units, such as tertiary filters, or directly to effluent disinfection. Decant equalization should be considered to reduce the size of treatment units and to provide for more consistent operations and controls.

Activated sludge processes are designed based on the mixed liquor suspended solids (MLSS) and the BOD (or COD) loading. The MLSS represents the quantity of microorganisms performing treatment in the aeration basin. (Some designers use the volatile portion of the mixed liquor, MLVSS, to more closely approximate the microorganism population.) The organic loading establishes the aeration system design requirements. Some industrial system designs are based on COD or COD:BOD ratios.

The fundamental formulas used to design activated sludge systems follow. Figure 6.45 illustrates the basic process components. Typical values of design data are included in subsequent discussions on individual processes.

$$\text{Volumetric loading} = \frac{(\text{CIN})(\text{QIN})(\text{CMF})}{\text{ABV}}$$

where CIN = influent BOD₅ concentration

QIN = influent flow per day

CMF = concentration to mass conversion factor

ABV = aeration basin volume

The F/M ratio is the food-to microorganism ratio expressed as the ratio of loading rate to mixed liquor suspended solids, or

$$\text{F/M ratio} = \frac{(\text{CIN})(\text{QIN})(\text{CMF})}{(\text{CML})(\text{ABV})(\text{CMF})}$$

where CML = suspended solids concentration in aeration basin (MLSS, mixed liquor suspended solids)

$$\text{Detention time} = \frac{\text{ABV}}{\text{QIN}}$$

Note: ABV is calculated by specifying the volumetric loading or the CML (MLSS) and F/M ratio.

$$\text{Percent recycle} = \frac{\text{CML}}{\text{CRS} - \text{CML}} \times 100$$

where CRS = suspended solids concentration in return sludge

$$\begin{aligned} \text{Daily oxygen requirements} &= \text{oxygen for respiration} \\ & \quad (0.35 \text{ to } 0.65 \text{ O}_2/\text{BOD removed}) + \text{oxygen for decay} \\ & \quad (0.05 \text{ to } 0.10 \text{ O}_2/\text{sludge in aeration basin}) \end{aligned}$$

$$\text{Sludge retention time} = \frac{(\text{ABV})(\text{CML})}{(\text{QRS})(\text{CRS}) - (\text{QIN})(\text{CIN})}$$

where QRS = return sludge flow rate

$$\text{Daily waste sludge flow} = \frac{(Y)(\text{BODR} - (D)(\text{ABV})(\text{CML}))}{\text{CRS}}$$

where Y = biological yield coefficient, which ranges from 0.3 to 0.8 (0.4 to 0.6 for domestic wastes)

BODR = BOD fraction removed, its concentration multiplied by QIN

D = biological decay coefficient, which ranges from 0.04 to 0.20 per day (0.06 to 0.10 per day for domestic wastes)

$$\text{Daily waste sludge} = (\text{CRS})(\text{QWS})(\text{CMF})$$

where QWS = waste sludge flow per day

Conventional Activated Sludge. The conventional activated sludge process is perhaps the most versatile and widely used biological process for secondary treatment of municipal wastewaters or wastewaters of similar character. A primary advantage of this system is its relatively low initial cost. Typical design data are presented in Table 6.43; the flow scheme is shown in Figure 6.46.

Extended Aeration. The basic principle of the extended aeration process is to retain the mixed liquor in the aeration basin long enough so that the production rate of new cells is the same as the decay rate of existing cells. The theoretical result is no excess sludge production. In practice, there will be excess sludge, but

TABLE 6.43 Conventional Activated Sludge Design Data (16, 46, 50, 52)

Parameter	Typical value
Volumetric loading	25–50 lb BOD ₅ /day/1000 ft ³ (0.4–0.8 kg BOD ₅ /day/m ³)
MLSS	1500–4000 mg/L
F/M ratio	0.2–0.5 lb BOD ₅ /day/lb MLSS (0.2–0.5 kg BOD ₅ /day/kg MLSS)
Detention time	4–8 h
Recycle	50–100%
Air supply	
Diffused aeration	800–1500 scf/lb BOD ₅ removed (50–95 m ³ /kg BOD ₅ removed)
Mechanical aeration	0.8–1.1 lb O ₂ /lb BOD ₅ removed (0.8–1.1 kg O ₂ /kg BOD ₅ removed)
Sludge retention time	5–10 days
Waste sludge	0.4–0.6 lb/lb BOD, removed (0.4–0.6 kg/kg BOD, removed)

the quantity will be far less than with other activated sludge processes. Common design data are shown in Table 6.44.

The term *oxidation ditch* is used to describe an extended aeration system that employs a race-track-shaped basin and horizontally mounted (brushes or disks) aerators to provide aeration and circulation. A later modification of this system is the carousel process, which uses vertically mounted aerators.

Contact Stabilization. The premise of the contact stabilization process is that colloidal and insoluble BOD is removed rapidly by biological sorption, synthesis, and flocculation during a relatively short contact time. Thus, a cost savings in tankage may be achieved. As the fraction of soluble BOD increases, the aeration volume increases toward that of conventional activated sludge, and the savings are diminished.

The preliminary or primary treated wastewater is discharged to an aerated contact basin. The return sludge goes to a reaeration (stabilization) basin where the flocculated and absorbed BOD is stabilized before discharge to the contact basin. Figure 6.47 illustrates the flow patterns, and Table 6.45 summarizes typical design data.

High Rate. The high-rate activated sludge process alone does not have a demonstrated history of producing an effluent with BOD₅ and suspended solids concentrations within the 30 mg/L range (each). The process limitation is that it uses a relatively short detention time and high volumetric loading; design data are presented in Table 6.46. Thus, applications are for pretreatment, or roughing, process before a second-stage activated sludge system.

Modified Aeration. Though not a secondary treatment process, the modified-aeration activated sludge process is useful for achieving BOD₅ removals in the 50 to 70% range. The process uses high volumetric

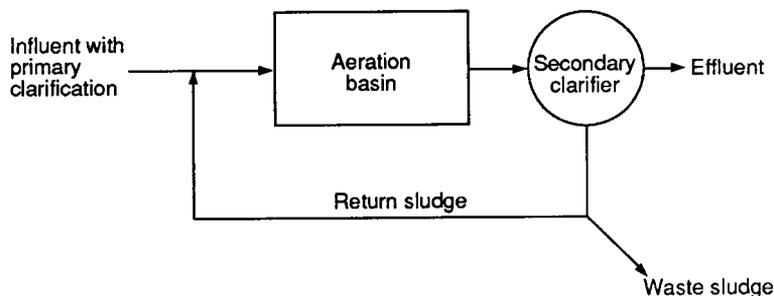
**FIGURE 6.46** Conventional activated sludge flow schematic.

TABLE 6.44 Extended Aeration Design Data (16, 46, 50, 52)

Parameter	Typical value
Volumetric loading	10–15 lb BOD ₅ /day/1000 ft ³ (0.15–0.25 kg BOD ₅ /day/m ³)
MLSS	2000–6000mg/L
F/M ratio	0.05–0.15 lb BOD ₅ /day/lb MLSS (0.05–0.15 kg BOD ₅ /day/kg MLSS)
Detention time	24 h; range, 18–36 h
Recycle	75–150%
Air supply	
Diffused aeration	1700–2000 scf/lb BOD ₅ removed (100–125 m ³ /kg BOD ₅ removed)
Mechanical aeration	1–1.5 lb O ₂ /lb BOD ₅ removed (1–1.5 kg O ₂ /kg BOD ₅ removed)
Sludge retention time	20–40 days
Waste sludge	0.15–0.3 lb/lb BOD ₅ removed (0.15–0.3 kg/kg BOD ₅ removed)

TABLE 6.45 Contact Stabilization Design Data (16, 46, 50, 52)

Parameter	Typical value
Volumetric loading	30–70 lb BOD ₅ /day/1000 ft ³ (0.5–1.1 kg BOD ₅ /day/m ³)
MLSS	
Contact	1000–2500 mg/L
Stabilization	4000–10,000 mg/L
F/M ratio	0.2–0.5 lb BOD ₅ /day/lb (0.2–0.5 kg BOD ₅ /day/kg) MLSS
Detention time	
Contact	1–3 h
Stabilization	3–6 h
Recycle	50–100%
Air Supply	
Diffused aeration	800–1200 scf/lb (50–75 m ³ /kg) BOD ₅ removed
Mechanical aeration	0.7–1.0 lb O ₂ /lb (0.7–1.0 kg O ₂ /kg) BOD ₅ removed
Sludge retention time	5–10 days
Waste sludge	0.4–0.6 lb/lb (0.4–0.6 kg/kg) BOD ₅ removed

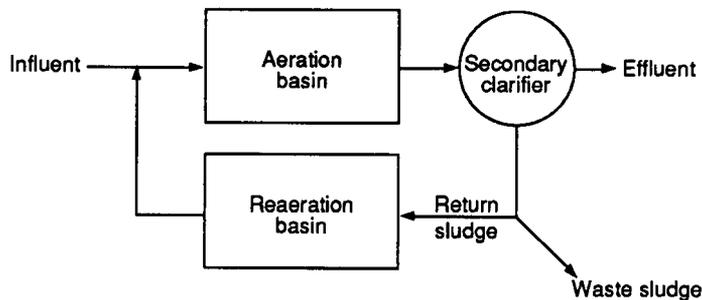


FIGURE 6.47 Contact stabilization flow schematic.

TABLE 6.46 High-Rate Activated Sludge Design Data (16, 50, 52)

Parameter	Typical value
Volumetric loading	50–150 lb BOD ₅ /day/1000 ft ³ (0.8–2.4 kg BOD ₅ /day/m ³)
MLSS	3000–5000 mg/L
F/M ratio	0.4–1.0 lb BOD ₅ /day/lb (0.4–1.0 kg BOD ₅ /day/kg) MLSS
Detention time	2–4 h
Recycle	25–100%
Air supply	
Diffused aeration	800–1200 scf/lb (50–75 m ³ /kg) BOD ₅ removed
Mechanical aeration	0.7–1.2 lb O ₂ /lb (0.7–1.2 kg O ₂ /kg) BOD ₅ removed
Sludge retention time	2–4 days
Waste sludge	0.5 to 0.7 lb/lb (0.5–0.7 kg/kg) BOD ₅ removed

TABLE 6.47 Modified-Aeration Activated Sludge Design Data (16, 50, 52)

Parameter	Typical value
Volumetric Loading	50–125 lb BOD ₅ /day/1000 ft ³ (0.8–2 kg BOD ₅ /day/m ³)
MLSS	500–2000 mg/L
F/M ratio	0.75–1.5 lb BOD ₅ /day/lb (0.75–1.5 kg BOD ₅ /day/kg) MLSS
Detention time	1–3 h
Recycle	10–100%
Air supply	
Diffused aeration	400–800 scf/lb (25–50 m ³ /kg) BOD ₅ removed
Mechanical aeration	0.4–0.7 lb O ₂ /lb (0.4–0.7 kg O ₂ /kg) BOD ₅ removed
Sludge retention time	Less than 2 days
Waste sludge	0.8–1.2 lb/lb (0.8–1.2 kg/kg) BOD ₅ removed

loadings, low MLSS concentrations, short detention time, and low air supply. Numerical design data are presented in Table 6.47.

Step Aeration. For the step (feed) aeration process, the influent wastewater enters the aeration basin along the basin's length and only return sludge enters at the head of the basin. This approach yields a more uniform oxygen demand in the basin and a more stable environment for the microorganisms. It also causes the lowest solids loading on the final clarifier for a given mass of mixed liquor in the aeration basin. Table 6.48 gives typical design data. A flow diagram appears in Figure 6.48.

Oxygen Activated Sludge. The distinguishing feature of the oxygen-activated sludge process is that high-purity oxygen rather than air is the oxygen source for the microorganisms (16, 54). Usually, the tanks are covered to maximize oxygen utilization. Otherwise, it is similar to the high-rate activated sludge process.

The process diagram for a three-stage covered system is shown in Figure 6.49. Design data are presented in Table 6.49. Complexity of operation and costs of oxygen generation are important considerations in evaluating this alternative. Oxygen systems are most favored when land area is limited and/or high-strength wastes are to be treated.

Sequencing Batch Reactor. A sequencing batch reactor (SBR) functions as a nonsteady-state activated sludge process operating on a fill and draw basis. The system combines biological (activated sludge) treatment processes and sedimentation in a single basin. The operating phases of a SBR system are listed in Table 6.50.

TABLE 6.48 Step-Aeration Design Data

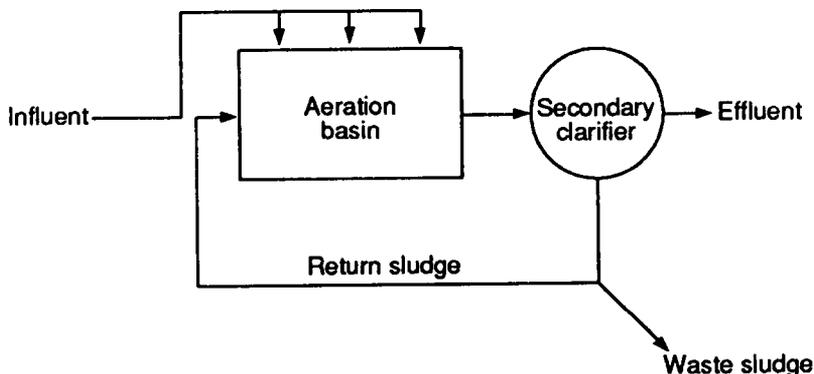
Parameter	Typical Value
Volumetric loading	30–50 lb BOD ₅ /day/1000 ft ³ (0.5–0.8 kg BOD ₅ /day/m ³)
MLSS	1000–3000 mg/L
F/M ratio	0.2–0.4 lb BOD ₅ /day/lb (0.2–0.4 kg BOD ₅ /day/kg) MLSS
Detention time	3–6 h
Recycle	25–50%
Air supply	
Diffused aeration	800 to 1200 scf/lb (50–75 m ³ /kg) BOD ₅ removed
Mechanical aeration	0.7–1.0 lb O ₂ /lb (0.7–1.0 kg O ₂ /kg) BOD ₅ removed
Sludge retention time	5–10 days
Waste sludge	0.5–0.8 lb/lb (0.5–0.8 kg/kg) BOD ₅ removed

In applications with widely varying flow rates and/or waste loads, equalization should be considered before the SBR. This is particularly applicable to many industrial process wastewater treatment applications. The sizing of equalization also should be coordinated with treatment objectives. Aerobic, anoxic, and anaerobic conditions may be defined by the operations strategy to meet treatment and effluent objectives.

After SBR treatment, the decant goes to additional treatment units, such as tertiary filters, or directly to effluent disinfection and discharge. Since, as a function of the cycle times, peak flow rates may be four to six times the average daily flow rates, decant equalization should be considered to reduce the size of subsequent treatment units and to provide for more consistent operations and controls.

Thus, when designing SBRs, there should be some method of reducing the peak decant rate before the treated wastewater is disinfected. Otherwise, the design rate of the chlorine dosage must meet very high rates of chlorine feed or ultraviolet dosage capacity with oversized contact basins. In addition, consideration must be given to the capacity of the receiving stream to accept the very high rate of discharge from the SBR system. This is especially true for small plants located on very small receiving streams.

Aeration System. The aeration system must supply the required quantity of oxygen and provide adequate mixing of the oxygen, mixed liquor, and wastewater. For the activated sludge process, aeration is provided

**FIGURE 6.48** Step aeration flow schematic.

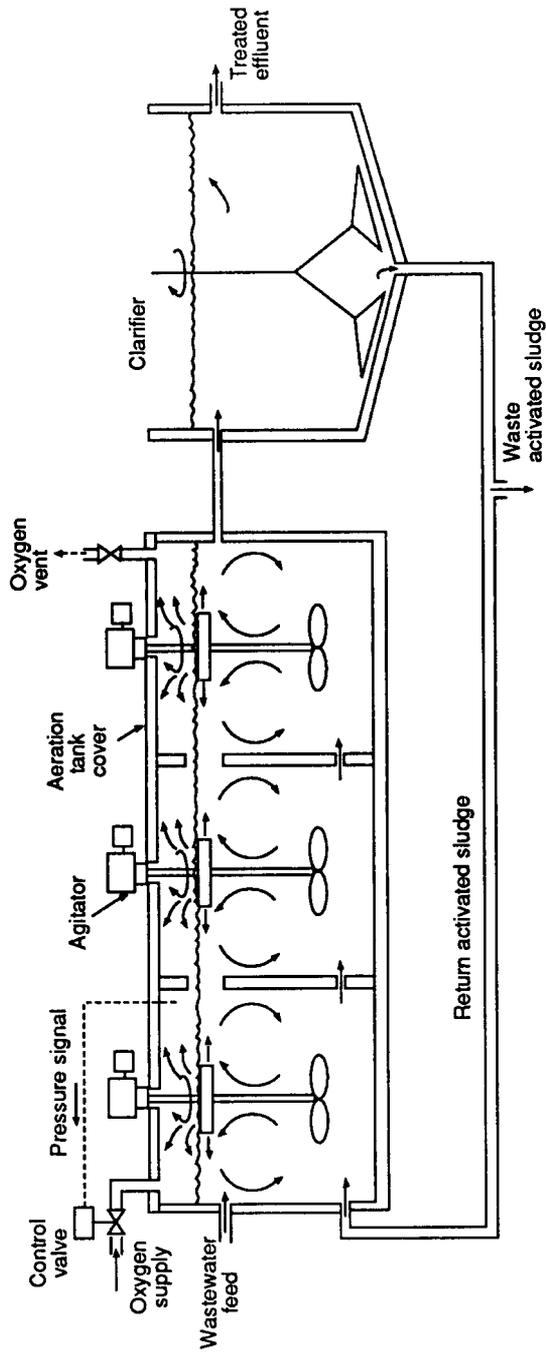


FIGURE 6.49 Oxygen-activated sludge flow schematic (54).

TABLE 6.49 Sequencing Batch Reactor Operating Phases

Phase	Operations
Static fill	Influent flow is introduced into the idle basin.
Mixed fill	Influent flow continues and mixing begins. Depending on the mode(s) of mixing, the reactor may be anoxic, aerobic, or a combination of both.
React fill	Influent flow and mixing continue and aeration begins.
React	Influent flow stopped and mixing and aeration continue.
Settle	Mixing and aeration stopped and clarification begins.
Decant	Supernatant is discharged.
Idle	Basing on standby to restart the process.
Waste sludge	Optional operation during the idle phase.

by diffused or mechanical aeration systems. Some designs separate the mixing and oxygen supply functions. Examples include turbine aeration systems and systems using independent submerged mixers.

The selection of aeration equipment requires a computation of oxygen transfer capability at seasonal temperatures for the specific wastewater to be treated. The following formula is used:

$$N = N_0 \alpha \frac{(BPC_s - C_L)}{C_{ss}} (1.024)^{(T-20)}$$

where N = Oxygen transfer rate under actual field operating conditions, lb O₂/hp·h (kg O₂/kW·h)

N_0 = Oxygen transfer rate in clean water under standard conditions (supplied by manufacturers), lb O₂/hp·h (kg O₂/kW·h)

α = Ratio of oxygen transfer in wastewater to that in clean water, 0.9 typical

B = Ratio of dissolved oxygen saturation of wastewater under aeration to that of clean water under the same conditions of temperature and pressure, usually 1

P = Ratio of barometric pressure at site to that of sea level, 0.9 to 1 typical

C_s = Dissolved oxygen saturation for clean water at operating temperature and one atmosphere pressure, mg/L

C_L = Dissolved oxygen to be maintained during actual operation, normally 2 mg/L

C_{ss} = Dissolved oxygen saturation for clean water at standard conditions of 20°C and one atmosphere pressure = 9.17 mg/L

T = Temperature of wastewater, °C.

The aeration equipment is selected on the basis of the lower value of N for the temperature ranges expected.

TABLE 6.50 Oxygen Activated Sludge Design Data

Parameter	Typical value
Volumetric loading	150–250 lb BOD ₅ /day/1000 ft ³ (2.4–4 kg BOD ₅ /day/1000 m ³)
MLSS	4000–8000 mg/L
F/M ratio	0.5–1.0 lb BOD ₅ /day/lb (0.5–1.0 kg BOD ₅ /day/kg) MLSS
Detention time	1.5–2 h
Recycle	25–100%
Oxygen supply	0.85–1.35 lb O ₂ /lb (0.85–1.35 kg O ₂ /kg) BOD ₅ removed
Sludge retention time	1.5–5.5 days
Waste sludge	0.3–0.9 lb/lb (0.3–0.9 kg/kg) BOD ₅ removed

In order to provide adequate mixing, the design should ensure a diffused air supply of 20 to 30 cfm/1000 ft³ (0.02 to 0.03 m³/min·m³) of tank volume. Mechanical aeration systems require 0.5 to 1.0 hp/1000 ft³ (13 to 26 kW/km³) for mixing of oxygen and 10 times more for complete mixing of solids.

Diffused Aeration A diffused air system includes a blower, pipe distribution network, and diffusers that are used to disperse air into the aeration tank. The diffusers are categorized by those that produce fine bubbles from porous media or that produce larger (coarse) bubbles. The fine-bubble systems have better oxygen transfer efficiencies but are less popular than coarse-bubble systems, which have lower capital and maintenance costs.

Porous diffusers are made of ceramic domes, plates, tubes, or plastic-cloth tubes or bags which produce fine bubbles. The airflow provides good mixing, but the spiral mixing pattern limits aeration tank geometry. The fine bubbles provide high oxygen transfer efficiency and the ability to vary airflow affords good operational flexibility. The oxygen transfer ranges from 1.8 to 2.5 lb O₂/hp·h (1.1 to 1.5 kg O₂/kW·h) under conditions of zero dissolved oxygen, 20°C, and 14.7 lb/in₂ (101 kN/m²) in clean water.

Coarse-bubble aeration is derived from nonporous diffusers, which may be made by drilling holes in pipes or by loosely attaching plates or disks to a base or pipe. These diffusers offer about 50 to 70% of the oxygen transfer of the porous type but are much less prone to clogging, resulting in lower maintenance cost and high system performance reliability.

Other aeration devices include tubular systems, which provide an entrained air-water mixture that is forced through a vertical cylinder equipped with a static mixer, and jet systems, which mix compressed air and pumped liquid in a nozzle at the point of discharge. The oxygen transfer efficiency of the tubular system is about the same as porous fine-bubble diffusers, and the jet systems are about 40% higher.

Mechanical Aeration. The key component of mechanical aeration is a surface or turbine aerator. The mechanical surface aerators are subdivided as low speed (radial flow), high-speed (axial flow), or horizontal shaft.

Low-speed aerators operate in the range of 20 to 60 r/min using a gearbox to reduce the motor speed. The basic configuration has the impeller agitating the water surface; modifications may include a draft tube and/or a lower mixing impeller. Typical oxygen transfer efficiency is 2.0 to 4.0 lb O₂/hp·h (1.2 to 2.4 kg O₂/kW·h) under standard conditions. The units may be floating or fixed mounted and be provided as two-speed units.

High-speed aerators are direct, motor-driven units mounted on floating structures and operated in the range of 300 to 1200 r/min. Due to concern about the shearing of biological floc and less mixing from the relatively low pumping capacity, high-speed aerators have been directed away from activated sludge systems, but are popular for use in aerated lagoons. Oxygen transfer efficiencies are essentially the same for high- and low-speed aerators.

Other types of mechanical aeration equipment includes disks or brushes mounted on a rotating horizontal shaft. Since the unit imparts movement as well as oxygen transfer, they are widely used in the oxidation ditch–race track type of aeration basin.

Turbine aerators combine the mixing and aeration features of surface aerators and diffused aeration, respectively, to provide high capacity input per unit volume (see Figure 6.50). There is extra flexibility in operation by varying the air supply. Using a fixed bridge or platform support, turbine aerators include one or more slow-speed impellers mounted over a coarse air diffuser.

Aerobic Pond Systems

Historically, aerobic wastewater stabilization pond systems have been a principal biological treatment method for a variety of wastewaters ranging from residential domestic to complex industrial. They may be used alone or in combination with other treatment processes. The advent of aeration via mechanical sources added yet a broader use of pond systems.

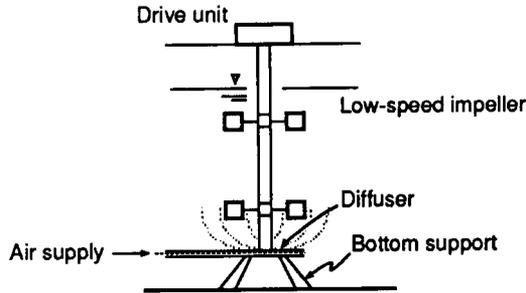


FIGURE 6.50 Turbine aerator.

Common design considerations for aerobic treatment ponds is presented in Table 6.51. Physical features of a three-cell pond system are shown in Figure 6.51, and a basic effluent structure is detailed in Figure 6.52.

The pond shape should be round, square, or rectangular. The latter should have rounded corners, and a length less than three times the width. Common-wall dike construction and baffles are permissible. The volume of a square or rectangular pond with sloped sides and rounded corners is calculated by the following formula:

$$V = [(LW) + (L - 2sd)(W - 2sd) + 4(L - sd)(W - sd)] \frac{d}{6}$$

TABLE 6.51 Aerobic Waste Stabilization Pond Design Consideration

Item	Typical criteria
Bottom material	Use clays, mixes, or liners to effectively eliminate or minimize seepage
Dike material	Compacted, relatively impervious material or special features, such as liners used on bottom
Dike top width	8–10 ft (2.4–3 m) minimum
Dike slopes	Maximum: 3 horizontal to 1 vertical, minimum: 4 horizontal to 1 vertical
Erosion control	As required by wind and/or wave actions
Freeboard	Minimum: usually 3 ft (1 m)
Vegetation	None along or below water surface
Liquid depth	Function of pond type
Number of cells	Minimum: two; common: three
Mode of operation	Series except in larger systems
Inlet structure	Leeward side location with discharge to concrete splash pad Depressed area for sludge collection recommended
Outlet structure	Windward side location with provisions to alter operating water level and to drain pond, if required
Surface drainage	Diverted from or around pond
Security fencing	Good practice around entire facility. Height to 7 ft (2-m) with 10-ft (3-m) gate.
Signage	Warning sign(s) suggested
Area lighting	As required

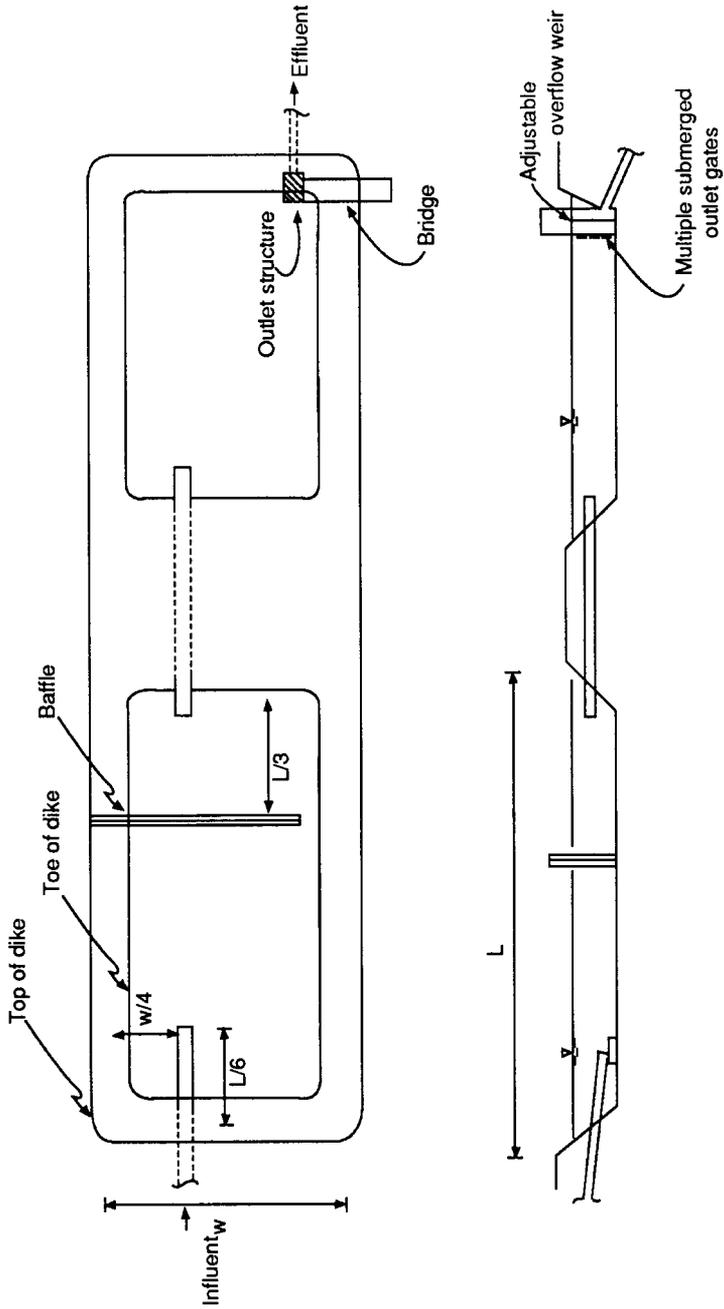


FIGURE 6.51 Three-cell pond system.

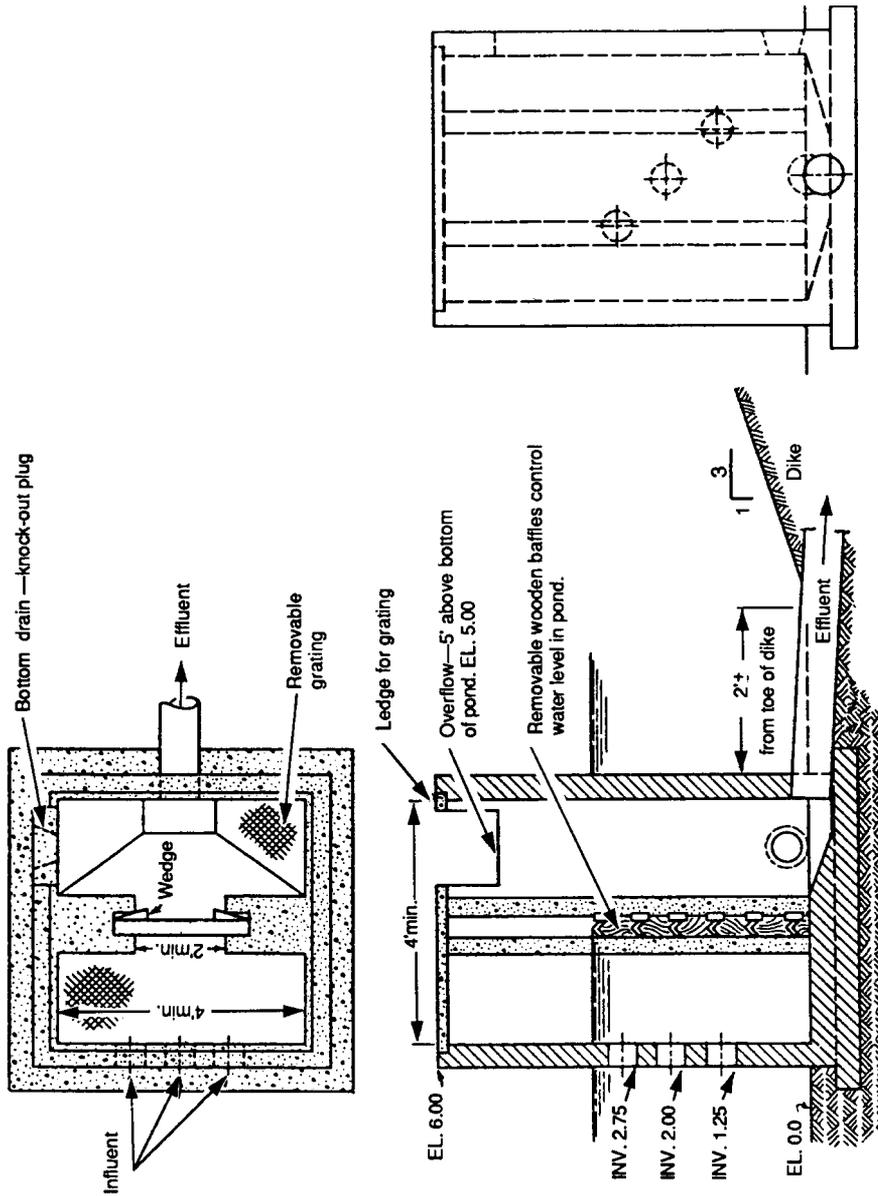


FIGURE 6.52 Subsurface pond effluent structure.

where V = Volume, ft³ (m³)
 L = Length of pond at water surface, ft (m)
 W = Width of pond at water surface, ft (m)
 s = Horizontal to vertical slope factor
 d = Depth of pond, ft (m)

The three principle types of aerobic ponds are

1. Aerobic
2. Facultative
3. Aerated

Furthermore, pond systems are characterized hydraulically as continuous discharge, controlled discharge, or retention (no discharge to surface waters).

Aerobic Ponds. Aerobic ponds, also known as high-rate aerobic ponds, use oxygen from photosynthesis and surface reaeration to maintain dissolved oxygen throughout the entire depth. As shown by the design data in Table 6.52, the ponds are shallow and have relatively short detention times. An algae removal process is needed to control effluent suspended solids.

The use of aerobic ponds is limited by the need for a warm, sunny climate and by effluent suspended solids (algae). These ponds are used generally to reduce soluble BOD from other treatment processes.

Facultative Ponds. The most common type of waste stabilization pond is the facultative pond, also known as oxidation pond, which is characterized by an aerobic surface zone with a gradient to an anaerobic bottom zone. Design data are presented in Table 6.53.

Oxygen is supplied by photosynthesis and surface reaeration. The aerobic microorganisms stabilize organic wastes and by-products from the anaerobic processes in the lower layer. The pond outlet withdraws somewhat below the surface to minimize algae but at a level of satisfactory dissolved oxygen.

Aerated Ponds. By supplementing natural processes with oxygen from mechanical aeration, the aerated pond reduces land requirements and provides flexibility for variable waste loads and climatic changes. Table 6.54 presents typical design data.

The actual design of a system requires consideration of partial or complete mixing (i.e., activated sludge) to meet the treatment objectives. Most systems use mechanical rather than diffused aeration; both methods

TABLE 6.52 Aerobic Pond Design Data

Parameter	Typical value
Organic loading	75–150 lb BOD ₅ /acre/day (85–170 kg BOD ₅ /ha/day)
Depth	0.5–1.5 ft (0.15–0.45 m)
Detention time	5–20 days
pH range	6.5–10.5 (varies with time of day)
Temperature	
Range	32 to 104°F (0–40°C)
Optimum	68°F (20°C)
BOD ₅ removed	80–95%

TABLE 6.53 Facultative Pond Design Data

Parameter	Typical value
Organic loading	15 to 50 lb BOD ₅ /acre/day (17–55 kg BOD ₅ /ha/day)
Depth	3–8 ft (0.9–2.4 m)
Detention time	30–180 day
pH Range	6.5–9
Temperature	
Range	32–104°F (0–40°C)
Optimum	68°F (20°C)
BOD ₅ removed	75–95%

of aeration were presented earlier in the activated sludge discussions. Pond depth and size must be coordinated with the aeration system to prevent erosion of the pond's bottom and dikes.

The BOD removal efficiency of an aerated pond system is a function of temperature and detention time, as follows:

$$E = 100 - \frac{100}{1 + K_T t}$$

and

$$K_T = K_{20} \theta^{(T-20)}$$

where E = Efficiency of BOD removal, %

K_T = BOD removal rate at temperature T , °C

t = Detention time, days

K_{20} = BOD removal rate at 20°C

θ = Temperature coefficient, 1.135 below 20°C and 1.055 over 20°C

For domestic wastewater, K_{20} varies between 0.5 and 1.0; values may be several times higher for some industrial wastes. Actual K_{20} rates should be determined by laboratory or pilot plant studies. Using a K_{20} of 0.5, Table 6.55 illustrates the impacts of temperature and detention time on aerated pond performance.

TABLE 6.54 Aerated Pond Design Data

Parameter	Typical value
Organic loading	20–300 lb BOD ₅ /acre/day (25–335 kg BOD ₅ /ha/day)
Depth	6–20 ft (1.8–6.1 m)
Detention time	5–20 days
pH Range	6.5–8
Temperature	
Range	32–104°F (0–40°C)
Optimum	68°F (20°C)
BOD ₅ removed	80–85%

TABLE 6.55 Aerated Pond BOD₅ Removal Efficiency

Temperature		Detention time, days						
°C	°F	2	4	6	8	10	15	20
4	39	12	21	28	35	40	50	57
6	43	15	25	34	40	46	56	63
8	46	18	30	40	47	52	62	68
10	50	22	36	46	53	59	68	74
12	54	27	42	52	59	64	73	78
14	57	32	48	58	65	70	79	82
16	61	38	55	64	71	75	82	86
18	64	44	61	70	76	80	85	89
20	68	50	67	75	80	83	88	91
22	72	53	69	77	83	85	89	92
24	75	55	71	79	83	86	90	93
26	79	58	73	80	85	87	91	93
28	82	61	75	82	86	88	92	94
30	86	63	77	84	87	89	93	94
32	90	66	79	85	88	90	93	95
34	93	68	81	86	89	91	94	95
36	97	70	82	88	90	92	95	96
38	100	72	84	89	91	93	95	96

Trickling Filter

The trickling filter process employs an attached-growth biological system based on passing (trickling) organic wastewater over the surface of a biological growth on solid media (50, 53). The process advantages include low life-cycle cost, resistance to shock loading, and ease of operation. However, its inefficiency in attaining high levels of treatment performance, considering seasonal variations, has restricted the size of conventional trickling filter systems to satisfy modern absolute effluent criteria.

The organic portion of the wastewater is degraded under aerobic conditions by the microorganisms, also referred to as slime, attached to the filter media. The thickness of the slime layer increases due to growth of the microorganisms until the outer layer absorbs all the organic matter and causes the inner layer next to the media to enter an endogenous growth and lose its ability to cling to the media. Losing the slime layer is called sloughing and is a function of organic as well as hydraulic loading. For the high-rate and roughing filters, increased hydraulic loadings are needed to flush the slough through the filter and avoid anaerobic conditions as a result of filter clogging.

Trickling filters are generally classified, with respect to the application rate of the organic and hydraulic loadings, as low-rate, high-rate, and roughing (16, 50, 53, 55). The process may be further categorized by media type, depth, number of stages, mode of distribution, and/or dosing frequency. Common physical design features are shown in Table 6.56.

Historically, the trickling filter medium was rock. The principal criteria for media are specific surface area (area per unit volume) and percent void space. Organic loadings are directly related to the specific surface area available for biological growths to treat the wastes. Likewise, high void spaces permit increased hydraulic loadings and enhanced oxygen transfer.

In order to achieve the benefits (e.g., cost and land area) of higher organic and hydraulic loadings over conventional rock systems, molded (Figure 6.53) or other forms of plastic and other durable, insoluble, and uniformly sized media (such as redwood block or laths, Figure 6.54) have been developed and are most com-

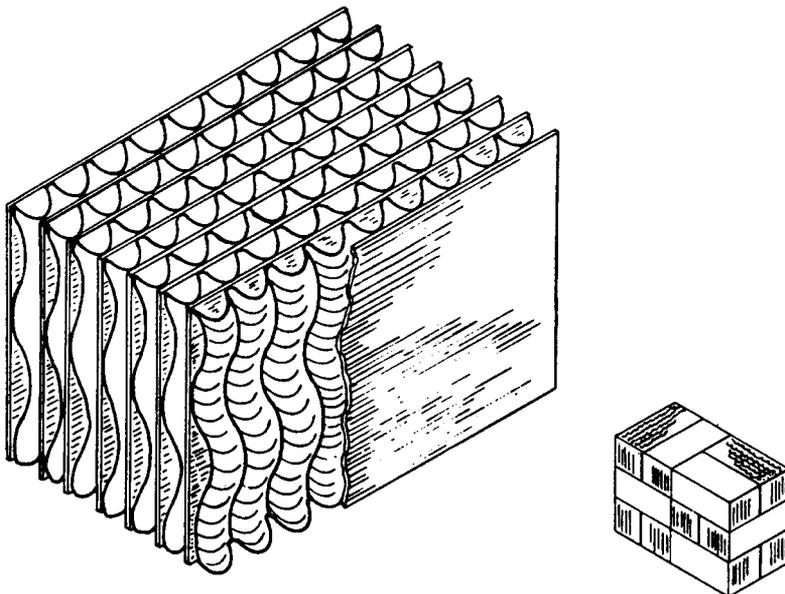
TABLE 6.56 Trickling Filter System Features

Design feature	Typical requirement
Distribution uniformity	$\pm 10\%$ of design volume per unit area
Distributor and medium clearance	6 in (0.15 m)
Freeboard above medium for windbreak	4 ft (1.3 m)
Underdrain slope	1% minimum
Underdrain system provisions for free passage of air	Flow half full at design peak hydraulic load
Effluent channel velocity	2 ft/s (0.6 m/s) at average daily application rate

mon in newer facilities. Table 6.57 shows a comparison of specific surface area and percent void space for selected media.

Continuous flow to the trickling filter media is desired to provide food for the treatment organisms and to prevent dehydration of the attached growth. This is especially important for the higher-rated systems using plastic media. In these cases, the minimum recommended application, or wetting, rate is 0.5 to 1.0 gal/min/ft² (30 to 60 m³/m²·d). When a dosing siphon (see Figure 6.55) is needed, it should be designed for applications every 3 to 5 mm at design flow so as to provide nearly continuous flow. Dosing siphons are rarely used in new designs and are limited to low-rate filters.

Flow is applied to the trickling filter by fixed nozzles or rotating distributor arms as illustrated in Figures 6.56 and 6.57, respectively. Generally, a fixed-nozzle system is found in older and/or smaller units and is accompanied by a dosing siphon. Fixed nozzles are used sometimes with roughing filters. It is noted that for the same hydraulic loading rate, the fixed-nozzle system has a lower shear velocity on the attached growth.

**FIGURE 6.53** Plastic trickling filter media (53).

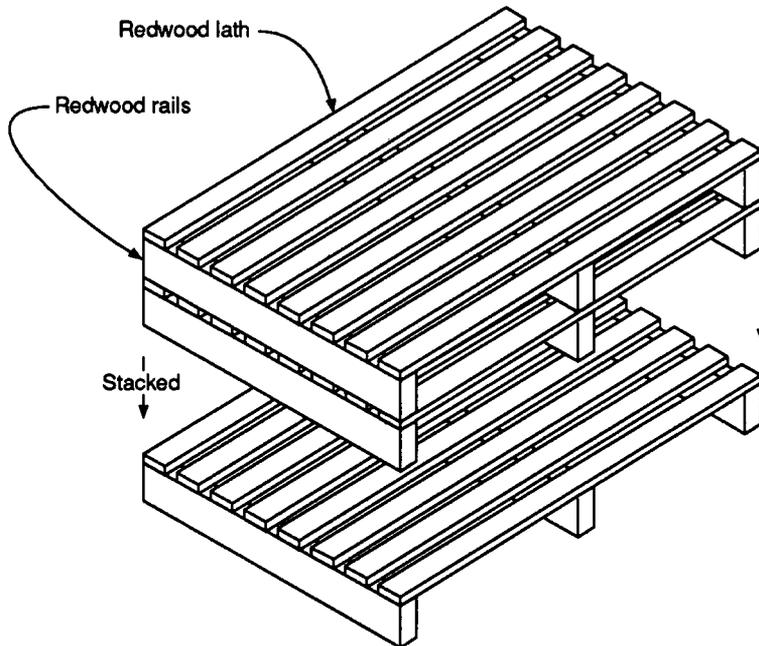


FIGURE 6.54 Redwood lath trickling filter media (53).

The rotating distributor system is currently most frequently used because of its reliability and ease of maintenance. The rotary distributor consists of two or four arms with the reaction force of the spray from orifices along the arm providing the rotating force for the assembly. A dosing siphon is used when the minimum flow during the day is insufficient to meet the hydraulic design of the distributor. The filter bed is typically 20 to 200 ft (6 to 65 m) in diameter. The tip speed of the distributor arms should be less than 4 ft/s (1.2 m/s) at peak design flow, and the bottom of the arm should be 0.5 to 1 ft (0.15 to 0.3 m) above the top of the media.

There must be adequate ventilation, $0.1 \text{ ft}^3/\text{min}/\text{ft}^2$ ($0.03 \text{ m}^3/\text{m}^2/\text{min}$), of the trickling filter to maintain aerobic conditions. With proper sizing of the underdrain system, this is normally accomplished by natural ventilation due to the difference in air and wastewater temperature. Forced ventilation may be needed for fil-

TABLE 6.57 Physical Properties of Selected Trickling Filter Media

Media	Nominal size, in (cm)	Specific surface area, ft^2/ft^3 (m^2/m^3)	Void space, %
Granite	1–3 (2.5–7.5)	19(63)	46
	4 (10)	13(43)	60
Slag	2–3 (5–7.5)	20(66)	49
	48 × 48 × 2	14(46)	76
Redwood	(19 × 19 × 0.8)		
	24 × 24 × 48	25–35(83–115)	94–97
Plastic	(9.5 × 9.5 × 19)		

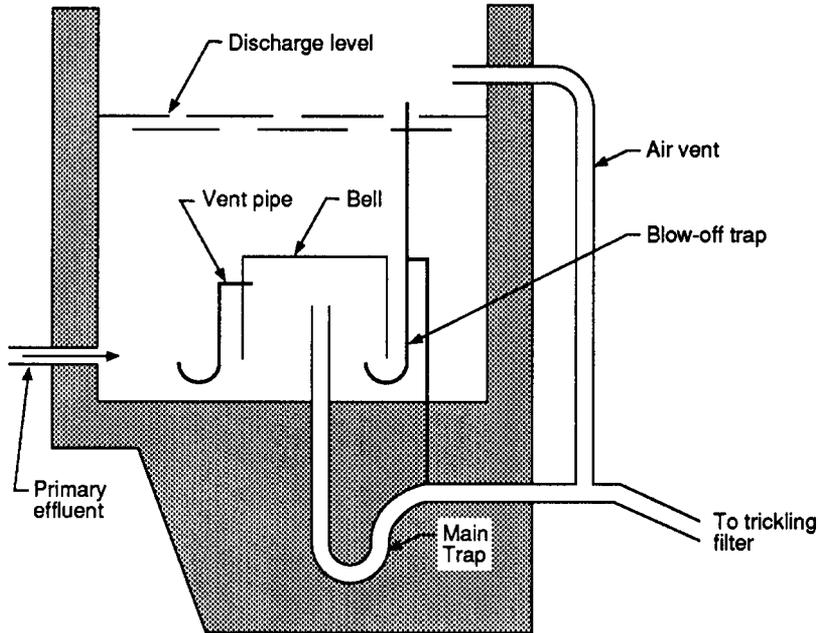


FIGURE 6.55 Typical dosing siphon (53).

ters with deep beds or ones that are heavily loaded, covered, or below grade. A typical design airflow is $1 \text{ ft}^3/\text{min}/\text{ft}^2$ ($0.3 \text{ m}^3/\text{m}^2/\text{min}$).

A number of design formulations have been developed for trickling filter systems. Among these are the National Research Council (56), Ten States Standard (57), Velz (58), Schulze (59), Germain (60), Eckenfelder (61, 62), and Galler and Gotaas (63) formulas. The applicability of each varies with such factors as medium, wastewater type, and recirculation, among others. Pilot tests offer the design engineer a project-specific means to evaluate design performance. The effects of temperature must be considered, as treatment performance declines markedly when the wastewater temperature drops below 50 to 59°F (10 to 15°C).

Design data and more process specifics for the low-rate, high-rate, and roughing trickling filter systems follow.

Low-Rate Trickling Filter. A typical low-rate trickling filter will have rock media with wastewater application by a rotary distributor. Common design data are contained in Table 6.58. Most of the BOD removal is accomplished in the upper 3 ft (1 m) and nitrification can occur in the remaining depth. The relationship of low-rate trickling filter and associated treatment units is shown in Figure 6.58.

High-Rate Trickling Filter. Using recirculation to increase the hydraulic loading, the high-rate trickling filter will accept higher organic loadings. There is a continuous sloughing of excess biological growths. The higher organic load precludes the development of nitrifying bacteria in the filter bed. Design data and flow schemes are presented in Table 6.59 and Figure 6.59, respectively. The media are composed of large rock or synthetic products. Multiple stages are necessary for comparable treatment with low-rate trickling filters.

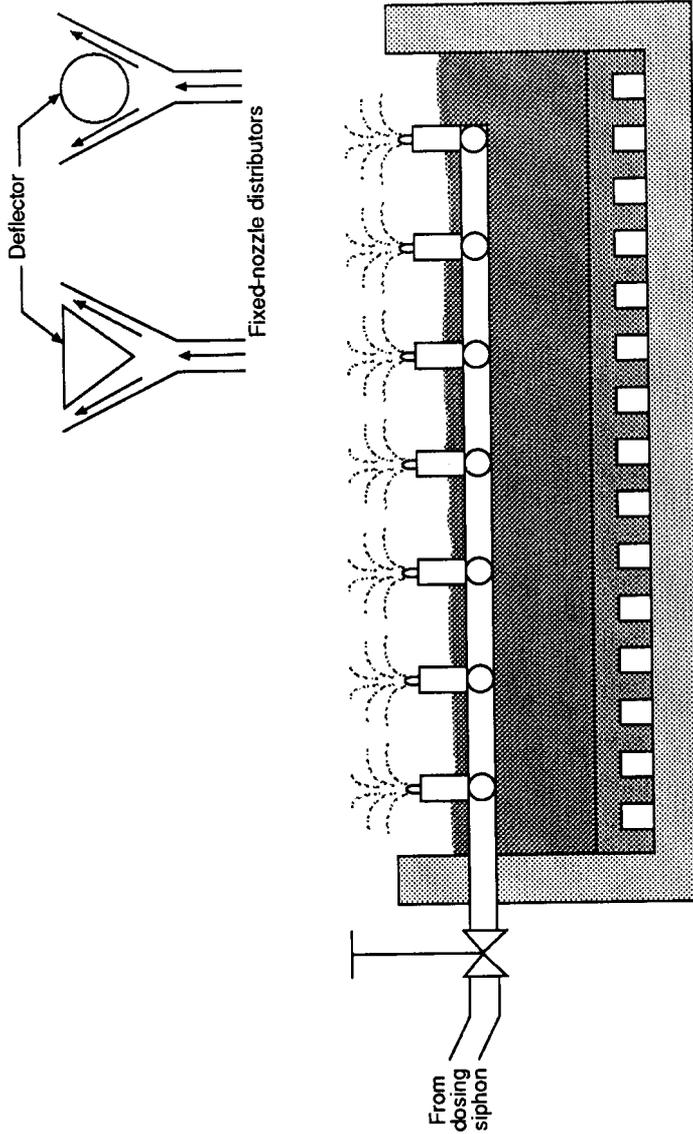


FIGURE 6.56 Fixed-nozzle distribution system (53).

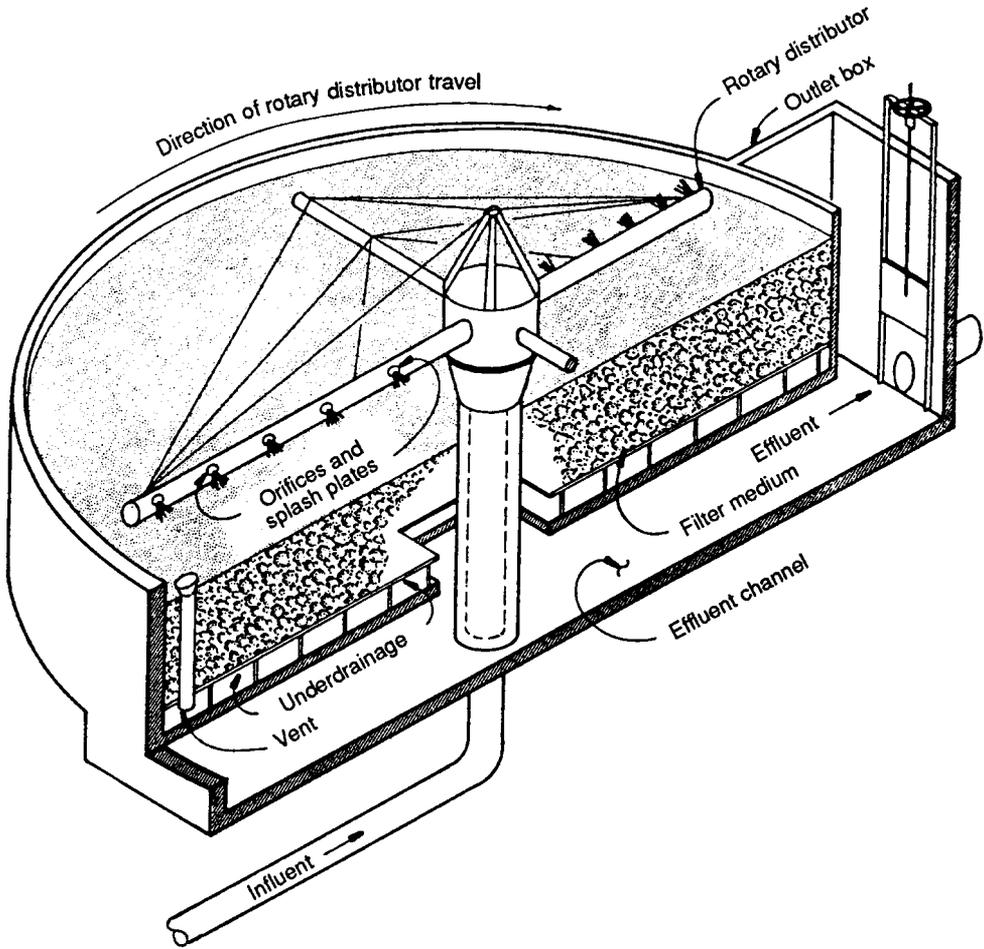


FIGURE 6.57 Cross section of typical trickling filter (53).

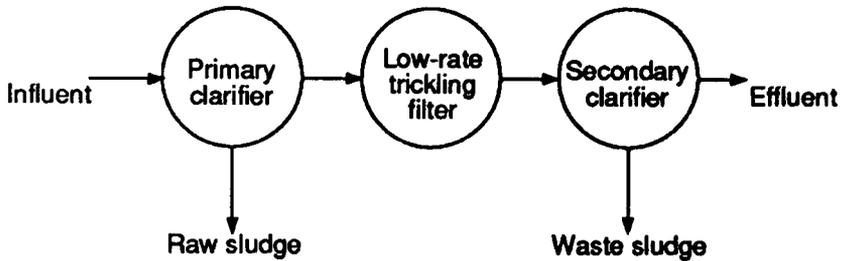


FIGURE 6.58 Low-rate trickling filter flow schematic.

TABLE 6.58 Low-Rate Trickling Filter Design Data

Parameter	Typical value
Organic loading	5–25 lb BOD ₅ /day/1000 ft ³ (200–1000 lb BOD ₅ /day/acre-ft) (0.08–0.4 kg BOD ₅ /day/m ³)
Hydraulic loading	25–100 gal/day/ft ² . (1–4 mgd/acre) (1–4 m ³ /day/m ²)
Recirculation ratio	None
Bed depth	5–10 ft (1.5–3 m)
Dosing interval	≤ 5 min
Sloughing	Intermittent
Nitrification	Highly nitrified
Clarifier surface	1000 gal/day/ft ² (40 m ³ /day/m ²)
BOD removal with sedimentation	80–90%

Roughing Filter. The primary function of a roughing filter is to reduce high organic loadings by serving as an intermediate treatment process upstream of an activated sludge, or perhaps secondary trickling filter, process. Although rock or other media may be used, the typical roughing filter uses elastic media. Maintenance of a wetting application rate around 0.75 gal/min/ft² (45 m³/m²/day) is an important design feature; other design data are presented in Table 6.60. The roughing filter installation commonly requires forced ventilation. A flow schematic, with typical flow path and structure options, is shown in Figure 7.60.

Rotating Biological Contactor

A rotating biological contactor (RBC) is an attached-growth process wherein the media are rotated through a basin of wastewater. The microorganisms are attached to large-diameter synthetic media mounted on a hori-

TABLE 6.59 High-Rate Trickling Filter Design Data

Parameter	Typical value
Organic loading	25–100 lb BOD ₅ /day/1000 ft ³ (1000–4000 lb BOD ₅ /day/acre-ft) (0.4–1.6 kg BOD ₅ /day/m ³)
Hydraulic loading	200–1000 gal/day/ft ² (10–45 mgd/acre) (8–41 m ³ /day/m ²)
Recirculation ratio	0.5–4
Bed depth	3–8 ft (0.9–2.4 m)
Dosing interval	≤ 5 min
Sloughing	Continuous
Nitrification	Limited
Clarifier surface overflow rate	800 gal/day/ft ² (32 m ³ /day/m ²)
BOD removal with sedimentation	65–85%

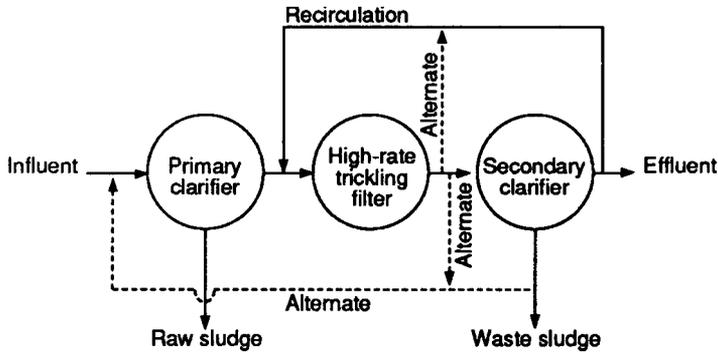


FIGURE 6.59 High-rate trickling filter flow schematic.

TABLE 6.60 Roughing Filter Design Data

Parameter	Typical value
Organic loading	100–500 lb BOD ₅ /day/1000 ft ³ (4000–20,000 lb BOD ₅ /day/acre-ft) (1.6–8 kg BOD ₅ /day/m ³)
Hydraulic loading	700–3000 gal/day/ft ² (30–130 mgd/acre) (28–122 m ³ /d/m ²)
Recirculation ratio	0.5–5
Bed depth	20–30 ft (6–9 m)
Dosing interval	Continuous
Sloughing	Continuous
Nitrification	Limited
Clarifier surface	1000 gal/day/ft ² (40 m ³ /day/m ²)
BOD ₅ removal with sedimentation	40–70%

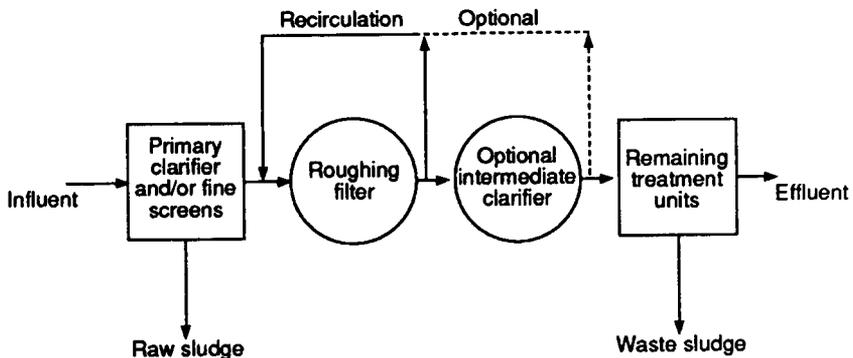


FIGURE 6.60 Roughing filter flow schematic.

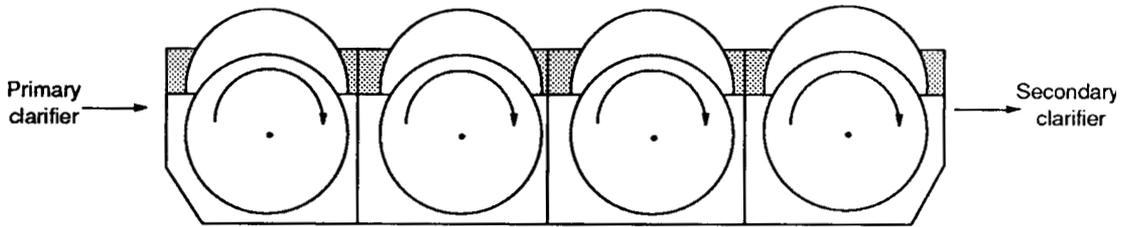


FIGURE 6.61 Rotating biological contactor flow schematic.

zontal shaft and placed at about 40% submergence in a contoured-bottom tank. Generally, the media are some 10 to 12 ft (3–3.5 m) in diameter and rotates at a peripheral velocity of 60 ft/min (0.3 m/s).

The preferred temperature range for an RBC system is 55 to 90°F (13 to 32°C). Thus, in colder climates, the units are enclosed for climatic control.

It is good practice to design parallel trains of at least two units in four or more stages. Figure 6.61 illustrates an RBC system used as the biological treatment process. Also, RBCs can be used in parallel or in series with existing trickling filters or activated sludge systems to provide higher and more reliable treatment levels.

Design standards and operating experience for RBCs are relatively limited compared to activated sludge and trickling filter processes. Thus, designs are usually based on data from pilot plant testing or from full-scale installations for treatment of similar wastewater characteristics. Some of the more common RBC design features are listed in Table 6.61.

ANAEROBIC TREATMENT

Anaerobic treatment is a biological process in which microorganisms convert organic compounds to methane, carbon dioxide, cellular materials, and other organic compounds. Initially, anaerobic treatment was applied to feedstocks, such as municipal wastewater sludges and meat-packing wastes. However, anaerobic treatment has recently been used for high-strength organic waste because of its potential for producing energy and a lower sludge growth rate. A list of industries in which anaerobic treatment has been applied is given in Table 6.62.

TABLE 6.61 Rotating Biological Contactor Design Data

Parameter	Typical value
Hydraulic loading per unit of medium surface area	0.75–2 gal/day/ft ² (0.03–0.08 m ³ /day/m ²)
Basin volume per unit of media surface area	0.12 gal/ft ² (0.005 m ³ /m ²)
Recycle	None
Sloughing	Continuous
Waste sludge	0.4–0.5 lb/lb (0.4–0.5 kg/kg) BOD ₅ removed
Clarifier surface overflow rate	800 gal/day/ft ² (32 m ³ /day/m ²)
BOD removal	80–90%

TABLE 6.62 Industrial Wastes Treated by Anaerobic Treatment

Beverage manufacturing	Poultry processing
Cheese whey	Pulp mill evaporate
Dairy	Organic chemicals
Distillery	Starch
Meat packing	Sugar beet
Petrochemicals	Sugar cane
Pharmaceutical	Tannery
Potato processing	Wool and textile

Anaerobic treatment converts organic wastes to methane and carbon dioxide in the absence of air. Anaerobic decomposition is a three-stage reaction: (1) hydrolysis of suspended organic solids into soluble organic compounds; (2) acetogenesis, or conversion of soluble organics to volatile fatty acids (mostly acetic acid); and (3) methanogenesis, or conversion of the volatile fatty acids into methane. Hydrolysis does not occur in all anaerobic treatment systems. However, for certain wastes, such as municipal wastewater treatment sludges, waste pharmaceutical cells, and some food waste, hydrolysis is an important reaction. In some cases, the hydrolysis step can be the rate-limiting reaction.

The second step, acetogenesis, is accomplished by a general class of microorganisms known as the volatile acid formers. The products of the acid-forming reaction are the acid salts of short-chain fatty acids, primarily acetic, propionic, and butyric acid. (Their corresponding acid salts are acetates, propionates, and butyrates.) These are measured by volatile acids analysis. Other products of the acid-forming reaction are carbon dioxide and new bacterial cells. The acid-forming steps can proceed over a broad range of environmental conditions, such as pH and temperature.

The third and final step in the series of anaerobic digestion reactions is the formation of methane and carbon dioxide. For most organic wastes, methane and carbon dioxide are formed from the decomposition of acetic acid. Methane, however, can be produced by decomposition of propionic and other volatile acids. The methane-forming bacteria are more sensitive to environmental conditions, such as pH, temperature, and inhibitory compounds, and are more fragile than the acid-forming bacteria. Furthermore, the growth rate of methanogenic bacteria is lower than the acid-forming bacteria. Therefore, it takes more time for the methane bacteria to recover from inhibition or shock conditions than it does for the acid-forming bacteria.

Anaerobic treatment can be defined biochemically as the conversion of organic compounds into carbon dioxide, methane, and microbial cells. For anaerobic treatment of organic nitrogen compounds such as domestic sludge, the end products will also include ammonia. The portion of the organic compound that is converted is termed the net cell yield. The cell yield will range from approximately 5% for fatty acids to approximately 20% for carbohydrates.

Due to low-yield coefficients, methane and carbon dioxide production account for the majority of the influent organics, i.e., COD, processed. The theoretical methane yield, with a bacterial yield coefficient of zero, is 5.4 ft³/lb (0.370 m³/kg) COD degraded. Since carbon dioxide is partially soluble in the wastewater, the gas production will be dependent on pH and alkalinity. Gas production rates applied to the influent COD must also consider the nonbiodegradable fraction of the COD. Since COD reduction is stoichiometrically related to methane production, the anaerobic digester performance can be easily calculated.

pH and Alkalinity Requirements. The optimum pH range for methane fermentation is between 7 and 8, but the methane bacteria generally are not harmed unless the pH drops below about 6.0. The lower the pH is and the longer the pH is at a low value, the more likely is a damaging upset. For this reason, it is recommended that the pH be maintained above 6.5 for anaerobic reactors. When the bicarbonate alkalinity drops below about 500 mg/L, and with the normal percentage of carbon dioxide (approximately 38%) in the reac-

tor gas, the pH will drop close to 6.0. Anaerobic treatment of organic nitrogen-containing wastes produces ammonium and bicarbonate which create alkalinity for the treatment system. The ammonium cation (NH_4^+) is in equilibrium with bicarbonate; without ammonium in solution, carbon dioxide would leave the solution as carbonate ion (CO_3^{2-})

Nutrient Requirements. Anaerobic microorganisms require nutrients to sustain growth. The nutrient requirements for anaerobic microorganisms are different from aerobic requirements because the cell yield is lower. Furthermore, the nitrogen and phosphorus requirements for anaerobic treatment are typically only 20% of those for aerobic treatment. Whereas a COD:N:P ratio of 100:5:1 is typical of aerobic treatment, a COD:N:P ratio of 100:1:0.2 is typical of anaerobic growth. Specific nitrogen and phosphorus requirements will depend upon the nature of the organic compounds to be biodegraded and the sludge age of the treatment system.

In addition to nitrogen and phosphorus, anaerobic microorganisms require other nutrients for growth. These nutrients (sulfur, iron, calcium, magnesium, sodium, and potassium) are typically present in sufficient concentrations in many industrial wastes. However, for some wastes, they can be deficient and may need to be added.

Temperature. As with other biological processes, anaerobic bacteria can be divided into three temperature ranges. In general, the higher the temperature of the anaerobic system is, the faster the substrate removal rate and the faster the cell decay rate. Normally, anaerobic reactors are operated in the mesophilic range, 77 to 104°F (25 to 40°C), or the thermophilic range, 122 to 158°F (50 to 70°C). Thermophilic systems will require a smaller reactor than a mesophilic system. However, because of their low cell yield, thermophilic processes are very slow to start up and cannot tolerate variations in loading, variations in substrate characteristics, or toxic compounds.

Toxicity. Methanogens are commonly considered to be the most sensitive to toxicity of all the microorganisms involved in the anaerobic treatment of organic wastewaters. However, methane bacteria, like most microorganisms, can tolerate a wide variety of toxicants. In addition, many toxic compounds are biodegraded in anaerobic reactors so that the methanogens are not affected by them. Acclimation to toxic compounds and reversibility of toxic effects are known to occur. The toxicity of a compound is related to its concentration, the length of exposure time, and the normal background concentration.

The toxicity of ions is generally attributed to the cation rather than the anion. The toxicity and/or inhibition concentrations of some common cations are presented in Table 6.63. Many of these cations are stimulatory at low concentrations but are toxic at higher concentrations.

Toxicity can be prevented by addition of other cations that act as cation antagonists. For example, the tox-

TABLE 6.63 Inhibition Concentrations of Various Ions (64)

Species	Stimulatory, mg/L	Moderately inhibitory, mg/L	Strongly inhibitory, mg/L
Sodium	100–200	3500–5500	8,000
Potassium	200–400	2500–4500	12,000
Calcium	100–200	2500–4500	8,000
Magnesium	75–150	1000–1500	3,000
Ammonia	—	1500–3000	3,000
Hydrogen sulfide*	—	—	300

*As soluble hydrogen sulfide.

ic effect of sodium can be reduced by adding potassium and further reduced by adding calcium. Antagonists are best added as a chloride salt. If antagonists are not available or are too costly, the best method to prevent toxicity may be dilution.

Hydrogen sulfide has been recognized to be toxic to anaerobic microorganisms, especially methanogens. Under strictly anaerobic conditions, the sulfate ion (SO_4^{2-}) is biochemically reduced to the sulfide ion (S^{2-}), which establishes an equilibrium with the various forms of hydrogen sulfide (H_2S , HS^- , and S^{2-}). The toxic concentration of total dissolved sulfide in anaerobic digestion has been reported to be 200 to 300 mg/L.

The toxicity concentration will depend upon the pH of the wastewater. At higher pH values, the less toxic sulfide form, HS^- , predominates, and at lower pH values, the more toxic form, H_2S , predominates. At a neutral pH, there will be equal concentrations of both species.

To prevent toxicity by hydrogen sulfide, the following alternatives should be considered:

1. Prevent hydrogen sulfide or sulfate from being introduced into the wastewater.
2. Dilute the wastewater below the toxic threshold.
3. Form an insoluble complex or precipitate to remove the hydrogen sulfide from the anaerobic reactor by adding iron or aluminum salts.
4. Purge the hydrogen sulfide from the wastewater.

Reactor Type

Anaerobic microorganisms can either be described as suspended growth or fixed film. Therefore, anaerobic reactors are configured to optimize the growth of these organisms.

Suspended-Growth Systems. The three primary types of suspended-growth reactors are the anaerobic lagoon, the anaerobic contact system, and the anaerobic upflow sludge blanket.

Anaerobic Lagoon. The anaerobic lagoon process is generally used where close control of the process is not required, and a high degree of treatment is unnecessary. Temperature is not generally controlled, and a clarifier may or may not be used depending on the retention time in the basin and the final effluent quality required. Sludge recycle from the clarifier to the anaerobic lagoon is seldom used. This treatment process is often used for pretreatment by industry (meat packing or food processing, for example) prior to discharge to a municipal or additional on-site treatment system. Typical design data are presented in Table 6.64. Organic removal, as measured by BOD_5 , may range from 40 to 75%, depending on the type of waste and the temperature. These lagoons are sized based on hydraulic detention time or alternatively on volumetric organic loading. Odors may be an operating problem.

Anaerobic Contact. The anaerobic contact process (see Figure 6.62) is a suspended-growth system which

TABLE 6.64 Anaerobic Lagoon Design Data

Parameter	Typical value	Remarks
Detention time	15–30 days	Range, 1–50 days
Depth	10–12 ft (3–3.6 m)	Range, 8–20 ft (2.4–6.1 m)
Organic loading	3–15 lb $\text{BOD}_5/1000 \text{ ft}^3/\text{day}$ (0.05–0.25 $\text{kg}/\text{m}^3/\text{day}$)	Widely varying due to wastewater characteristics
pH	7	Range, 6.8–7.2
Temperature	85 to 95°F (30–35°C)	Range: 45 to 120°F (7–95°C)
Number of units	2 or more	Parallel operation
BOD_5 removal	70–80%	Range, 50–90%

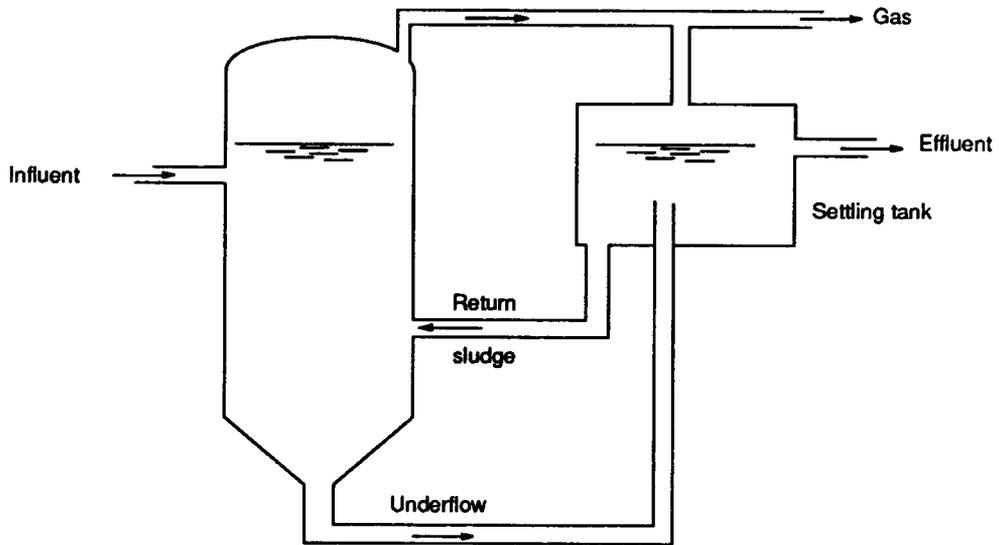


FIGURE 6.62 Anaerobic contact reactor.

employs a gravity solids separation unit for recycle of biological solids just as in the activated sludge process. However, the biological solids from the anaerobic reactor are actively producing gas as they leave the reactor. Therefore, a vacuum degasifier is utilized to remove excess gas which would otherwise interfere with the solids settling process. Proper control of the system depends on effective settling and solids recycle.

Anaerobic Upflow Blanket. The third type of anaerobic suspended-growth system is similar in principle to the anaerobic contact process. The upflow blanket process utilizes a combined anaerobic reactor–solids separation unit from which clarified liquid (effluent) overflows the reactors (see Figure 6.63). The contact time in the reactor is set by the flow and volume. Solids concentration is controlled through periodic wasting of biological solids, and clarification is maintained by settling the upward flow velocity in the reactor to less than the settling rate of the biomass. The solids control technique is the most critical aspect of the process. The design and operation of the system depends on maintaining a granular, flocculant biomass which will provide the necessary organic removal while at the same time allowing solids and liquid separation and growth control. The other aspects of the system, such as organic loading, temperature, and pH requirements, are similar to the anaerobic contact process.

Fixed-Growth Systems. Because of the difficulties associated with clarification in suspended growth reactors, fixed-growth systems are often utilized for treatment of wastes that are free from suspended solids. The general types are the upflow, downflow, and fluidized bed.

Anaerobic Upflow Filter. The anaerobic upflow filter reactor contains either plastic, wood, or similar materials that provide the sites for the attached-growth microorganisms. The media can either be randomly dumped or stacked. Some examples of randomly dumped media include pall rings and coarse aggregate. Examples of stacked media included inclined or vertical corrugated plates. As shown in Figure 6.64, the upflow reactor generally contains an open space at the bottom where solids can be collected and removed from the system.

The reactor oftentimes includes a system to inject a pressurized gas, such as nitrogen or the collected reactor gas, into the media to agitate the bed and promote sloughing of solids or unplugging the bed. The reac-

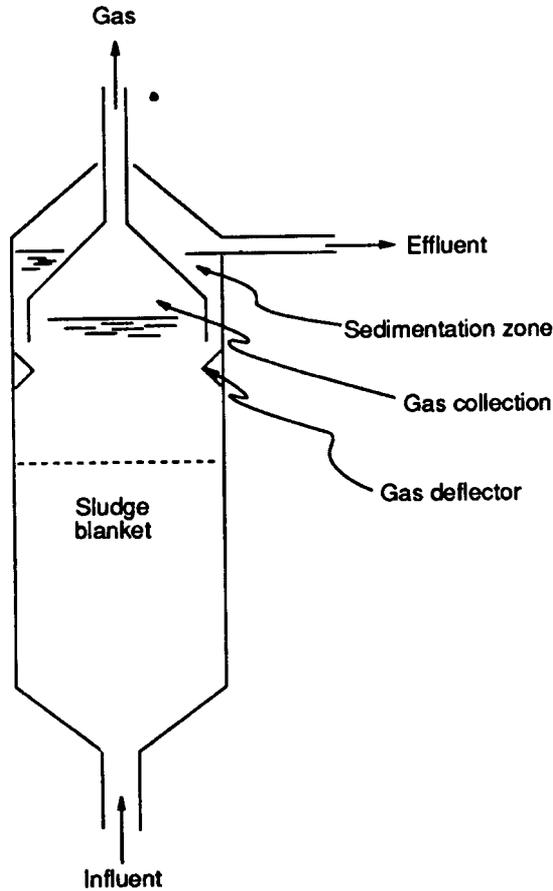


FIGURE 6.63 Anaerobic upflow blanket reactor.

tor overflow may be recycled for pH control. The recycle rate will depend upon the influent loading; recycle rates up to 10 times the influent rate have been designed.

Anaerobic Downflow Filter. The anaerobic downflow filter reactor is similar to the upflow reactor in many respects. However, downflow reactors that contain stacked media, such as vertical corrugated plates, can treat wastewaters high in suspended solids because the solids are flushed through the media. The effluent (under flow) from the downflow media is recycled to the top of the reactor. Figure 6.65 illustrates this system.

Anaerobic Fluidized Bed. Another type of fixed-film anaerobic system is the fluidized bed process. The anaerobic reactor is operated in an upflow mode with a high effluent recycle rate 10 to 15 gpm/ft² (575 to 975 m³/m²-d) and uses sand as the inert media for attached microbial growth. A schematic of a typical system is shown in Figure 6.66. The advantage of this design is the high surface area per unit volume of reactor available for biological solids growth. This has been shown to result in allowable loading rates of 0.31 to 1.25 COD/day/ft³ (5 to 20 kg COD/day/m³) on many wastes, and this corresponds to a smaller reactor for the same removal as compared to other anaerobic process configurations. The disadvantage of the fluidized bed system is the energy requirements to fluidize the bed. An important process control of the sys-

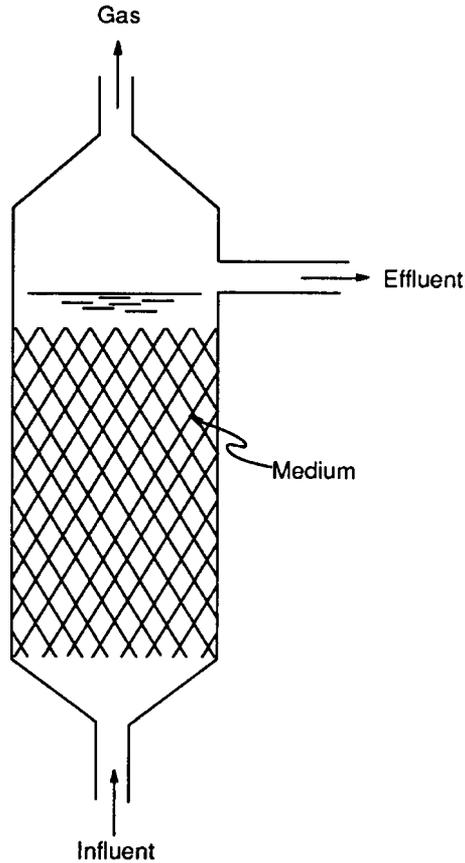


FIGURE 6.64 Anaerobic upflow filter reactor.

tem is the separation of the excess biomass from the sand for disposal of the solids and recycle or reuse of the sand.

Combination System. Anaerobic treatment can also be accomplished in a reactor that utilizes both suspended and fixed-growth organisms. These reactors are upflow reactors having an open space at the bottom to promote suspended growth. The upper zone contains a cross-flow or a similar type of media. The purpose of this zone is to promote the growth of attached microorganisms and to provide an environment whereby the suspended growth microorganisms will settle to the lower zone. A schematic is shown in Figure 6.67.

Design Considerations. Many models have been developed to describe the anaerobic treatment. The Monod-derived kinetic description, relating limiting substrate concentration to microbial growth rate, has been the basis of most anaerobic digestion models. Other researchers have used first-order models to describe anaerobic treatment. The best model used to describe anaerobic treatment kinetics should be based upon data collected from wastewater treatability studies.

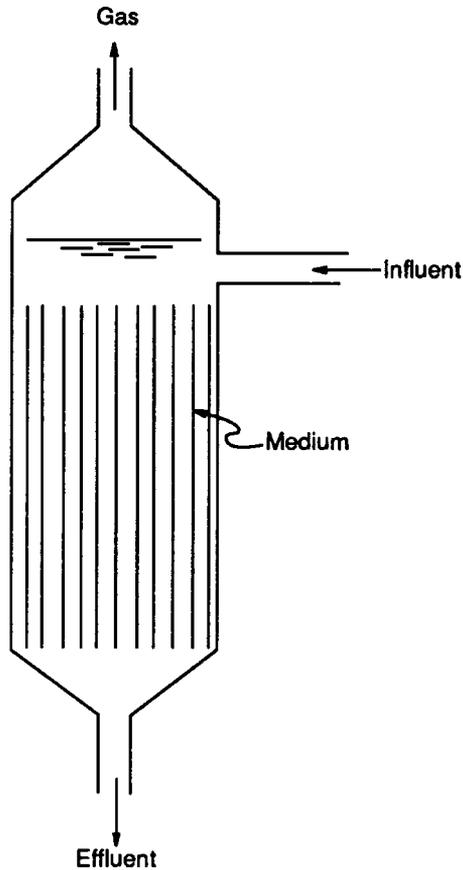


FIGURE 6.65 Anaerobic downflow filter reactor.

Because anaerobic treatability studies can be costly as well as time consuming, the design of anaerobic treatment systems are oftentimes based upon organic loading rate as measured by COD or ROD5. The COD loading rate is defined as mass of COD applied to the reactor volume per unit time. Volume is considered when determining the reactor volume.

A summary of typical anaerobic treatment design data using COD loading rates is shown in Table 6.65. Comparable removal efficiencies may be obtained with the different reactor configurations. The wide range in the organic loading rate that can be found for anaerobic reactors is caused by the type of waste treated, the success of maintaining biological solids in the system, and the temperature of the reaction.

AQUATIC PLANT SYSTEMS

Aquatic plant treatment systems consist of one or more shallow ponds in which one or more species of tolerant vascular plants, characterized as free floating or suspended root. The shallower depths and the presence

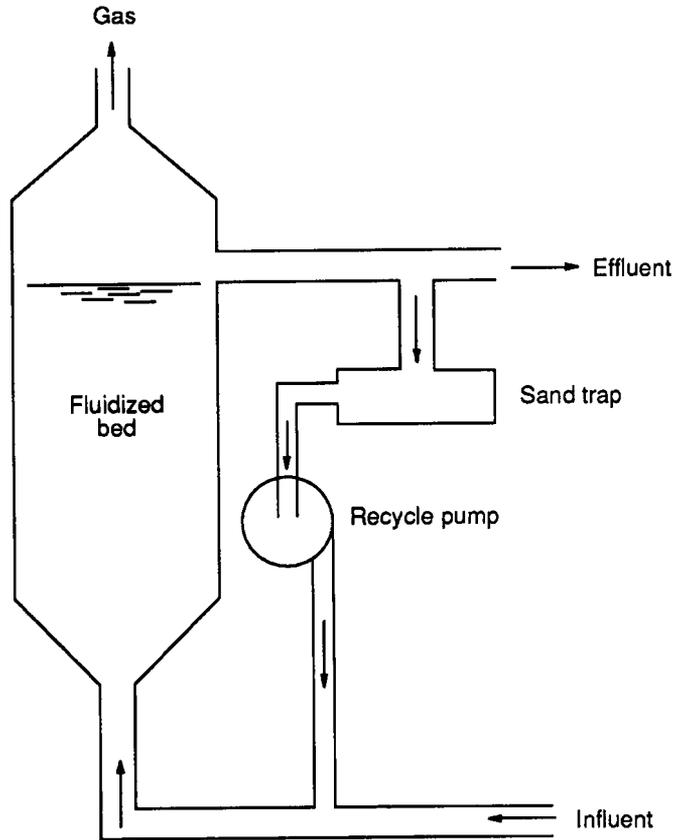


FIGURE 6.66 Anaerobic fluidized bed reactor.

TABLE 6.65 Anaerobic Treatment Design Data

Anaerobic reactor type	Typical COD loading rate	
	kg/m ³ /day	lb/100 ft ³ /day
Lagoon	1–2	60–120
Contact	1–5	60–300
Upflow blanket	5–10	300–625
Upflow filter	1–10	60–625
Downflow filter	5–15	300–950
Fluidized bed	5–30	300–1875
Combination	5–20	300–1250

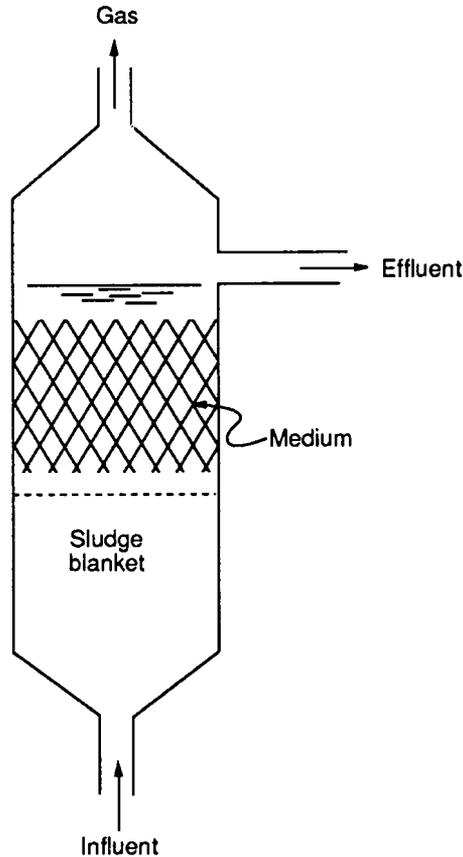


FIGURE 6.67 Combination anaerobic reactor.

of aquatic macrophytes in place of algae are the major differences between aquatic plant treatment systems and stabilization ponds. The presence of plants is of great practical significance because the effluent from aquatic systems is of higher quality than the effluent from stabilization pond systems for equivalent or shorter detention times (97–103).

In aquatic systems, wastewater is treated principally by bacterial metabolism and physical sedimentation, as happens in conventional trickling filter systems. The aquatic plants themselves provide very little actual treatment of the wastewater; their function is to provide components of the aquatic environment that improve the wastewater treatment capability and/or reliability of that environment.

Aquatic plants have the same basic nutritional requirements as plants growing on land and are influenced by many of the same environmental factors. Aquatic plant treatment systems are designed to achieve a specific wastewater treatment goal and are divided into two categories:

- Floating aquatic plants. The most common plants include water hyacinth, duckweed, and pennywort. The system is distinguished by the ability of these plants to derive their carbon dioxide and oxygen needs directly from the atmosphere. Free floating and suspended root plants receive their mineral nutrients from the water.

- Submerged aquatic plants. These systems may include plants such as waterweed, water milfoil, and watercress and are distinguished by the ability of these plants to absorb oxygen, carbon dioxide, and minerals from the water column. Submerged plants are inhibited by high turbidity in the water because their photosynthetic parts are below the water

Floating Plant Systems

Floating plants have their photosynthetic parts at or just above the water surface, with roots extending down into the water column. In photosynthesis, floating aquatic plants use atmospheric oxygen and carbon dioxide. Nutrients are taken up from the water column through the roots. These roots are an excellent medium for the filtration/adsorption of suspended solids and growth of bacteria. Root development (growth rate) is a function of nutrient availability in the water and nutrient demand of the plant. Thus, in practice, the density and depth of treatment medium (plant roots) will be affected by wastewater quality/pretreatment and factors affecting plant growth rate, such as temperature and harvesting.

With floating plants, the penetration of sunlight into water is reduced and the transfer of gas between water and atmosphere is restricted. Consequently, while floating plants tend to keep the wastewater nearly free of algae, anoxic or anaerobic conditions may develop depending on design parameters, such as organic and hydraulic loading rates, and the species and coverage density of the selected floating plants.

Floating Plants. Water hyacinth and duckweed wastewater treatment systems are most common, and applications include extensive use in improving the quality of waste stabilization pond effluents and as a component in advanced wastewater treatment systems. The major characteristics of water hyacinths that make them an attractive biological support media for bacteria are their extensive root system and rapid growth rate. Duckweed systems may be used alone in a treatment system and in combination with water hyacinths in a polyculture system (97, 101).

Water Hyacinths. Water hyacinth (*Eichhornia crassipes*) is a perennial, freshwater aquatic vascular plant with rounded, upright, shiny green leaves and spikes of lavender flowers. The petioles of the plant are spongy, with many air spaces that contribute to the buoyancy of the hyacinth plant. When grown in wastewater, individual plants range from 20 to 47 in (0.5 to 1.2 m) from the top of the flower to the root tips. The plants spread laterally until the water surface is covered and then the vertical growth is increased. Hyacinths are very productive photosynthetic plants. These attributes are advantageous when used in a wastewater treatment system. However, rapid growth of water hyacinths had caused nuisance problems in many warm-water waterways until biological control agents were found to reduce their populations to manageable levels.

Water hyacinths reproduce primarily by vegetative propagation, but seeds may be a major source of reinfestation once the parent plants have been removed. Water hyacinths also develop a large canopy, which may provide a good competitive edge over other floating aquatic plants growing in the same system. In fact, water hyacinths may be used for abating algal bloom problems in waste stabilization ponds experiencing effluent problems with high suspended solids from algae.

Duckweed. Duckweeds are small, green freshwater plants with leaflike fronds from one to a few millimeters in width. *Lemna* spp. and *Spirodela* spp. have a short root, usually less than 10 mm (0.4 in) in length. Duckweeds are the smallest and the simplest of the flowering plants and have one of the fastest reproduction rates. A small cell in the frond divides and produces a new frond; each frond is capable of producing at least 10–20 more during its life cycle. *Lemna* sp. grown in wastewater effluent at 80 °F (27 °C) doubles in frond numbers, and therefore in area covered, every four days.

Duckweed systems are developed by following the conventional design procedures for facultative ponds. Effluent from a duckweed covered system should exceed performance of conventional facultative ponds for BOD₅, suspended solids and nitrogen removal. The effluent from a duckweed system is likely to be anaero-

bic and post-aeration may be necessary. The advantage of duckweed systems over a similar additional facultative pond cell is lower algae concentrations in the effluent.

Temperature. As in other biological processes, growth rates in aquatic plant systems depend on temperature. Both air and water temperatures are important in assessing plant vitality. Widespread use of water hyacinth systems is limited by the plant's sensitivity to cold temperatures. Duckweed is more cold tolerant than water hyacinths and may be grown at temperatures as low as 45 °F (7 °C).

The optimum water temperature for water hyacinth growth is 70–85 °F (21–30 °C). In a flowing system, hyacinths may tolerate air temperatures as low 32 °F (0 °C) for short periods of time. As temperatures drop or remain near freezing, leaves are destroyed and the plant is finally killed. If a water hyacinth system is to be used in a colder climate, housing the system in a greenhouse would be necessary to maintain the temperature in the optimum range. On the other hand, air temperatures above 85 °F (30 °C) will begin to burn/breakdown hyacinth leaves.

Wind. Water hyacinth and especially duckweed are sensitive to wind and may be blown in drifts to the leeward side of the pond. Redistribution of the plants requires manual labor. If drifts are not redistributed, decreased treatment efficiency may result, because algae will develop in the system and elevated pH values. Water hyacinth are sensitive to pH above the 7.5–8 range. Also, piles of decomposing plants can result in the production of odors. Major disadvantages of duckweed are their shallow root systems and sensitivity to wind.

System Configurations. Shallow, rectangular basins with a high length-to-width ratio are usually designed for aquatic treatment systems to reduce the potential for short-circuiting and to simplify harvesting operations. Small basins or surface baffles are needed to minimize distribution of plants by wind action. The use of baffles and influent distribution manifolds helps to maximize the detention time. Influent manifolds and multiple inlet/step feed systems also can be used effectively for recycling treated effluent to reduce the influent concentrations of wastewater constituents. An effluent manifold across the basin will maintain a low velocity into the manifold, which serves to maintain quiescent conditions near the outlet. If variable operation depths are planned, it should be possible to remove effluent at a depth of 1 ft (0.3 m) below the most shallow operating depth.

Design Considerations. Design parameters used to size aquatic systems include organic loading rate, hydraulic loading rate, water depth, detention time, and harvesting/vegetation management. Design parameters based on the required level of treatment have been summarized in Table 6.66. Design criteria for effluent polishing using duckweed in facultative ponds are summarized in Table 6.67 (97–101).

Organic Loading Rates. Organic loading is a key parameter in the design and operation of water hyacinth systems, which represent the majority of aquatic plant treatment systems. Three types of hyacinth systems can be described, based on the level of dissolved oxygen and the method of aerating the pond:

- Aerobic Nonaerated, for Secondary Treatment of Screened or Settled Wastewater. This type of system is most common.
- Aerobic Nonaerated, for Nutrient Removal of Secondary Treated Wastewater. These systems apply well to locations where no mosquitoes or odors can be tolerated. With aeration, higher organic loadings are possible and land requirements are reduced.
- Aerobic Aerated, for Secondary Treatment of Screened or Settled Wastewater.

Hydraulic Loading Rate. The hydraulic loading rates applied to water hyacinth facilities vary widely and usually are governed by the organic loading rate. For secondary treatment objectives, the hydraulic loading rate is typically between 20,000–65,000 gpd/ac (200 and 600 m³/ha-d). For advanced secondary treatment

TABLE 6.66 Design Criteria for Water Hyacinth Systems (97)

Factor	Type of Water Hyacinth System		
	Aerobic nonaerated	Aerobic nonaerated	Aerobic aerated
Influent wastewater	Screened or settled	Secondary effluent	Screened or settled
Influent BOD ₅ , mg/L	130–180	30	130–180
Expected Effluent			
BOD ₅ , mg/L	>30	>10	>15
SS, mg/L	>30	>10	>15
TN, mg/L	>15	>5	>15
BOD ₅ loading, lb/acre-d	35–70	9–35	135–270
(kg/ha-d)	(40–80)	(10–40)	(150–300)
Hydraulic loading, gal/acre-d	>21,000	>85,000	58,000–107,000
(m ³ /ha-d)	(>200)	(>800)	(550–1000)
Water depth, ft (m)	1.6–2.6 (0.5–0.8)	2–3 (0.6–0.9)	3–4.5 (0.9–1.4)
Detention time, days	10–36	6–18	4–8

with supplemental aeration, hydraulic loading rates may approach 110,000 gpd/ac (1,000 m³/ha-d). However, organic loading rates generally will control the actual hydraulic loading.

Water Depth. The range of depths for water hyacinth ponds is 1.5–4.5 ft (0.5–1.4 m) with most applications at depths of 2–3.5 ft (0.6–1 m). The critical concern is to provide adequate depth for the root system to penetrate through most of the liquid flowing through the hyacinth pond. A greater depth is sometimes desirable for the final cell in a series of hyacinth ponds, since the hyacinth roots will be longer when fewer nutrients are present in the water.

Vegetation Management. Vegetation management encompasses the harvesting of aquatic plants and depends on water quality objectives for the system, the growth rates of the plants, and the effects of predators, such as weevils. Harvesting of aquatic plants is needed to maintain a crop with high metabolic uptake of nutrients. Nitrogen and phosphorus removal by the plants is achieved only with frequent harvesting.

Plant growth may be described as a percentage of pond surface covered over a period or by plant density, expressed in units of wet plant mass per unit of surface area. Loosely packed water hyacinths can cover the water surface at low plant densities, 2–3 lb/ft² (10–15 kg/m²), and avoid problems associated with algae pro-

TABLE 6.67 Design Criteria for Effluent Polishing with Duckweed Systems (97)

Factor	Secondary treatment
Influent wastewater	Facultative pond effluent
Expected effluent	
BOD ₅ , mg/L	>10
SS, mg/L	>10
TN, mg/L	>5
BOD ₅ loading, lbs/acre-d (kg/ha-d)	20–25 (22–28)
Hydraulic loading, gal/acre-d (m ³ /ha-d)	>55 (>50)
Water depth, ft (m)	5–7 (1.5–2)
Detention time, days	15–25
Water temperature, °F (°C)	>45 (>7)

duction. Higher densities of up to 10 lb/ft² (50 kg/m²) may occur before growth is completely inhibited and the hyacinth foliage dies and adds to the pond loading. Thus, harvesting should begin before reaching the higher densities and should continue until densities approach of 3–5 lb/ft² (15–25 kg/m²).

Harvested plants are typically dried and landfilled; the drying process may be a source of significant odors. Windrow composting may be used to allow some drying and stabilization before land application, such as a slow release fertilizer for pasture land. Water hyacinth may be chopped before anaerobic digestion to produce methane, blended with other material to form animal feed, and the sugars fermented to produce alcohol; these techniques may be used in a large-scale operation to receive some by-product value.

Nutrient Removal. Nitrogen removal by plant uptake can only be accomplished if the plants are harvested. Nitrate, produced through nitrification, is removed by denitrification and plant uptake, as is organic nitrogen. In the past, there has been some question as to whether nitrification or plant uptake is the principal ammonia–nitrogen conversion mechanism that ultimately leads to nitrogen removal. Typical nitrogen loading rates are 400–800 lb/ac-d (450–900 kg/ha-d).

Phosphorus removal from aquatic macrophyte systems results from plant uptake, microbial immobilization into detritus plant tissue, retention by the underlying sediments, and precipitation in the water column. Since phosphorus is retained by the system, the ultimate removal from the system is achieved by harvesting the plants and dredging the sediments. Typical phosphorus loading rates may range up to 100–200 lb/ac-d (115–230 kg/ha-d). Particularly in cases where phosphorus removal occurs ahead of the plant system, phosphorus may become limiting.

Submerged Plant Systems

Submerged plants are either suspended in the column or rooted in the bottom sediments. Typically, their photosynthetic parts are in the water column. The potential for use of submerged plants for polishing of effluent seems, at least theoretically, an attractive option. The tendency of these plants to be shaded out by algal growths and to be killed or severely harmed by anaerobic conditions limits their practical usefulness.

An advantage of submerged plant systems is enhanced removal of organic nitrogen. Ammonia removal from the water column is related to the photosynthetic processes, which removes carbon dioxide from the water (unlike hyacinths), thus raising the pH and driving ammonia to the gaseous form that can diffuse into the atmosphere. Ammonia gas is the most toxic form of nitrogen for fish. This mechanism is of some concern for healthy populations of mosquito fish, which are generally encouraged in aquatic treatment ponds for mosquito control.

WETLAND TREATMENT SYSTEMS

Constructed wetland wastewater treatment systems are relatively easy systems to operate and maintain and extremely energy efficient when compared to conventional mechanical systems. Constructed wetland systems, including restoration of natural wetlands, enhance the aesthetic value of the area and provide educational value as a nature study area. Plant communities, hydroperiod, soils, and other factors interact to establish the uniqueness of individual wetland systems (97, 102, 104–107).

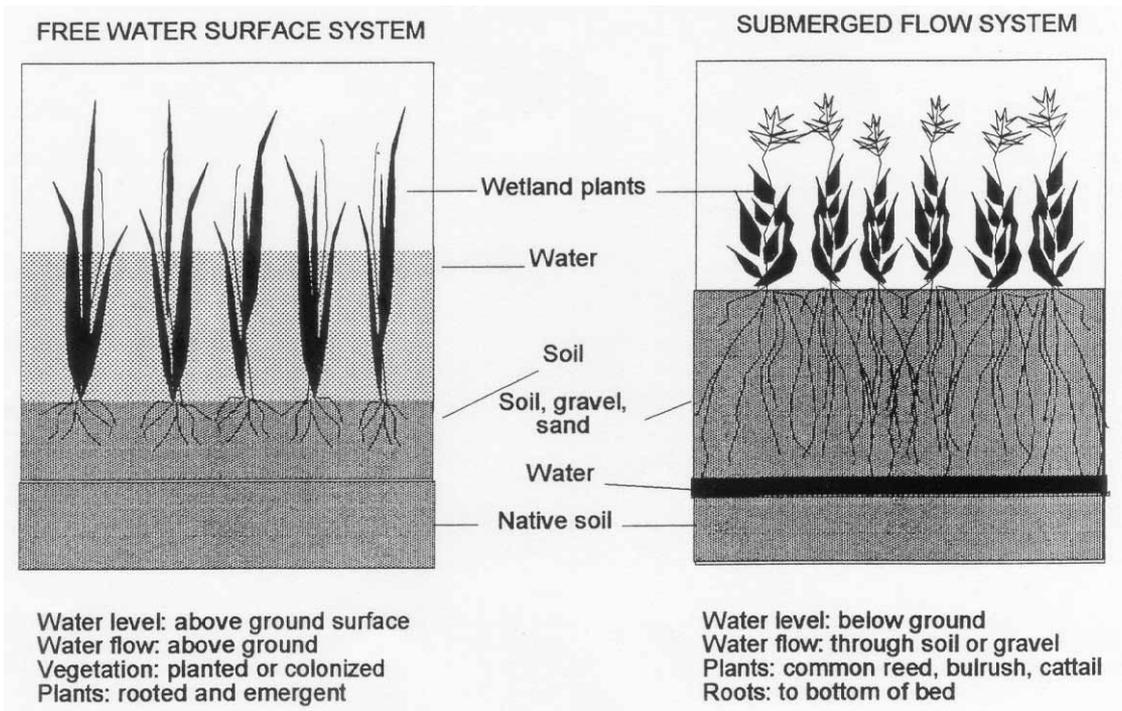
Over the years, wetlands have provided wastewater treatment for pollutants from sources ranging from rainfall runoff in forested areas to intentional discharges of community sewage. Much of the wastewater treatment in a wetland occurs by means of bacterial metabolism and physical sedimentation. The study of wetland functions and values (Table 6.68) is the essential first step in understanding how wetlands perform as a wastewater treatment facility.

TABLE 6.68 Wetland Functions and Values

Flood water storage	Nutrient removal/retention
Flooding reduction	Chemical and nutrient sorption
Erosion control	Food chain support
Sediment stabilization	Fish and wildlife habitat
Sediment/toxicant retention	Migratory waterfowl usage
Groundwater recharge/dischage	Recreation
Natural water purification	Uniqueness/heritage values

Constructed wetland treatment systems may range from the creation of a marsh in a natural setting to extensive construction involving considerable earth moving and creation of impermeable barriers. These wetland systems are classified by the predominate flow regimes, above and below ground, as shown in Figure 6.68:

- FWS—Free water surface system with emergent vegetation consisting of basins or channels, barriers to seepage, relatively shallow water depths, and low flow velocities.
- SFS—Subsurface flow systems with emergent vegetation consisting of all subsurface flow via a trench or bed underdrain network, barriers to seepage, and very low flow velocities. SFS systems also are called vegetated submerged bed (VSB) systems.

**FIGURE 6.68** Wetland treatment system flow regimes.

The layout of the constructed wetland treatment system varies by site. In a FWS system, multiple cells are developed in both parallel and series configurations. Long length-to-width ratios may be used to enhance plug flow characteristic, and prevailing topographic features may be used or modified to have a more natural effect. The configuration of a SFS wetland system includes multiple beds, arranged in parallel. Since very low velocities are maintained in these systems, short length-to-width ratios are necessary (97, 98, 107).

Siting Considerations

Essential considerations for siting potential wetland treatment systems include:

- Proximity to pretreatment facility and an acceptable point of discharge
- Potential for restoration of a degraded wetland habitat
- Identification of vegetative community or cover type
- Proximity to or potential to provide a habitat for listed species
- Study of existing surface and groundwater hydrology
- Classification of soils
- Delineation of topographic features

Prior to initiating field studies, potential sites may be identified and screened using published data. Typical information gathered includes aerial photographs, Soil Conservation Service (SCS) soils maps, Federal Emergency Management Agency (FEMA) flood studies, U.S. Geological Survey (USGS) topographic maps, and local land use plans (97, 98, 102).

Hydrology. Hydrology is the most important physical feature controlling plant composition. The hydroperiod, which is the relationship of water depth and period of inundation over an annual cycle, is critical for successful development of a wetland treatment site. Performance is dependent on the ability to retain water and control hydraulics through the system. Attention should focus on the site's runoff, runoff, and infiltration characteristics and its relation to both surface water and groundwater resources. Annual local climate factors (precipitation, evaporation, and temperature) and their influences on hydrology are considered in the final wetland system design.

Soils. Soil is one of the most important physical components of both natural and constructed wetlands. Mineral composition, organic matter, moisture regime, temperature regime, chemistry, and depth interact greatly to influence the types of plants that will succeed on the surface and the microorganisms that will live in the soil phase. For the wetland treatment system, soil types (mineral or organic, Table 6.69) and characteristics (permeability and drainage) are influential in development of the site. Permeability is especially important in that it reflects the ease with which water and air move through the soil. Infiltration into and drainage within the soil are, or can be, affected by subsurface impervious layers.

Since surface water level control is imperative, FWS wetland systems require soils with very low or no permeability, unless there is a natural impervious stratum to restrict drainage. When permitting a constructed wetland system, compaction of medium-textured soils, such as a loam or silt loam, or installation of a clay or synthetic impermeable barrier below coarse-textured (sandy) soils may be required to isolate and maintain the system's water controls. Otherwise, accretion of organic matter will significantly reduce vertical permeability over time.

For a SFS wetland, coarse-textured (sandy), highly permeable soils are desirable, but there must be a natural, or installed, impervious subsurface barrier to restrict vertical wastewater migration. The barrier serves the primary purpose of maintaining hydrology for the emergent wetland vegetation. In areas with shallow groundwater, the impervious barrier also protects this water resource from contamination.

Soil type influences the wastewater treatment performance. For example, clays (with their sorption capa-

TABLE 6.69 Comparison of Mineral and Organic Soil Characteristics (97, 101, 103)

Attribute	Mineral soil ¹	Organic soil ²
Density	High	Low
Drainability	Poor	Moderate
Organic carbon	Less than 12–20%	Greater than 12–20%
Permeability	Low	Moderate
pH	Slightly acid to neutral	Acid, pH less than 6
Sorptive capacity	High	Low

¹For mineral soils with high clay content. Attributes do not apply to those with high sand or gravel content.

²For most organic soils. Attributes do not apply to well-decomposed organic soils, such as peat.

bility) can be important in the removal of phosphorus and metals. Also, mineral soils allow chemical precipitation of phosphorus. The importance of soils solely to complement specific treatment goals may not be cost effective. On the other hand, organic soils offer almost no potential for removal of metals or phosphorus, but enhance nitrogen removal.

Topography. In general, topography is not a siting constraint for constructed FWS wetlands, since soil and drainage factors usually favor farm fields, pastures, or bottom lands with negligible vertical relief relative to alternate sites. However, such sites may sustain dike/berm and other physical damages from uncontrolled external flooding. For gravity flow across a site divided into strata, an elevation difference of about 2 ft (0.7 m) per strata usually is necessary.

For SF5 wetland systems, topography is important. A slope of 1% or more is needed to function as a gravity system. Site preparation often represents the major cost element for either type of constructed wetland system.

Design Considerations

Wetland systems provide wastewater treatment by significantly reducing oxygen demanding substances (BOD and ammonia), suspended solids, nutrients (nitrogen and phosphorus), and other pollutants, such as metals. Primary, secondary, and incremental removal mechanisms include, physical, chemical, and biological operations as well as plant uptake (97, 98, 101, 102, 107–109).

Design development includes the determination of treatment process criteria, definition of physical features of the wetland, and selection of the wetland plant species. Design criteria are analogous to those of conventional (mechanical) facilities, such as BOD and nutrient loading rates, hydraulic loading rates, and detention time. For FWS systems, water depth is an added criterion. An important, early determination is the degree of preapplication treatment needed before application to the wetland system.

Preapplication Treatment. The level of wastewater treatment before application to the wetland system is a major project decision. Pretreatment affects not only the process criteria for the wetland system, but also determines structural and other physical feature needs for the project. Typically, the greater the level of pretreatment for established effluent criteria, the lower the land requirements for the wetland system. Conversely, the capital and operating cost of the total treatment works often increases with expanding preapplication treatment facilities.

Preliminary treatment to remove grit and large solids and primary treatment to remove settleable solids and some BOD should be provided as a minimum. Pretreatment to secondary or even advanced level is most

desirable. These lower-strength wastewaters more closely approach the dilute wastes historically assimilated by natural wetlands as one of their fundamental functional values.

Biochemical Oxygen Demand. Biological metabolism is the primary process for removal of BOD in a wetland system. The diverse microbial populations in the water column will consume the soluble organic waste products. In the soil and sediment (organic) layer, the organics are sorbed in the soil and, later, biologically oxidized into stable end-products. Also, sedimentation in FWS systems and filtration in SFS systems are processes that remove BOD associated with suspended organic materials.

The principal biological activity in FWS wetlands is from algae and/or bacteria, wherein microbial growths are attached to structures, e.g., plant stems and litter. Loading rates may be lower on SFS wetland treatment systems due to the limited availability of oxygen. The plant root systems are key to treatment performance, making the selected plant species especially important.

Selection of the BOD loading rate requires consideration of the organic load/oxygen demand to be applied on an area basis with respect to the oxygen required for microbial metabolism and provided by the atmosphere (FWS systems) and the vegetation. Some BOD may help with denitrification, but organic soils and litter provide the significant carbon source required by denitrification bacteria.

In a FWS system, the treatment process is best characterized as an attached-growth biological reactor under plug flow conditions. This is because the primary treatment mechanism, especially for BOD, is provided by microorganisms attached to the wetland vegetation exposed to a single pass of wastewater flow. Moreover, algae also contribute to the treatment performance as they do in a facultative/oxidation pond system. Likewise, if not filtered by the system, algae will contribute effluent suspended solids and BOD.

Typically, BOD loading rates to constructed wetland treatment systems approach 100 lb/day/acre (112 kg/day/ha) for both FWS or SFS systems. This loading rate is twice the maximum rate typically applied to facultative ponds. The attached microbial population and the oxygen transfer capability of the wetland plants contribute to the success of higher loadings to wetland systems. In practice, with secondary and higher levels of pretreatment, the actual BOD loading rate may be much lower. Other more limiting factors, such as nutrient loadings, dictate land area requirements.

Suspended Solids. Settleable suspended solids not removed by pretreatment are readily removed in the early stages of a FWS by gravity settling in the quiescent, and shallow, areas downstream of the inlet, whether it is of a point or distributed design. Especially, in the case of wastewaters receiving little pretreatment, the distributed inflow design will safeguard against development of adverse and detrimental conditions from sludge buildups. In the SFS wetland, the settleable solids are removed by filtration as the wastewater moves through the underdrain system.

For colloidal suspended solids, the primary removal mechanism is by bacterial metabolism in both types of wetland treatment systems. Some filtering and/or sorption will occur in a SFS system. In the FWS wetland, suspended solids may be produced as algae, which also will produce diurnal swings in dissolved oxygen concentrations.

Nutrients. Wetlands are an effective means to control the discharge of the nutrients nitrogen and phosphorus. The mechanisms for nutrient removal in a wetland system are direct plant uptake, chemical precipitation, uptake by algae and bacteria, soil absorption, nitrification/denitrification, and loss by insect and fish uptake.

The density and types of plants and litter zones in the system will determine the direct and indirect performance rates of these mechanisms. Plant selection should be based on the species suited to the treatment goals and having the ability to establish a stable community that will survive over the spectrum of the system's hydroperiod and other environmental conditions. The selected plant species must coexist and maintain essentially full coverage throughout the year.

Both nitrogen and phosphorus must be considered at the same time. Reduction of ammonia nitrogen by nitrification is expected to occur in the presence of an adequate supply of oxygen. Denitrification for nitro-

gen removal requires a carbon source to complete the conversion. An important consideration in wetland nitrogen removal is plant uptake recycled to sediments. Design must manage sediments to prevent nutrient release to the water column. As a conservative element in the wetland, phosphorus is not removed but is stored in sediments or discharged.

Even though nitrogen-to-phosphorus ratios are important to the functions of a wetland, they may not be listed in permit limits. Furthermore, reported design loading rates vary widely. Nutrient loading rates of 1 to 4 lb/day/acre (1.1 to 4.5 kg/day/ha) for nitrogen and 0.1 to 0.3 lb/day/acre (0.11 to 0.34 kg/day/ha) for phosphorus may be used. Wetlands tend to become nitrogen-limited quickly with the application of wastewaters.

A primary mechanism for removal of nitrogen is through nitrification and subsequent denitrification processes performed by the system's microorganisms. Other mechanisms include nitrogen fixation, mineralization, and chemical transformation.

Under natural conditions, living plants exhibit an average N:P ratio of approximately 7:1 for emergent plant species. When high concentrations of nitrogen and phosphorus are present, the plants may increase their nutrient uptake, by a specific ratio, greater than current needs, known as luxury uptake.

A performance issue is that the macrophytes compete among themselves and with the system's microorganisms for excess nitrogen. Nitrogen removal objectives favor the microorganisms, because they provide the ultimate nitrogen sink—release of nitrogen gas in the denitrification process. Key factors in the wetland system that affect the denitrification rate are pH, temperature, organic carbon, nitrate levels, and the ecological interactions and exposure times of the denitrifying bacteria within the system.

In a FWS system, nitrification occurs under aerobic conditions in the water column. The nitrates then diffuse into the (normally organic-rich) sediments and soil where anaerobic conditions necessary for denitrification exist. Nitrification and denitrification processes in SFS systems can occur, but the availability of oxygen for nitrification may be limiting.

In natural and constructed wetland systems, the principal phosphorus removal mechanisms are:

- Precipitation of phosphates under elevated pH conditions
- Sorption within the bottom soils
- Uptake by the macrophytic plants
- Fixation by algae and bacteria

Unlike denitrification for nitrogen removal, there is no one long-term sink for phosphorus removal or retention. This deficiency means that all the phosphorus removal mechanisms be considered in the system design process.

The soil and bottom sediments can be a significant phosphorus sink in mineral soils. In these areas, dissolved inorganic phosphorus is removed through adsorption by clays and precipitation as insoluble complexes of aluminum, calcium, and iron. The aluminum and iron reactions prefer acid to neutral soils; the calcium reactions require alkaline conditions. Plants may then take up phosphorus, especially that adsorbed by the soil.

To take advantage of mineral soils as a sink, the water must be moving in the soil column to afford reaction opportunities. The SFS wetlands reduce the influence of water–soil interchanges of phosphorus and force movement through the soil column; however, this approach limits the system's hydraulic capacity without a significant increase in wetland treatment area. If the area soils are mostly organic, special provisions would be required to seek deeper, more mineral soils if the soil is to be considered as a significant phosphorus removal mechanism.

Metals. The mechanisms for metal removal in wetland systems are chemical oxidation or reduction causing precipitation, sorption by plants or soil, and simple filtration or sedimentation. The presence of clay soils in a SFS enhances the opportunities of removal by adsorption. Filtration also will remove suspended metals in a SFS wetland system. In a FWS system, sedimentation will occur, and the development of neutral to alkaline conditions will favor precipitation of metals. In both systems, plant uptake may take place.

Flow

The ability to provide internal hydraulic controls is a significant difference in natural and constructed wetland treatment systems. Designed features for flexible water management would normally disturb plant communities in a natural system and would necessitate increased land area to achieve similar treatment goals. Design of the constructed wetland system can ensure specific hydrologic and hydraulics conditions and provide for targeted treatment of specific parameters.

Hydraulic Loading Rate. The hydraulic loading rate is closely tied to site-specific conditions, such as climatic conditions, soil characteristics (especially permeability), and selected plant species. Also, the BOD loading rate and evaporation rates inversely influence the hydraulic loading rate. As an order of magnitude guide, a hydraulic loading rate of 20,000 to 40,000 gpd/acre (2,140,000 to 4,280,000 m³/d/ha) may be used.

Detention Time. To achieve the desired treatment performance, the wastewater must be in the wetland system a sufficient time for the primary biological, and any secondary chemical and physical (sedimentation and sorption), actions to take place. Other factors, such as water depth and plant selection, also influence determination of detention times.

Biological activities are typically controlled in order to achieve treatment and avoid sluggish conditions contributing to creation of anaerobic conditions. Detention times on the order of 10 to 20 days should be expected, which is about half that of a facultative pond. The physical design, via internal flow depth and routing controls, can provide flexibility to satisfy various detention time requirements.

Water Depth. In a FWS treatment system, the selection of water depth is dependent on treatment goals and requires correlation with litter zone development, system detention time, types of vegetation, and pollutants requiring treatment. Water depth influences vegetation diversity. For example, deep water with high nutrient (especially nitrogen) levels is likely to lead to a monoculture or very low diversity.

Comparing vegetative groups, depths of 2.5 to 3 ft (0.75 to 1 m) may be used in a deep marsh community. A shallow marsh community will have water depths of less than 1.0 to 1.5 ft (0.3 to 0.5 m). Most emergent plant species, i.e., those affording the most diversity, perform best in depths less than 1.0 ft (0.3 m). On the whole, an average system water depth may range from 1.5 ft (0.5 m) to over 2 ft (0.6 m).

In a SFS wetland treatment system, the water flow is maintained below the ground surface, and the bed depth is controlled by the depth of penetration of the plant root systems. It is the root system that provides the oxygen for the SFS system.

Flow Controls. Influent flow distribution should be considered; its importance is inversely proportional to the degree of pretreatment. A point discharge of only screened wastewater will quickly establish odorous and objectionable sludge deposits in a FWS system and possibly blind the media in a SFS system. On the other hand, an influent for polishing after advanced treatment is more amenable to point influent discharge (97, 98, 101, 102, 107).

In constructed FWS wetlands, swales and/or perforated pipe may be used to disperse the influent. The inlet zone of a SFS wetland system should contain large gravel that will distribute but not “filter” the influent. Table 6.70 shows typical characteristics of SFS system media.

Internal flow controls are especially critical to successful operation and treatment performance of the wetland system. Weirs, gates, or other regulating devices are used to manage water depths, control velocities, divert flows during abnormal flow periods, or isolate cells for maintenance. For example, water levels must be fluctuated to maintain the selected vegetation to optimize the performance of individual cells.

When discharges from cells in parallel are planned, as in a SFS system, a common effluent collection must be included in the design. If the final discharge is unique, such as to a contiguous natural wetland, ef-

TABLE 6.70 Typical Media Characteristics for Constructed SFS Wetland Treatment Systems (103)

Media type	Maximum 10% grain size, mm	Porosity	Hydraulic conductivity k_s , ft ³ /ft ² d	K_{20}
Medium sand	1	0.42	1380	1.84
Coarse sand	2	0.39	1575	1.35
Gravelly sand	8	0.35	1640	0.86

fluent distribution from the final cell(s) should be considered, in which case, an alternate point or means of verifying permit compliance must be negotiated.

Often, a lake will be considered as a means to not only collect flows from the system but also to add polishing the treatment process and to improve the aesthetic value of the project. Implementation may be easy if a contiguous borrow area is needed during construction for dike and berm materials.

Vegetation

Wetland plants serve an important role by providing structure to support the algae and bacteria population that provide wastewater treatment capability and reliability in a community environment (97, 98, 100–102, 107).

Plant Selection. For wetland wastewater treatment systems, many plant species are available. The two principal groups are free-floating plants, such as hyacinths and duckweed, and rooted macrophytes, such as cattails and reeds. Operationally, the main difference between the two groups is that the floating plants require regular harvesting to maintain effectiveness. The macrophytes may be used to create shallow or deep marsh communities of bulrushes or cattails, shallow marshes of grass/herbaceous species, and forested (hardwood) swamps in a constructed wetland system.

Fundamental to the emergent plant selection process is the identification of plant species that can survive in design hydroperiods and that form dense stands or litter zones. Also, plants that provide high nutrient and mineral sorption capability often are desirable. Compatibility with hydrological design factors is required. Table 6.71 includes a variety of emergent plants that may be considered for use in cells of differing hydroperiods within a constructed wetland system.

Manipulation of operating water depths can create a limiting condition affecting plant diversity. With multiple cells and water level controls, the designer/operator may selectively influence the distribution of

TABLE 6.71 Candidate Emergent Plants For Constructed Wetland Treatment Systems

Common name	Scientific name	Common name	Scientific name
Arrowheads	<i>Sagittari</i> spp.	Pondweeds	<i>Potamogeton</i> spp.
Bladderworts	<i>Utricularia</i> spp.	Reed	<i>Phragmites australis</i>
Bulrushes	<i>Scirpus</i> spp.	Rush	<i>Juncus</i> spp.
Cattails	<i>Typha</i> spp.	Saw grass	<i>Cladium jamaicense</i>
Maidencane	<i>Panicum</i> spp.	Sedges	<i>Carex</i> spp.
Manna grass	<i>Glyceria</i> spp.	Spikerushes	<i>Eleocharis</i> spp.
Pickerelweed	<i>Pontederia cordata</i>	Waterweeds	<i>Elodea</i> spp.

plant species. Nevertheless, maintaining a relatively stable water level for extended periods can further limit diversity by preventing the germination of seeds from annuals and perennials. The hydroperiod, seed bank composition, and lesser factors determine the impacts on species composition.

The availability of nutrients affects the diversity and distribution of emergent plant species, the growth rate, and structural allocation of growth, for the competitive species. Moreover, nutrient uptake is an important part of the wetland's wastewater treatment performance, especially during the plant establishment period.

Cattails (principally) and bulrushes are among the most researched plant species for wetlands treatment systems. Both are capable of growing in shallow and relatively deep (for plants) waters, achieving similar heights, and forming very dense stands and well-developed litter zones. Furthermore, they provide the most common plant communities used in the deep marsh portion of a constructed wetland treatment system.

With shallower operating water levels, a grass/herbaceous marsh (also referred to as mixed or shallow marsh) community may be developed, providing increased surface areas for attached microorganisms. These communities have an increased plant species diversity, the capability to reduce and stabilize nutrient concentrations, and provide expanded potential for wildlife habitat. Some of mixed marsh plant species are adapted to continuous submergence. Since the shallower depths yield a reduced detention time on an area basis, the mixed marsh may be indirectly controlled by land availability.

Forested swamps have the greatest land requirements, since water depths are lowest and seasonal drawdowns are needed. A few tree species, such as cypress, gum, and tupelo, can survive prolonged periods of inundation. Most other wetland tree species require periodic, complete water drawdown to survive. Forested swamps contain trees that are capable of serving as relatively long-term sinks for nutrients and encourage more diverse wildlife.

Establishment. A period of time—the establishment period—is required to achieve full operability and treatment performance of a constructed wetland system. The establishment period may vary from 6 to 12 months. Plantings must be monitored and replaced as necessary during this time. In addition to plant loss due to natural conditions, animal foraging is a common problem during the establishment period. Often, the major problem is intrusion by undesirable plant species, which may be controlled by alternate means, including control of water levels. Flooding favors the wetland plant species and often is an especially effective control technique. Also, during this time, plant diversity will likely increase with the emergence of volunteer plant species.

Vegetation Management. The common wetland system categories, based on vegetation and in descending order of operating and maintenance costs, are floating plants (hyacinths), cattail or bulrush monocultures, and grass/herbaceous marshes. In floating plant systems, harvesting is a critical procedure for continued nutrient removal, as plant tissue is the primary nutrient sink. For emergent vegetation, needs for harvesting are not well supported.

LAND TREATMENT

Land treatment combines biological and physical–chemical treatment to provide a high degree of wastewater renovation; the system design is very site-specific (4, 65, 67). The treatment effluent may be discharged to surface waters, or it may percolate into the groundwater.

The three most common types of land treatment systems are slow rate, rapid infiltration, and overland flow. Even though each is most applicable to certain site characteristics, general site selection guidelines in Table 6.72 can be used. More site-specific requirements for the three land treatment systems are shown in Table 6.73. Once the candidate site(s) is selected, field testing can begin (see Table 6.74). Important considerations from soil investigations are presented in Tables 6.75 and 6.76.

TABLE 6.72 Land Treatment Site Selection Guidelines (65)

Characteristic	Process	Remarks
Soil permeability	Overland flow Rapid infiltration and slow rate	High-permeability soils are more suitable to other processes. Hydraulic loading rates increase with permeability.
Potential groundwater pollution	Rapid infiltration and slow rate	Affected by the (1) proximity of the site to a potential potable aquifer, (2) presence of an aquiclude, (3) direction of groundwater flow, and (4) degree of groundwater recovery by wells or underdrains.
Groundwater storage and recovery	Rapid infiltration	Capability for storing percolated water and recovery by wells or underdrains is based on aquifer depth, permeability, aquiclude continuity, effective treatment depth, and ability to contain the recharge mound within the defined area.
Existing land uses	All processes	Involves the occurrence and nature of conflicting land use.
Future land use	All processes	Future urban development may affect the ability to expand the system.
Size of site	All processes	If there are a number of small parcels, it is often difficult to purchase or lease the needed area.
Flooding hazard	All processes	May exclude or limit site use.
Slope	All processes	Steep grades may (1) increase capital expenditures for earthwork, and (2) increase the erosion hazard during wet weather.
	Rapid infiltration Overland flow	Steep grades often affect groundwater flow pattern. Steep grades reduce the travel time over the treatment area and treatment efficiency. Flat land requires extensive earthwork to create grades.
Water rights	All processes	May require disposal of renovated water in a particular watershed within a particular stretch of surface water.

TABLE 6.73 Site Characteristics for Land Treatment Systems (65)

	Slow rate	Rapid infiltration	Overland flow
Grade	Less than 20% on cultivated land; less than 40% on noncultivated land	Not critical; excessive grades require much earthwork	Finish slopes 2–8%*
Soil permeability	Moderately slow to moderately rapid	Rapid (sands, sandy loams)	Slow (clays, silts, and soils with impermeable barriers)
Depth to groundwater	0.6–1 m (minimum) [†]	1 m during flood cycle [‡] ; 1.5–3 m during drying cycle	Not critical [‡]
Climatic restrictions	Storage often needed for cold weather and during heavy precipitation	None (possibly modify operation in cold weather)	Storage usually needed for cold weather

*Steeper grades might be feasible at reduced hydraulic loadings.

[†]Underdrains can be used to maintain this level at sites with high groundwater table.

[‡]Impact on groundwater should be considered for more permeable soils.

Note: 1 m = 3.28 ft.

TABLE 6.74 Field Tests for Land Treatment Systems (65)

Properties	Processes		
	Slow rate	Rapid infiltration	Overland flow
Wastewater constituents	Nitrogen, phosphorus, SAR,* EC,* boron	BOD, SS, nitrogen, phosphorus	BOD, SS, nitrogen, phosphorus
Soil physical properties	Depth of profile Texture and structure	Depth of profile Texture and structure	Depth of profile Texture and structure
Soil hydraulic properties	Infiltration rate Subsurface permeability	Infiltration rate Subsurface permeability	Infiltration rate (optional) —
Soil chemical properties	pH, CEC, exchangeable cations (% of CEC), EC,* metals, [†] phosphorus adsorption (optional)	pH, CEC, phosphorus adsorption	pH, CEC, exchangeable cations (% of CEC)

*May be more significant for arid and semiarid areas.

[†]Background levels of metals such as cadmium, copper, or zinc in the soil should be determined if food-chain crops are planned.

Prior to use of a land application system, some wastewater pretreatment is needed; levels recommended by the EPA are presented in Table 6.77. Following these processes, the system should include storage and, in some cases, screening facilities preceding the distribution and application system. The components of storage design are the volumes required for water balance, operational and climatic factors, and emergency storage.

A summary comparison of land application design features is shown in Table 6.78. More particular discussions follow on each of the primary land treatment systems.

TABLE 6.75 Soil Characteristics for Land Treatment Systems (65)

Properties	Processes		
	Slow rate	Rapid infiltration	Overland flow
Wastewater constituents	Nitrogen, phosphorus, SAR,* EC,* boron	BOD, SS, nitrogen, phosphorus	BOD, SS, nitrogen, phosphorus
Soil physical properties	Depth of profile Texture and structure	Depth of profile Texture and structure	Depth of profile Texture and structure
Soil hydraulic properties	Infiltration rate Subsurface permeability	Infiltration rate Subsurface permeability	Infiltration rate (optional) —
Soil chemical properties	pH, CEC, exchangeable cations (% of CEC), EC,* metals, [†] phosphorus adsorption (optional)	pH, CEC, phosphorus adsorption	pH, CEC, exchangeable cations (% of CEC)

*May be more significant for arid and semiarid areas.

[†]Background levels of metals such as cadmium, copper, or zinc in the soil should be determined if food-chain crops are planned.

TABLE 6.76 Textural Properties of Mineral Soils (66, 67)

Soil class	Feeling and appearance	
	Dry soil	Moist soil
Sand	Loose, single grains, which feel gritty. Squeezed in the hand, the soil mass falls apart when the pressure is released.	Squeezed in the hand, it forms a cast that crumbles when touched. Does not form a ribbon between thumb and forefinger.
Sandy loam	Aggregates easily crushed; very faint velvety feeling initially, but with continued rubbing the gritty feeling of sand soon dominates.	Forms a cast that bears careful handling without breaking. Does not form a ribbon between thumb and forefinger.
Loam	Aggregates are crushed under moderate pressure. Clods can be quite firm. When pulverized, loam has velvety feel that becomes gritty with continued rubbing. Casts bear careful handling.	Cast can be handled quite freely without breaking. Very slight tendency to ribbon between thumb and forefinger. Rubbed surface is rough.
Silt loam	Aggregates are firm, but may be crushed under moderate pressure. Clods are firm to hard. Smooth, flourlike feel dominates when soil is pulverized.	Cast can be freely handled without breaking. Slight tendency to ribbon between thumb and forefinger. Rubbed surface has a broken or rippled appearance.
Clay loam	Very firm aggregates and hard clods that strongly resist crushing by hand. When pulverized, the soil takes on a somewhat gritty feeling due to the harshness of the very small aggregates that persist.	Cast can bear much handling without breaking. Pinched between the thumb and forefinger, it forms a ribbon whose surface tends to feel slightly gritty when dampened and rubbed. Soil is plastic, sticky, and puddles easily.
Clay	Aggregates are hard. Clods are extremely hard and strongly resist crushing by hand. When pulverized, it has a gritlike texture due to the harshness of numerous very small aggregates that persist.	Cast can bear considerable handling without breaking. Forms a flexible ribbon between thumb and forefinger and retains its plasticity when elongated. Rubbed surface has a very smooth, satin feeling. Sticky when wet and easily puddled.

TABLE 6.77 Levels of Preapplication Treatment (65)

Type of land treatment	Situation	Recommended preapplication treatment
Slow rate	Isolated location; restricted public access; crops not for human consumption.	Primary.
	Controlled agricultural irrigation; crops not to be eaten raw by humans.	Biological (ponds or in-plant processes) with control of fecal coliforms to <1000 MPN/100 mL.
	Public access areas such as parks and golf courses.	Biological (ponds or in-plant processes) with disinfection to log mean fecal coliforms of ≤ 200 MPN/100 mL.
Rapid infiltration	Isolated location; restricted public access. Urban location; controlled public access.	Primary. Biological (ponds or in-plant processes).
Overland flow	Isolated site; no public access. Urban location; no public access.	Screening or comminution. Screening or comminution with aeration to control odors during storage or application.

TABLE 6.78 Typical Design Data for Land Treatment Processes (65)

Feature	Slow rate	Rapid infiltration	Overland flow
Application techniques	Sprinkler or surface ^a	Usually surface	Sprinkler or surface
Annual loading rate, m	0.5–6	6–125	3–20
Field area required, ha ^b	23–280	3–23	6.5–44
Typical weekly loading rate, cm	1.3–10	10–240	6–40 ^c
Minimum preapplication treatment in the U.S.	Primary sedimentation ^d	Primary sedimentation ^d	Grit removal and comminution ^d
Disposition of applied wastewater	Evapotranspiration and percolation	Mainly percolation	Surface runoff and evapotranspiration with some percolation
Vegetation	Required	Optional	Required

^aIncludes ridge-and-furrow and border strip.

^bField area in hectares not including buffer area, roads, or ditches for 3785 m³/day (1 mgd) flow.

^cRange includes raw wastewater to secondary effluent, higher rates for higher level of preapplication treatment.

^dWith restricted public access; crops not for direct human consumption.

^eWith restricted public access.

Note: m = 3.28 ft; ha = 2.47 acres; cm = 0.39 in.

Slow-Rate Systems

Slow-rate systems are the most common form of land treatment. They are able to provide the highest degree of treatment and generally cause few adverse environmental impacts. A schematic of a slow-rate system appears in Figure 6.69.

Water applied to a slow-rate/irrigation site is either transmitted through the soil to the groundwater or is returned to the atmosphere through evapotranspiration; the system is designed to prevent surface runoff. Underdrains may be required under some geologic and hydrologic conditions.

The upper layer of soil provides most of the pollutant removal. Soil bacteria break down applied organic materials, and nitrification and denitrification take place. The soil matrix removes solids and pathogenic organisms. Metals are removed to a high degree, primarily through absorption onto clay materials. Plants utilize nitrogen and phosphorus as nutrients during growth.

Conventional preliminary and biological treatment processes are usually provided ahead of slow-rate systems. Grit and coarse solids (such as rags) will damage downstream equipment and clog distribution devices. High levels of organic materials may reduce the permeability of the upper layers of soil.

Dissolved salts may be a serious problem for slow-rate systems. For example, high concentrations of sodium applied to soils containing clay may cause the soil structure to disperse, and the soil may then become less permeable. The sodium adsorption ratio (SAR) is defined as follows:

$$\text{SAR} = \frac{\text{Na}}{[(\text{Ca} + \text{Mg})/2]^{0.5}}$$

where all concentration values are on a milliequivalents per liter basis. Values of SAR above 9 may decrease the permeability of fine-textured soils.

Nitrogen removal often controls the design of slow-rate systems, because of a nitrate-nitrogen limit of 10 mg/L in groundwater. This limit is to prevent methemoglobinemia, a disease that occurs in infants who have ingested too much nitrate. Nitrogen should be applied to the soil in the form of organic nitrogen or ammonia, since nitrate ions are highly mobile and are readily leached through the soil matrix.

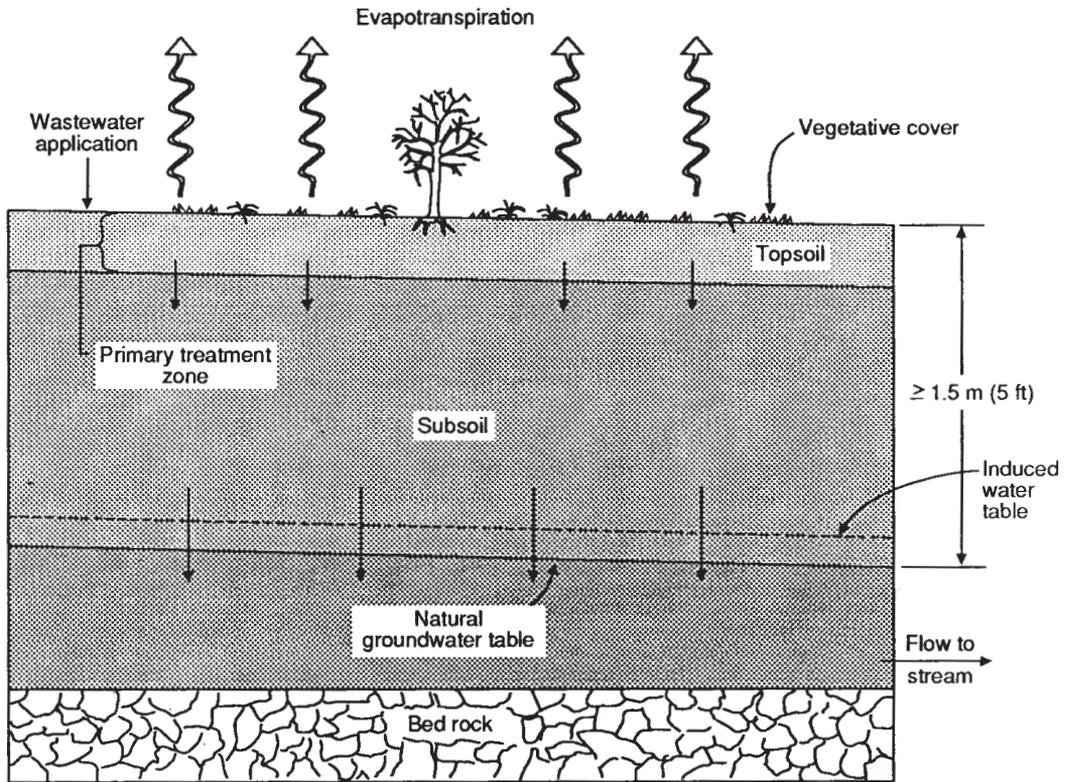


FIGURE 6.69 Slow-rate process schematic.

The rate of nitrogen uptake by crops varies throughout the life cycle of the plant and is crop-specific. The amount of phosphorus in applied wastewaters is usually in excess of plant requirements, but most soils have the ability to store significant quantities of it. Table 6.79 provides information on nitrogen, phosphorus, and potassium uptake rates for selected crops.

The greatest nutrient removals can generally be achieved with perennial crops, which are cut frequently at early stages of growth. Annual crops have lower potential for harvesting nutrients since they grow only part of the available growing season.

Forests can also remove significant amounts of nutrients from wastewater. Nutrient uptake depends upon the species and stand density, age, length of season, and temperature. Some of the nutrients that are taken up are returned to the forest floor as leaf fall and dead wood. Estimates of the annual nitrogen storage for a number of forest ecosystems, including understory vegetation, during the period of active tree growth are shown in Table 6.80. The actual amount of nitrogen that can be removed with a forest crop depends upon the type of harvesting technique used. The amount of phosphorus uptake by trees is usually quite small. However, the relatively low pH of forest soils favors the retention of phosphorus.

Another significant factor in slow-rate system design is the hydraulic loading rate, which must be determined from a water balance so that wastewater runoff from an application site is minimized. The water balance equation is

$$P + W = E + S$$

TABLE 6.79 Nutrient Uptake Rates for Selected Crops (65)

	Nitrogen	Phosphorus	Potassium
Forage crops			
Alfalfa*	225–540	22–35	175–225
Bromegrass	130–225	40–55	245
Coastal bermudagrass	400–675	35–45	225
Kentucky bluegrass	200–270	45	200
Quackgrass	235–280	30–45	275
Reed canarygrass	335–450	40–45	315
Ryegrass	200–280	60–85	270–325
Sweet clover*	175	20	100
Tall fescue	150–325	30	300
Orchardgrass	250–350	20–50	225–315
Field crops			
Barley	125	15	20
Corn	175–200	20–30	110
Cotton	75–110	15	40
Grain sorghum	135	15	70
Potatoes	230	20	245–325
Soybeans*	250	10–20	30–55
Wheat	160	15	20–45

*Legumes will also take nitrogen from the atmosphere.

Note: Units in kilograms per hectare per year which equals pounds per acre per year.

TABLE 6.80 Estimated Net Nitrogen Uptake by Forests* (65, 68)

	Tree age, yr	Average annual nitrogen uptake kg/ha·yr (lb/acre·yr)
Eastern forests		
Mixed hardwoods	40–60	220
Red pine	25	110
Old field with white spruce plantation	15	280
Pioneer succession	5–15	280
Southern forests		
Mixed hardwoods	40–60	340
Southern pine with no understory	20	220t
Southern pine with understory	20	320
Lake states forests		
Mixed hardwoods	50	110
Hybrid poplar [†]	5	155
Western forests		
Hybrid poplar [‡]	4–5	300–400
Douglas fir plantation	15–25	150–250

*Regions of the United States.

[†]Principal southern pine included in these estimates is loblolly pine.

[‡]Short-term rotation with harvesting at 4 to 5 years; represents first-growth cycle from planted seedlings.

where the precipitation P and applied wastewater W equal the evapotranspiration E and percolation S . Data on precipitation and evapotranspiration may be obtained from the Soil Conservation Service. Percolation rates are very site-specific and should be verified in the field. The actual percolation rate used in the water balance should not exceed 4 to 10% of the permeability of the most restrictive soil layer.

The average nitrogen content of the percolate can be estimated by using the water balance along with a nitrogen mass balance. The system design will usually be controlled either by groundwater nitrate levels or by the hydraulic application rate.

Sprinkler systems are used for wastewater application and offer the advantage of uniform distribution, variable application rates, and topographic flexibility. Characteristics of sprinkler systems are shown in Table 6.81. Typically, rest periods of at least three days are needed to reestablish a zone of aeration in the upper soil layer; thus, separate sprinkler systems are established to permit rotation of application sites.

Rapid Infiltration

Rapid infiltration is becoming more widespread as a method of wastewater treatment. The systems are designed for high hydraulic loadings and may be extremely attractive where suitable geologic formations of rapid permeability occur. Almost all of the applied wastewater percolates through the soil (see Figure 6.70). There is little evapotranspiration, since vegetation is not generally used. The percolate may be removed by wells or underdrains, or it may recharge the local groundwater system. Wastewater renovation is due to physical and chemical interactions between the soil and wastewater constituents and the action of soil microorganisms.

Preapplication treatment is primarily directed toward minimizing soil clogging and avoiding the creation of odors. Biological secondary treatment is usually provided, but there are cases where only primary or no treatment have been used.

The primary method of nitrogen removal is denitrification, since no plant uptake occurs. Some control over denitrification can be exercised by manipulating the loading and resting periods. Sufficient carbon must be provided for denitrification to occur. Approximately 2 mg/L of carbon must be provided for each 1 mg/L of nitrogen that is converted to nitrogen gas.

The hydraulic loading rate must be based on the permeability of the most restrictive layer in the soil profile. As with slow-rate systems, the actual design loading should be from 4 to 10% of the clean water rate, as shown in Table 6.82. System operation must be intermittent to allow rest periods for the soil to dry out to increase infiltration potential and to permit aerobic bacteria to break down wastewater constituents. Typical loading cycles are presented in Table 6.83.

The method of wastewater application to rapid-infiltration basins is usually by flooding, although sprinkling may be used. Where flooding is practiced, either a bare or a vegetated surface can be provided. Since rapid infiltration involves discharging large quantities of water onto a limited area, groundwater mounding may be a problem. This can be overcome by the installation of recovery wells or an underdrain system.

Overland Flow

Overland flow is a treatment process wherein wastewater is applied to the upper portions of sloped, vegetated surfaces and flows down the surface to collection ditches (see Figure 6.71). Pollutants are removed by biological, chemical, and physical methods as the wastewater flows through the application site. Since the wastewater flows downslope, relatively impermeable soils are used. Soil slopes of from 2 to 8% are satisfactory, with no differences in quality of treatment with slopes in the range.

Nitrogen removal by overland flow systems is not well understood. Major mechanisms for nitrogen removal are crop uptake and biological nitrification and denitrification, which appears to be the most impor-

TABLE 6.81 Sprinkler System Characteristics (65)

	Typical application rate, cm/h	Labor required per application, h/ha	Nozzle pressure range, N/cm ²	Size of single system, ha	Shape of field	Maximum grade, %	Maximum crop height, m
Solid set							
Permanent	0.13–5.08	0.02–0.04	21–69	Unlimited	Any shape	—	—
Portable	0.13–5.08	0.08–0.10	21–41	Unlimited	Any shape	—	—
Move-stop							
Hand move	0.03–5.08	0.2–0.6	21–41	<1–16	Any shape	20	—
End tow	0.03–5.08	0.08–0.16	21–41	8–16	Rectangular	5–10	—
Side-wheel roll	0.25–5.08	0.04–0.12	21–41	8–32	Rectangular	5–10	1–1.2
Stationary gun	0.64–5.08	0.08–0.16	35–69	8–16	Any shape	20	—
Continuous move							
Traveling gun	0.64–2.54	0.04–0.12	35–69	16–41	Any shape	—	—
Center pivot	0.51–2.54	0.02–0.06	10–41	16–65	Circular*	5–15	2.4–3
Linear move	0.51–2.54	0.02–0.06	10–41	16–130	Rectangular	5–15	2.4–3

*Travelers are available to allow irrigation of any shape field.

Note: 1 cm/h 0.39 in/h; 1 h/ha = 0.4 h/acre; 1 N/cm² 1.45 lb/in² 1 ha = 2.47 acres; 1 m 3.28 ft.

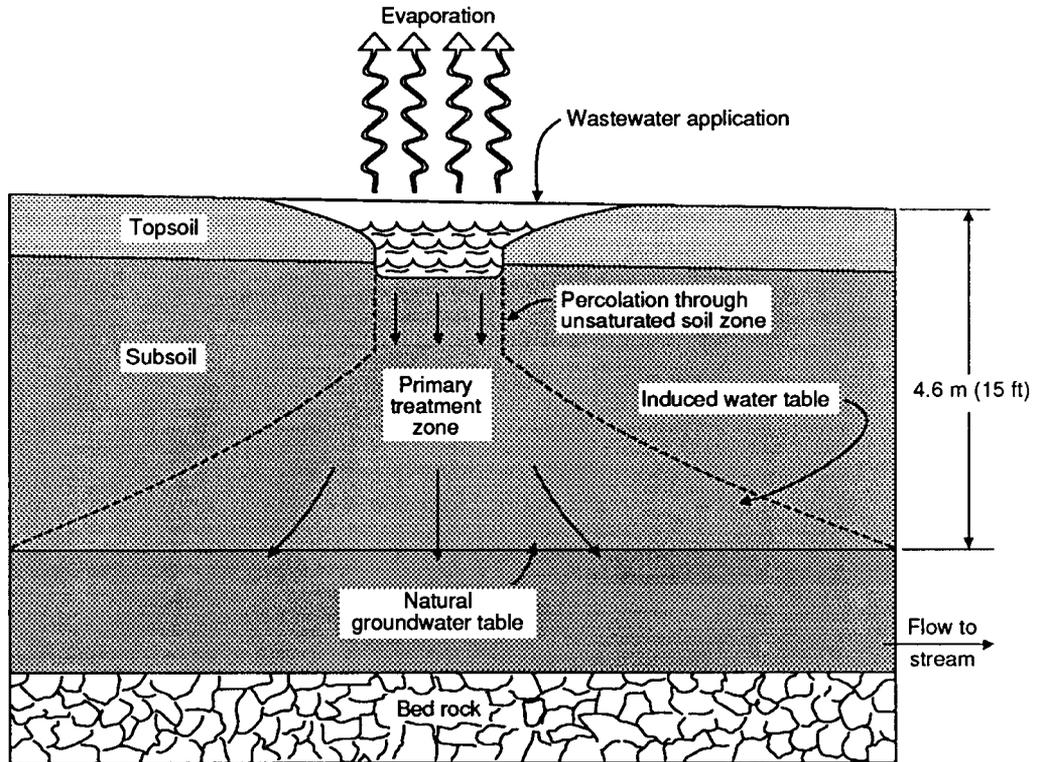


FIGURE 6.70 Rapid-infiltration process schematic.

tant. Generally accepted relationships are that the total nitrogen removal is inversely related to the application rate and length of slope, that continuous wastewater application reduces nitrification rates, and that both nitrogen removal and nitrification are temperature-sensitive.

Overland flow systems are not as effective in removing phosphorus as slow-rate or rapid-infiltration systems. This is because the wastewater does not have as much opportunity to contact the soil components that fix phosphorus.

Wastewater distribution may be by surface application, low-pressure sprays, or high-pressure impact sprinklers. Surface methods include gated, slotted, or perforated pipe, and bubbling orifices. These methods

TABLE 6.82 Adjustment Factors for Rapid Infiltration Hydraulic Loading Rates (66)

Test procedure	Adjustment factor for annual loading rate
Basin flooding test	10–15% of “effective” rate observed
Air entry permeameter and cylinder infiltrometers	2–4% of “effective” rate observed
Laboratory permeability measurements	4–10% of “effective” rate observed, or of restricting soil layer

TABLE 6.83 Typical Rapid Infiltration Loading Cycles (65)

Loading cycle objective	Applied wastewater	Season	Application period, days*	Drying period, days
Maximize infiltration rates	Primary	Summer	1-2	5-7
		Winter	1-2	7-12
	Secondary	Summer	1-3	4-5
		Winter	1-3	5-10
Maximize nitrogen removal	Primary	Summer	1-2	10-14
		Winter	1-2	12-16
	Secondary	Summer	7-9	10-15
		Winter	9-12	12-16
Maximize nitrification	Primary	Summer	1-2	5-7
		Winter	1-2	7-12
	Secondary	Summer	1-3	4-5
		Winter	1-3	5-10

*Regardless of season or cycle objective, application periods for primary effluent should be limited to 1 to 2 days to prevent excessive soil clogging.

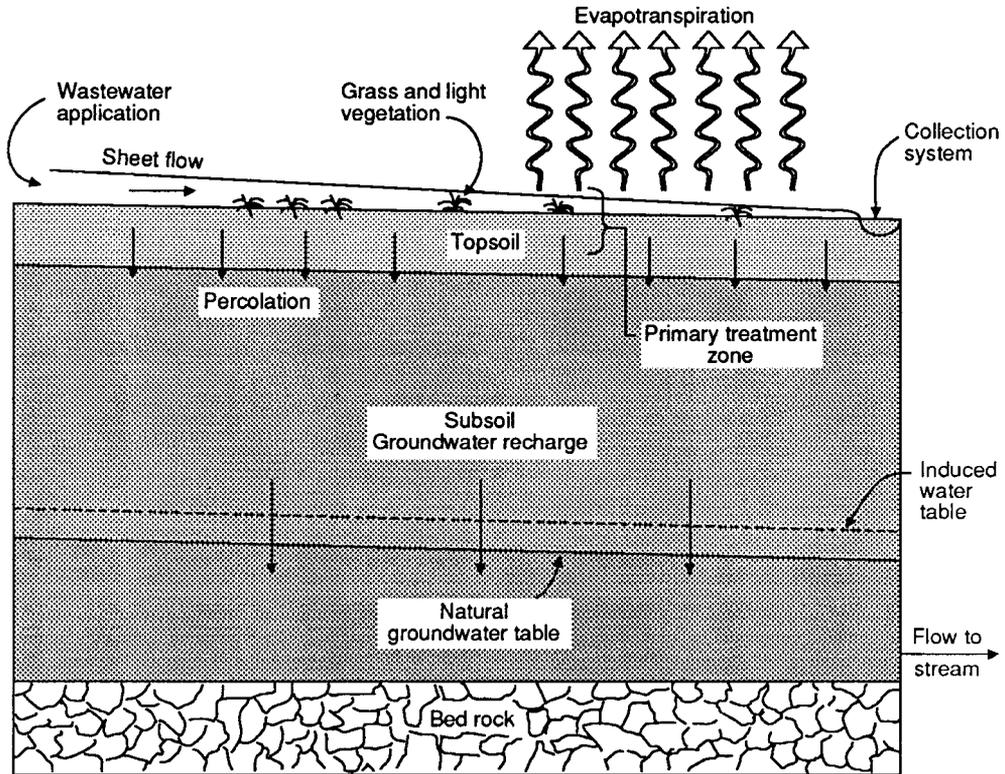


FIGURE 6.71 Overland flow process schematic.

TABLE 6.84 Suitable Overland Flow Grasses (66)

Common name	Perennial or annual	Rooting characteristics	Method of establishment*	Growing height, cm
Cool-season grasses				
Reed canary	Perennial	Sod	Seed	120–210
Tall fescue	Perennial	Bunch	Seed	90–120
Rye grass	Annual [†]	Sod	Seed	60–90
Redtop	Perennial	Sod	Seed	60–90
Kentucky bluegrass	Perennial	Sod	Seed	30–75
Orchard grass	Perennial	Bunch	Seed	15–60
Warm-season Grasses				
Common bermuda	Perennial	Sod	Seed	30–45
Coastal bermuda [‡]	Perennial	Sod	Sprig	30–60
Dallis grass	Perennial	Bunch	Seed	60–120
Bahia	Perennial	Sod	Seed	60–120

*Optimum planting period for a specific grass varies according to geographical location.

[†]Used as a nurse crop.

[‡]Includes other improved Bermuda grass varieties.

Note: 1 cm = 0.39 in.

require less energy to operate but may require more operational attention. Discharge orifices may become plugged with debris if adequate pretreatment is not provided.

Since relatively impermeable soils are used with overland flow, groundwater contamination should not be a problem. However, special attention must be given to collection of both treated runoff and storm runoff. Use of the 25-year return frequency for storm runoff channels has been suggested by the EPA; local requirements may be more stringent.

A primary operational consideration with overland flow systems is cover crop management; suitable grasses are listed in Table 6.84. Grass is usually cut two or three times a year. This removes nutrients from the site and allows channeling problems to be more readily observed. Some financial return may also be realized by the sale of hay. The site must be allowed to dry before harvesting so that agricultural equipment will not leave ruts or tracks that can become undesirable flow channels.

PHYSICAL AND CHEMICAL TREATMENT

The singular and/or combined results of physical and chemical treatment are important aspects of wastewater treatment and control. Equalization and spill control are needed in some projects to protect the treatment works from shock pollutants and hydraulic loadings. Neutralization and temperature control are processes that prepare the wastewater for effective use of downstream treatment processes. Oil and grease and heavy metal removal are processes for control of specific pollutants and may be used for pretreatment before discharge to a treatment plant or for independent treatment prior to discharge to a stream. Finally, post-aeration and disinfection processes are used to prepare a treated effluent for final discharge to the receiving stream.

Equalization

Equalization is used to minimize the variability of wastewater flow rates and composition (16, 69, 70, 71). These variations occur because of diurnal flows from domestic sources, cyclic industrial operations, and

rainfall (infiltration/inflow) events. Treatment facilities can be made to operate at maximum efficiency when the flow and strength of wastewater are relatively constant. In addition, certain parts of the facilities can be made smaller, such as biological reactors, oxygen transfer systems, and clarifiers, thereby minimizing capital expenditures.

Equalization is more routinely employed in industry than at municipal facilities, because many industries use batch production processes. However, there are now also a large number of municipal equalization installations. Equalization is particularly attractive at municipal plants that have high peak to average flow ratios. Design alternatives for equalization systems are shown in Table 6.85.

Wastes containing potential toxic or inhibitory compounds clearly need to be equalized prior to biological treatment to a greater degree than those with slightly fluctuating organic or hydraulic loads. If practical, equalization systems should be evaluated through pilot testing or evaluation of full-scale data to determine the tolerance to variability of the particular process under consideration.

Equalization facilities are either of the in-line or off-line type (see Figures 6.72 and 6.73). All flows go through in-line basins, which discharge at a constant rate. Off-line basins only receive flow when the flow rates or concentrations are greater than average and only discharge when plant influent flows or concentrations are less than average.

Numerous methods are available to evaluate equalization of varying wastewater flows and pollutant concentrations and to compute in-line or off-line equalization/storage volume. Methods include the simple flow balance, simple composition balance, combined flow and composition balance, sine wave method, rectangular wave method, and others (110–114).

Flow Balance. The most common technique for the design of flow equalization is the mass diagram approach. This approach develops a plot of cumulative flow versus time. Such a graph is called a Rippl diagram and is commonly used in the design of water storage reservoirs. An example of a Rippl diagram for equalization of flow is shown in Figure 6.74. Determination of the equalization volume required from the Rippl diagram is first made by constructing lines parallel to the treatment flow rate and tangent to the wastewater flow graph. The difference between the two parallel lines at the point tangent to the flow curves is the equalization volume required.

TABLE 6.85 Equalization System Design Alternatives

Condition	Alternative
Possible Locations	Off-line in collection system After headworks After primary clarification Before advanced treatment
Flow regime	In-line Off-line
Sizing techniques	
Hydraulic	Mass diagram
Hydraulic and organic	Statistical techniques Interactive procedures
Mixing methods*	Surface aeration Diffused air aeration Turbine mixing Inlet flow distribution and baffling

*Mixing not always provided.

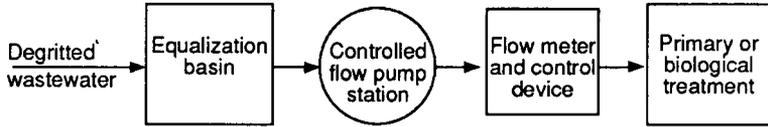


FIGURE 6.72 In-line equalization basin flow schematic.

The simple flow balance approach can also be calculated using the formula:

$$Q_i \Delta t = \Delta V + Q_o \Delta t$$

- where Q_i = average flow into the basin over Δt
- Δt = time interval
- V = change in stored volume during Δt
- Q_o = average flow out of the basin over Δt

An example tabular solutions for the 24-hour hydrograph shown in Figure 6.75 is presented in Table 6.86. The selected time interval is 2 hours. In this case, the in and out volume (V_i and V_o) were calculated for the in and out flow rate (Q_i and Q_o) for the time interval (Δt):

$$Q_i \Delta t = V_i$$

$$Q_o \Delta t = V_o$$

$$\Delta V = V_i - V_o$$

The working volume of equalization required is the difference in the maximum and minimum sums of the calculated volume changes. For the example in Table 6.86, the

$$\text{Working Equalization Volume Required} = (0.264 \cdot 10^6 \text{ gal}) - (-2.347 \cdot 10^6 \text{ gal}) = 2.611 \cdot 10^6 \text{ gal}$$

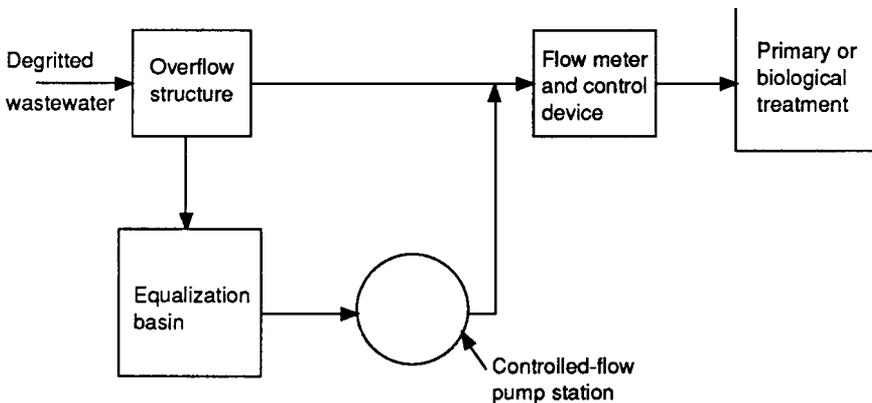


FIGURE 6.73 Off-line equalization basin flow schematic.

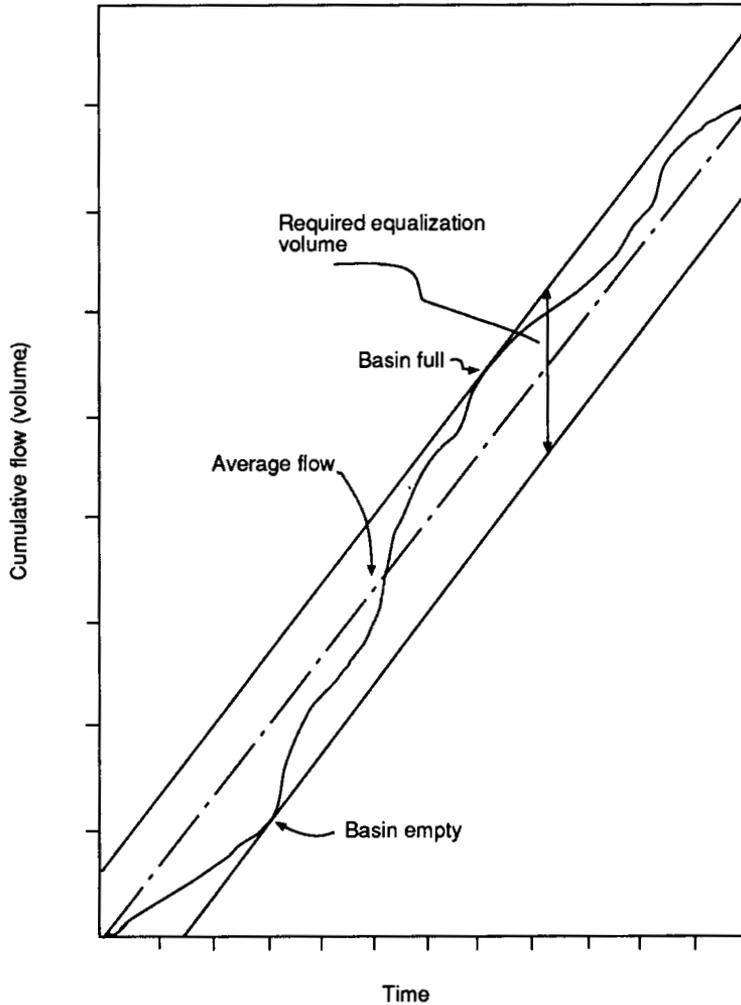


FIGURE 6.74 Rippl diagram for flow equalization.

Composition Balance. On the other hand, if the flow rate is constant and a constant-volume, in-line equalization basin is completely mixed, a simple composition or concentration balance may be calculated using the formula:

$$\Delta c = \frac{Q}{V} \Delta t (c_i - c)$$

where Δc = change in concentration during Δt

Q = constant flow rate

V = constant basin volume

WASTEWATER DISPOSAL

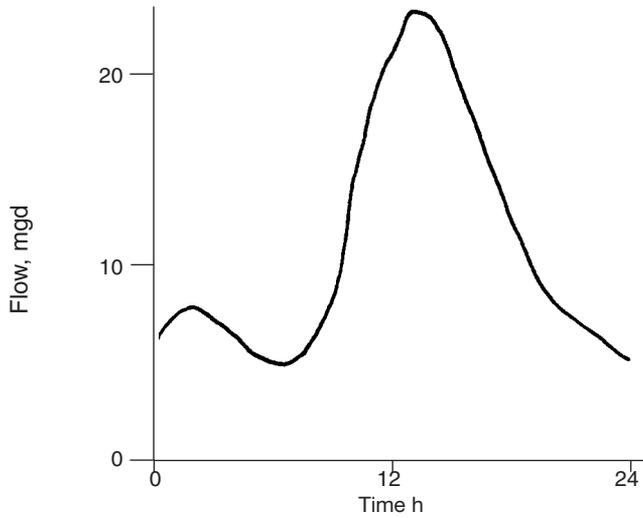


FIGURE 6.75 24-Hour hydrograph.

TABLE 6.86 Example Tabular Equalization Solution

Time, hr	Q_i , mgd	$V_i(Q_i \Delta t)$, 10^6 gal	Total V , 10^6 gal	$V_o(Q_o \Delta t)$, 10^6 gal	ΔV , 10^6 gal	$\Sigma \Delta V$, 10^6 gal
0	5	0.000				
		0.292	0.292	0.944	-0.653	-0.653
2	7	0.583				
		0.542	0.833	0.944	-0.403	-1.056
4	6	0.500				
		0.458	1.292	0.944	-0.486	-1.542
6	5	0.417				
		0.458	1.750	0.944	-0.486	-2.028
8	6	0.500				
		0.625	2.375	0.944	-0.319	-2.347
10	9	0.750				
		1.208	3.583	0.944	0.264	-2.083
12	20	1.667				
		1.833	5.417	0.944	0.889	-1.194
14	24	2.000				
		1.792	7.208	0.944	0.847	-0.347
16	19	1.583				
		1.417	8.625	0.944	0.472	0.125
18	15	1.250				
		1.083	9.708	0.944	0.139	0.264
20	11	0.917				
		0.792	10.500	0.944	-0.153	0.111
22	8	0.667				
		0.583	11.083	0.944	-0.361	-0.250
24	6	0.500				
		0.250	11.333			
Totals	141	11.333		11.333	-0.250	
Average	11.333					

Δt = time interval
 c_i = average influent concentration over Δt
 c = initial concentration in basin

The equalization is modified if decomposition occurs in the basin. For example, if first-order decay is expected, the formula becomes

$$\Delta c = \frac{Q}{V} \Delta t (c_i - c) - Kc \Delta t$$

where K = decay coefficient

If the basin is not completely mixed, the simple concentration balance equation is (69)

$$\Delta c = \frac{(Q/V) \Delta t (c_i - c) - Kc \Delta t}{1 + (\Delta t/2) (Q/V + K)}$$

Flow and Composition Balance. The balance equations become more complex as more realistic changes in flow and composition are considered. For example, in a completely mixed in-line basin, the basic relationships for flow and concentration changes, respectively, are

$$V = Q_i \Delta t - Q_o \Delta t$$

and

$$\Delta c = \frac{Q_i c_i \Delta t - c(Q_o \Delta t + \Delta V)}{V + \Delta V}$$

If Δc is much smaller than c and a large Δt is used, suggesting that $c + \Delta c/2$ is the average concentration during Δt , the iteration equation becomes

$$\Delta c = \frac{Q_i c_i \Delta t - c(Q_o \Delta t + \Delta V)}{V - \Delta V + (Q_o \Delta t/2)}$$

The calculations will be affected by the initial conditions of c and V . If other system variables are periodic, c and V will eventually become periodic also, and the initially assumed conditions will become unimportant. Several cycles may be required for this to occur.

More complete load and concentration dampening can be provided by increasing the basin dead volume. The calculations discussed above can be repeated using various values of equalization volume. Once this mathematical evaluation is complete, the design engineer will have available data indicating the effluent dampening effect (i.e., reduction in organic concentration variability) of various size equalization basins based on the raw wastewater variability entering the basin. The critical decision to be made is how large a basin to select. This depends upon how much variability is cost effective for the downstream process, which is a function of the type of process and the treatability of the wastewater.

Sine Wave Method. The sine wave method approximates equalization conditions by assuming that both flow and concentration are represented by sine waves in phase with each other and with a period of one day.

As such, this method is limited for typical flow variations in municipal wastewater systems. However, since the method is not iterative, it is useful for making first estimates for diurnal flow variations. The flow equation takes the form of:

$$Q_i(t)Q_o(t) = Q_A - (Q_P - Q_A) \sin 2\pi t$$

where $Q_i(t)$ = influent flow as a function of time

Q_A = average flow

Q_P = peak influent flow

t = time in days

If a constant outflow rate is desired, the formula can be integrated over a time interval, e.g., $t = 0.5$ to $t = 1.0$ day, yielding a working volume of equalization storage:

$$V = \frac{Q_P - Q_A}{\pi}$$

Rectangular Wave Method. The rectangular wave method assumes that flows are approximated by rectangular rather than sine waves and the flow ratio of peak to average equals the average to minimum. The approach also may be used for concentration and mass loading conditions. For a constant outflow, the required storage volume is:

$$V = \frac{Q_A(x - 1)^2}{(x^2 - 1)}$$

where x = peak-to-average flow ratio, equals average-to-minimum flow ratio

Mixing. Floating mechanical aerators are the most common approach to providing mixing and oxygen dispersion. Aeration or mixing requirements for medium-strength municipal wastewaters range from 15 to 40 hp/10⁶ gal (3 to 8 kW/10³ m³). Higher power levels can be required when solids with a specific gravity significantly greater than 1.0 must be kept in suspension.

A common mistake in equalization basin design is to neglect minimum required depths for aeration devices. Storage below this depth becomes dead volume and cannot be used for damping flow variations unless mixing is discontinued.

Many unmixed basins that will retain wastewater for no more than one day have been operated satisfactorily. These should be lined and washdown water should be provided.

Spill Control

Provision for handling spills of toxic materials or concentrated waste streams is an important part of pre-treatment design. The EPA as well as many state and local governments require certain industries to maintain spill control plans. Many techniques have evolved over time, such as dikes around storage vessels and locks on critical valves to control tank dumps, washouts, etc.

Almost all manufacturing processes suffer product losses or production problems at times. Usually, these waste products are discharged to a process sewer. Since the strength or toxicity of these materials may be many times greater than what normally enters the treatment system, an emergency storage or off-line storage basin is required.

No precise mathematical approach is available for designing these types of basins, since the design depends upon the particular pollutant of concern. Typical sensing parameters for diversion are pH, TOC, flow, or conductivity. Operation may be automatic (see Figure 6.76) or manual. The contents of the basin are returned to the treatment works to provide an influent concentration of the particular pollutant below objectionable levels.

Off-line storage basins should be provided at industries that utilize significant quantities of toxic pollutants. This is the case whether the industry has its own treatment plant or discharges to a municipal treatment facility.

The primary regulation for oil and fuel spill control is 40 CFR 112, which includes federal requirements for spill prevention control and countermeasure (SPCC) plans. This EPA regulation also identifies requirements and alternatives for containment. For example, above-ground storage installations should provide containment for the largest single tank plus sufficient freeboard to allow for precipitation. Similarly, fuel transfer areas should have containment for at least the maximum capacity of any single tank car or tank truck compartment expected to operate at the site. The table of contents for a typical SPCC plan is presented in Table 6.87.

Neutralization

One of the most common requirements for chemical treatment is the neutralization of excess acidity or alkalinity in wastewater. The indicator used to measure acidity or alkalinity is pH. The acid range is below pH 7, and the alkaline range is above pH 7. The types of acidity and alkalinity are illustrated in Figure 6.77. Wastewaters with pH values outside the range of 6 to 10 may be corrosive to equipment and may cause other pH-related problems in a treatment system. Discharges to the environment usually must have a pH of between 6 and 9.

One of the critical items in neutralizing a wastewater is to determine the nature of the ions causing the acidity or alkalinity. This can be done by preparing titration curves in the laboratory showing the quantity of alkaline material necessary to adjust the pH of an acid wastewater, Figure 6.78, to a selected pH and vice-versa for an alkaline wastewater, Figure 6.79. Both the quantity of neutralizing chemical required and the shape of the titration curve are very important. Some wastewaters show very little change in pH with the addition of a neutralizing chemical (highly buffered), and others show a very dramatic change in pH with little chemical addition. This is important in the design of a system if very close control of pH is required, e.g., \pm

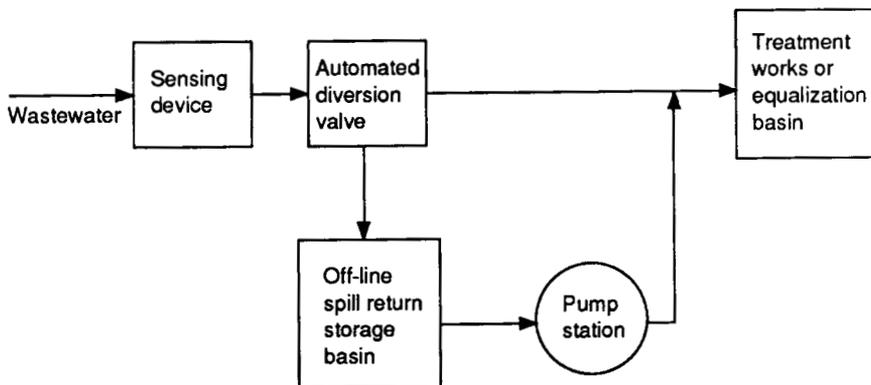
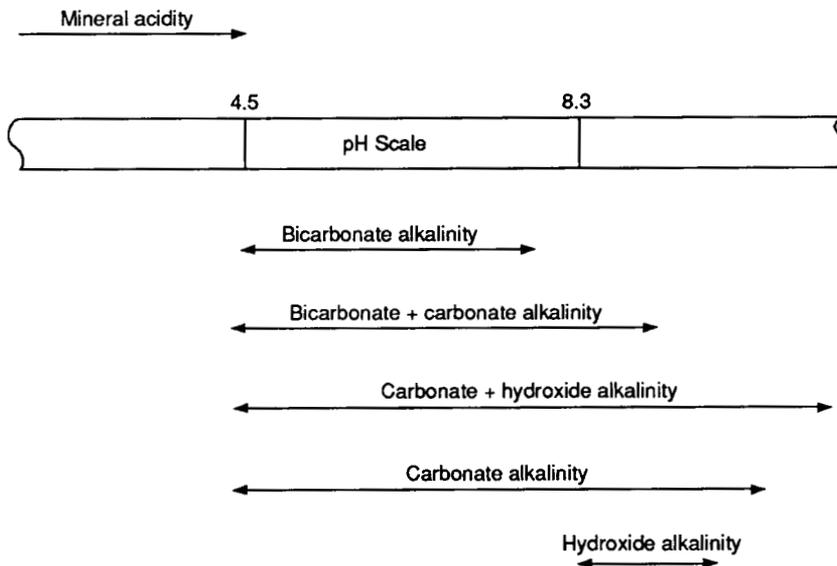


FIGURE 6.76 Off-line spill storage basin flow schematic.

TABLE 6.87 Typical SPCC Plan: Table of Contents Certification

Statements
Section 1— Introduction
Section 2— Facility Description
Receiving Area
Above-Ground Storage
Below-Ground Storage
Section 3— Spill Prevention
Above-Ground Operations
Below-Ground Operations
Section 4— Spill Control and Countermeasures
Spill Response
Spill Containment and Cleanup
Postspill Report
Section 5— Plan Implementation
Facility Improvements
Inspections and Records
Personnel Training
Security
Appendix A— 40 CFR 112 Pollution Prevention Regulations
Appendix B— Documentation of Previous Spills
Appendix C— State and Federal Legal Actions
Appendix D— Inspection Report Forms
Appendix E— Personnel Training Information



Note: Methyl orange end point at pH of 4.5
 Phenolphthalein end point at pH of 8.3

FIGURE 6.77 Types of acidity and alkalinity

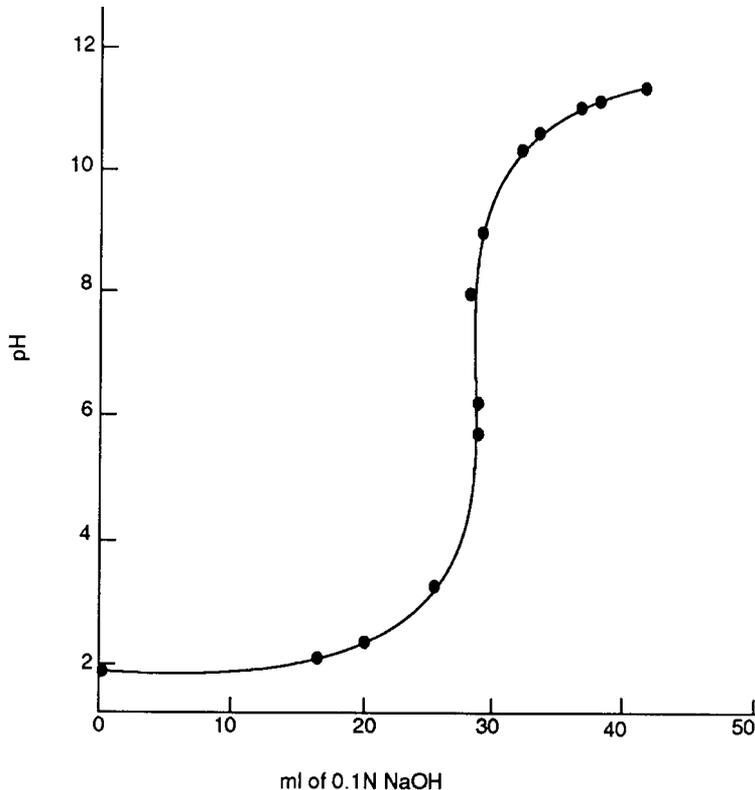


FIGURE 6.78 Titration curve for acidic wastewater.

1 pH unit, and minor quantities of acid or alkaline reagents change the pH significantly. In these cases, longer residence times and multistage neutralization systems will be required.

Neutralization is generally accomplished in one, two, or three stages using either feedback, feedforward, or feedback–feedforward control. Examples of each of the three types of chemical feed systems are shown in Figures 6.80 to 6.82. The choice of control is dependent upon waste characteristics, hydraulic retention time of the reaction vessel, mixing, reaction lags, and other factors. (A discussion of fundamentals and suggested design philosophy of pH control is contained in numerous references, including Refs. 72 and 73.).

The first-stage reaction tank is generally completely mixed, and the subsequent tanks may or may not be mixed depending on the system requirements. Unmixed neutralization tanks can be used for attenuation where short-term fluctuations in pH are dampened strictly through holding time. Typical detention times for each stage of pH control are between 2 and 10 min, with some considerably longer if limestone or another relatively slow reacting chemical is being used for neutralization. The reaction time should also be investigated in the laboratory.

To achieve good mixing, the influent and effluent devices should also be located at opposite sides of the reaction tank, ideally with one at the top of the tank and one at the bottom. If circular tanks are used, then sidewall mixing baffles must be provided. The chemicals either should be added directly at the point of inflow to the system or directly into the mixing vortex. Care should be exercised in the latter case to assure that the chemicals will not corrode the impeller or impeller shaft. The pH probe for feedback control should

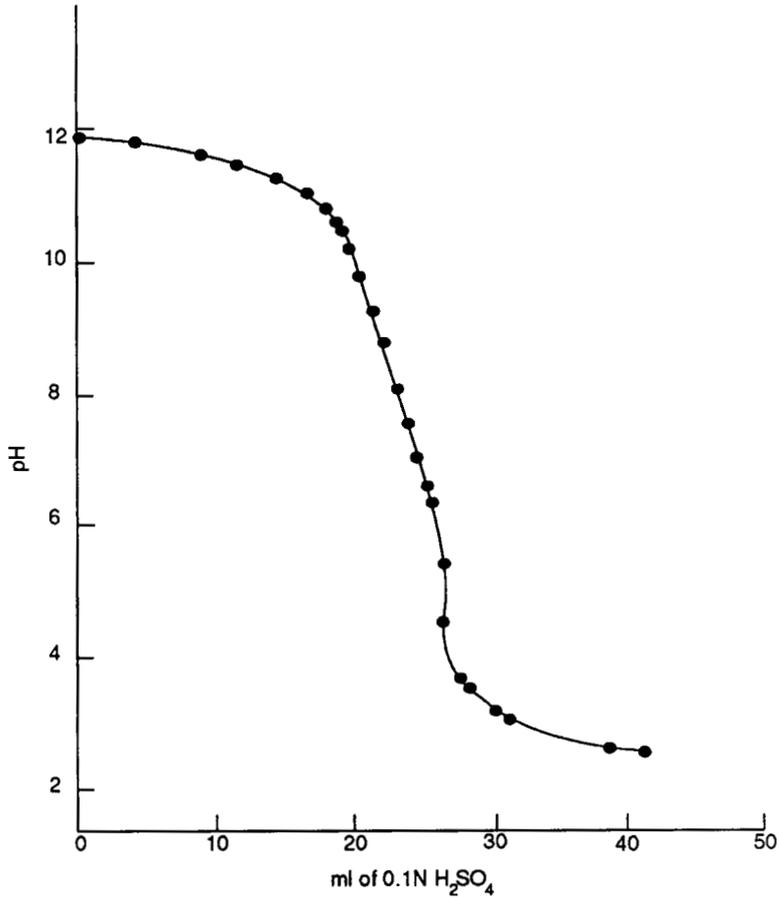


FIGURE 6.79 Titration curve for alkaline wastewater.

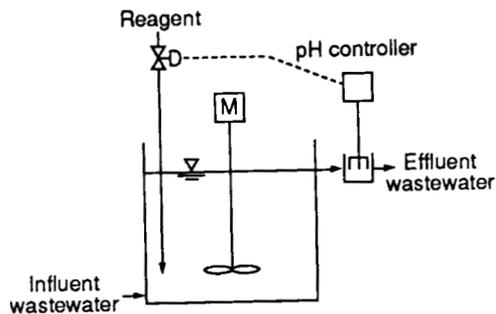


FIGURE 6.80 Feedback mode of pH control.

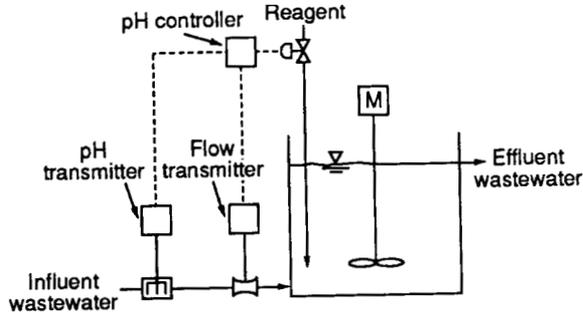


FIGURE 6.81 Feedforward mode of pH control.

be located in the outflow of the reactor just outside the reactor. This provides the most representative sample for process control.

Common neutralizing chemicals are caustic soda, lime, calcium or sodium carbonate, and limestone for acid wastes and sulfuric, hydrochloric, and nitric acids for alkaline wastes. The selection of neutralizing chemical is dependent on the quantity, required cost, availability, and by-product formation, such as sludge, if any. Characteristics of common neutralization reagents are shown in Table 6.88.

The performance of pH control systems is highly dependent on the maintenance of the primary elements (pH probes) and other instruments. If these are in good condition and the system is properly designed, set-point pH values can generally be met. The design of the system is a matter of selecting the proper neutralizing chemical, choosing an adequate reactor retention time, choosing the number of stages, choosing a control concept (feedback, feedforward, or feedforward–feedback), and proper hydraulic design of the system to minimize short-circuiting and assure good pH probe position of accurate measurements. A typical design flow diagram for a two-stage pH control system is shown in Figure 6.83.

Temperature Control

The effects of temperature are most often associated with thermal pollution from power-generating or large industrial facilities. In practice, wastewaters with elevated temperatures have numerous adverse impacts on

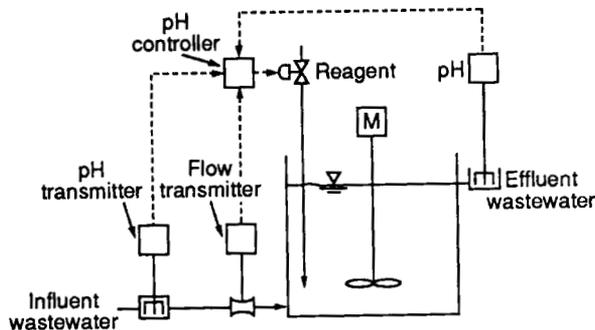


FIGURE 6.82 Feedback-feedforward mode of pH control.

TABLE 6.88 Characteristics of Neutralization Reagents (51)

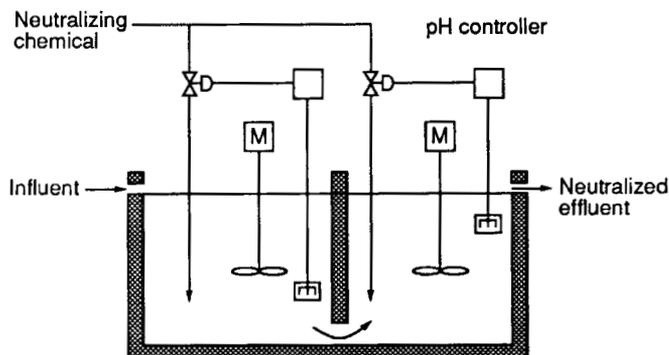
Chemical reagent	Formula	Neutralization requirements, mg/L*	Neutralization factor [†]
Basicity			
Calcium carbonate	CaCO ₃	1.0	1.0/0.56 = 1.786
Calcium oxide	CaO	0.560	0.56/0.56 = 1.000
Calcium hydroxide	Ca(OH) ₂	0.740	0.74/0.56 = 1.321
Magnesium oxide	MgO	0.403	0.403/0.56 = 0.720
Magnesium hydroxide	Mg(OH) ₂	0.583	0.583/0.56 = 1.041
Dolomitic quicklime	[(CaO) _{0.6} (MgO) _{0.4}]	0.497	0.497/0.56 = 0.888
Dolomitic hydrated lime	{[Ca(OH) ₂] _{0.6} [Mg(OH) ₂] _{0.4} }	0.677	0.677/0.56 = 1.209
Sodium hydroxide	NaOH	0.799	0.799/0.56 = 1.427
Sodium carbonate	Na ₂ CO ₃	1.059	1.059/0.56 = 1.891
Acidity			
Sulfuric acid	H ₂ SO ₄	0.98	0.98/0.56 = 1.750
Hydrochloric acid	HCl	0.72	0.72/0.56 = 1.285
Nitric acid	HNO ₃	0.63	0.63/0.56 = 1.125

*The quantity of reagent required to neutralize 1 mg/L of acidity or alkalinity, expressed as calcium carbonate.

[†]Assumes 100% purity of all compounds.

treatment processes. For example, temperature reduces oxygen transfer capacities and increases basin stratification. On the other hand, a minimum temperature is desirable for anaerobic processes. In the sewer system, high temperatures accelerate biological production of hydrogen sulfide and may soften asphalt jointing compounds.

In general, direct discharge of steam and boiler blow-off is prohibited by most sewer use ordinances, and maximum permissible discharge temperatures are in the area of 150°F (65°C). Limits for point sources are more site-specific and tend to be specified in terms of permissible changes in ambient water temperature.

**FIGURE 6.83** Two-stage, continuous neutralization system.

The treatment and/or cooling of wastewaters takes several forms. For high-temperature wastewaters, heat exchangers provide a means for recovering the heat. Tube heat exchangers and shell exchangers are often used for heating and cooling because they can be cleaned easily. However, plate exchangers are used because of high efficiency.

If the heat is simply to be wasted, cooling may be achieved with ponds or towers for liquids and with condensate manholes for steam. For small volumes and low heat loads, cooling ponds and spray ponds are usually effective. Mechanical and natural draft cooling towers are used for larger flows and greater heat loads.

A minimum temperature is required for biological processes. Whereas minimum temperature requirements are generally not a problem for municipal treatment systems, they can be a problem for industrial treatment systems with long hydraulic retention times. In these cases, deep basins with covers are typically employed.

Oil and Grease Removal

Oil and grease must generally be removed from wastewater since these materials can foul instruments and equipment, interfere with other processes (particularly gravity settling), and may accumulate in unwanted areas of the treatment system, causing a hazard or performance problem. Furthermore, oil and grease are very damaging to the environment and, if it were to pass through to a receiving body of water, could cause a significant pollution problem.

Oily wastewaters are produced in petroleum production, refining, and storage; petrochemical complexes; steel finishing mills; metalworking industries; food processing; textile scouring; and cooling and heating. Less significant grease loads tend to come from residential or food preparation sources. The resulting effluents vary widely in volume (1 to 10,000 L/s) and in oil content (100 to 100,000 mg/L). The removal of oil and grease from wastewaters can be accomplished by the use of several well-known and widely accepted techniques. However, the performance of any given separation technique will depend entirely on the condition of the oil–water mixture; therefore, the nature of a particular oily waste stream must be determined before proper treatment can be selected.

The principal factors affecting the design of oil-water separators are

1. The amount of oil present in the wastewater
2. The oil droplet size distribution
3. The presence of surfactants or chemical emulsifiers
4. The specific gravity of the oil
5. The specific gravity of the wastewater
6. The wastewater temperature
7. The concentration of suspended solids.

The primary advantages and disadvantages of several oil separation processes are summarized in Table 6.89. Gravity separation and air flotation are the most well known and widely used.

The waste oil and grease (waste fats) from public and private kitchen and food preparation areas comes from two sources. The concentrated wastes from deep fryers and other vessels are collected separately for by-product recovery or other off-site disposal. Diluted wastes originate from washup and spills; these normally are discharged to a (gravity) grease trap prior to release to the sanitary sewer. Sewer use ordinances typically require grease traps and establish the limits of pretreatment requirements.

Free Oil. Oil and grease present in wastewater as droplets with little or no associated water is referred to as free oil. Free oil will float to the surface due to its low specific gravity.

The three main forces acting on a discrete oil droplet are buoyancy, drag, and gravity. The buoyancy of an

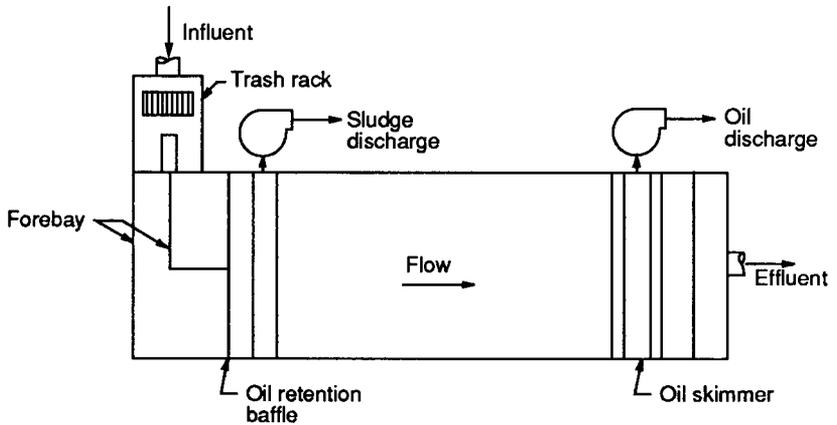
TABLE 6.89 Process Comparison for Oil and Grease Removal

Process	Description	Advantages	Disadvantages
Gravity separation	API, CPI, TPS, PPI	Potential for treatment of suspended solids; effective removal of free and dispersed oil; simple and economical treatment operation	Limited efficiency for removal of emulsified oil; will not remove soluble oil; restricted to the removal of oil droplets greater than 20 µm
Air flotation	DAF, IAF	Potential for treatment of suspended solids; effective removal of dispersed and emulsified oil with chemical addition; reliable process which effectively treats shock loads	Chemical sludge handling when coagulants are used
Chemical flocculation	Used in conjunction with gravity separation and air flotation	Potential for treatment of high suspended solids	Chemical sludge produced
Filtration	Sand, anthracite, multimedia, crushed graphite	Effective suspended solids removal; has application to the separation of free, dispersed and emulsified oil	Requires backwashing and the backwash creates a subsequent treatment problem
Coalescence	Fibrous media	Effective removal of all oil components except soluble oils	High levels of suspended solids will induce fouling; potential for biological fouling; needs extensive pretreatment; not demonstrated as a practical process in full-scale operation
Membrane process	Reverse osmosis, ultrafiltration, hperfiltration	Soluble oil removal demonstrated in laboratory tests	Low flux rates; membrane fouling and limited membrane life; not demonstrated as a practical process in full-scale operation; needs extensive pretreatment
Biological processes	Activated sludge, RBS	Effective soluble oil removal	Requires extensive pretreatment to reduce influent oil levels to less than 40 mg/L
Carbon adsorption	GAC will act both as effective filtration medium and coalescing separator; PAC for removal of soluble oils only	Effective removal of all oil components including soluble oils	Requires extensive pretreatment; process is expensive; carbon must be regenerated or replaced; not demonstrated as a practical process for full-scale operation.

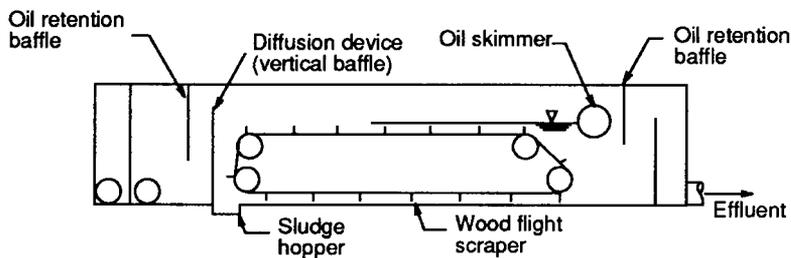
oil droplet is proportional to its volume and the drag is proportional to the projected area of the droplet. As the diameter of an oil droplet decreases, the ratio of its volume to surface area also decreases. Because of this droplet size relationship, larger droplets tend to rise while smaller droplets remain suspended. This relationship is defined by the well-known Stokes law. The concept assumes that the terminal velocity of a rising droplet is: (1) proportional to the specific gravity difference between the oil and water; (2) proportional to the square of the oil droplet diameters; and (3) inversely proportional to the viscosity of the water.

Gravity Separation. A large number of simple gravity oil separation devices are available, varying from API (American Petroleum Institute) separators to parallel-plate separators, as shown in Figures 6.84 and 6.85, respectively. Once the oil is at the surface, it still must be removed from the wastewater. This can be accomplished by slotted pipes; telescoping pipes; dip tubes; parallel plates; or belt, drum, or rope skimmers.

The API has developed procedures to use for design of gravity oil–water separators, based upon experiment and plant operating data (74). The API recommends using Stokes law for calculating the oil globule rise velocity, using an oil globule diameter of 0.015 cm. API, published data for estimating the specific gravity (Figure 6.86, page 6.164), and absolute viscosity (Figure 6.87, page 6.165), when actual measurements are not available; values are for temperature between 40 and 120°F (4 and 10°C). The information in Table 6.90 (page 6.166) is factored into the design of a gravity separator. Flow factors are given in Figure 6.88 (page 6.167).



(a)



(b)

FIGURE 6.84 Gravity oil-water separator. (a) Plan; (b) section.

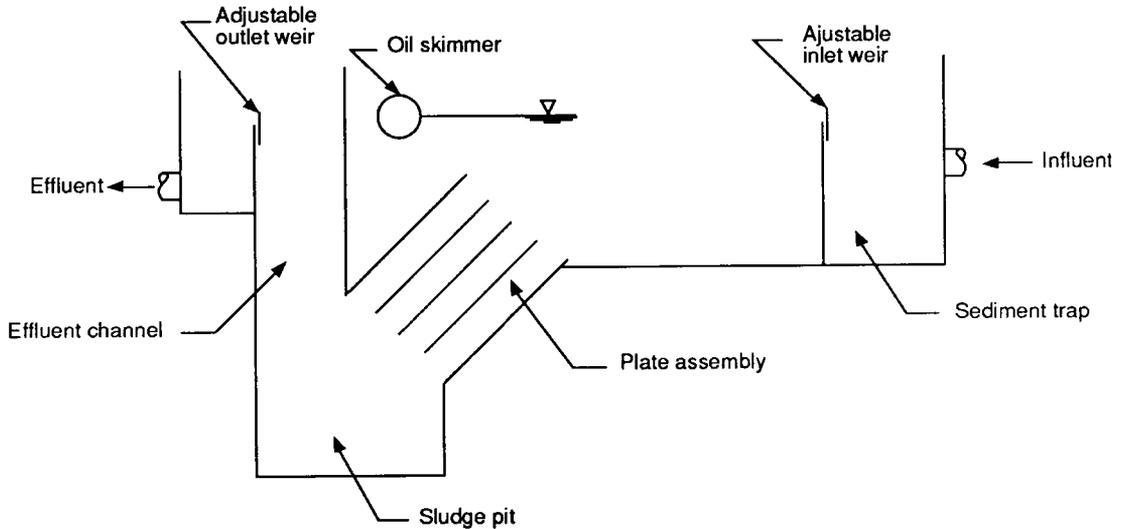


FIGURE 6.85 Parallel plate oil-water separator.

Air Flotation. The second common method of oil and grease removal is through induced (IAF) or dissolved (DAF) air flotation, which are shown schematically in Figures 6.89 and 6.90, respectively. (Flotation processes are discussed in the suspended solids section.) The air flotation process is used where simple gravity separation is not sufficient.

Air is introduced (either at atmospheric pressure or dissolved under pressure) to provide air bubbles, which tend to attach to the oil droplets and then float the oil quickly to the surface. More rapid oil removal can be achieved than by gravity alone, resulting in smaller treatment units. Also, better performance can sometimes be achieved, as the air bubbles may attach to suspended particles that are covered with oil and would not be removed in a simple gravity system. Finally, it is often necessary to use chemical coagulants with flotation units. Chemicals, such as alum, ferric chloride, and cationic or anionic polyelectrolytes, are used to improve the efficiency of oil and grease removal.

Emulsions. Oil may also be present in wastewater in the form of an emulsion. This is where the oil is actually dispersed in the water in a stable fashion. Two types of emulsions are discussed: mechanical emulsions and chemical emulsions. Mechanical emulsions are created through the process of pumping and otherwise mixing the oil-water solution. Chemical emulsions are generally intentionally formed by using chemicals to stabilize the emulsion for an industrial process need or other use. Both types of emulsions can be present in a wastewater at the same time and also along with free oil. Careful sampling, experimental work, and analysis are necessary to differentiate between the various types.

The objective in treatment of wastewater containing emulsified oils is to destabilize the emulsion so that the oil or grease will separate by gravity or flotation. Once the emulsion is broken, the same removal techniques applicable to free oil can be utilized.

Destabilization of a mechanical emulsion can be accomplished by techniques such as coalescence, chemical coagulation, or agglomeration. Essentially what is one is to promote interdroplet contact with the purpose of developing larger droplets that will be much easier to remove. These larger droplets will have a lower specific gravity and therefore separable by use of the techniques previously described.

The treatment of chemical emulsions can be considerably more difficult due to the characteristics of the

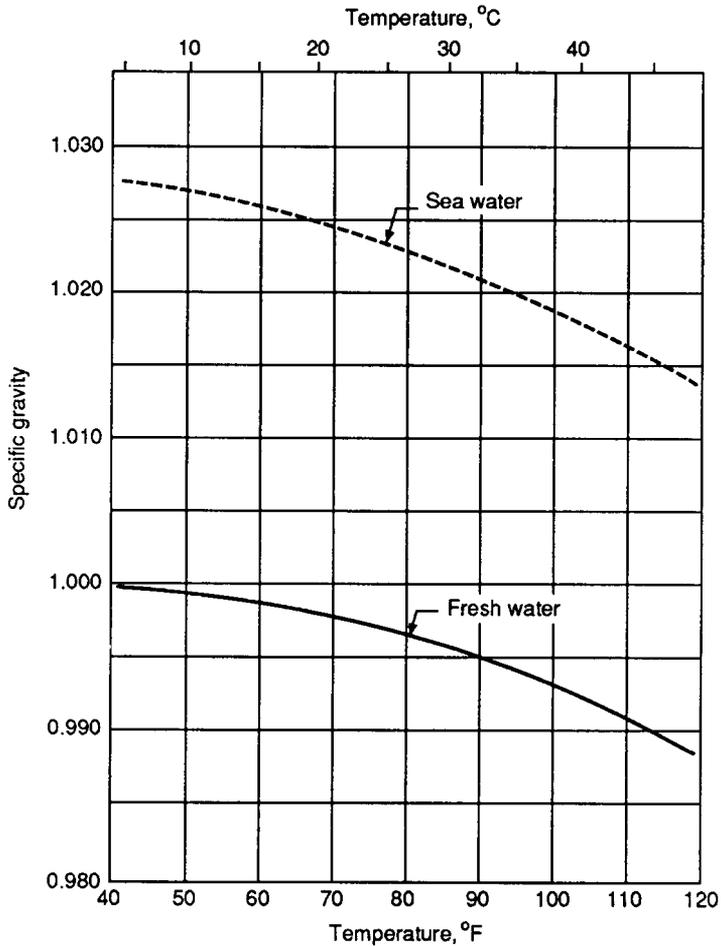


FIGURE 6.86 Specific gravity of clean water (74).

waste, but several tried and proven techniques are available for investigation. These involve the use of coagulants, pH adjustment, and heat. Acid cracking with or without heat is the most common approach used and is generally done on a batch basis. Sulfuric or hydrochloric acid is added to a pH of 1 to 2 along with a coagulant and heating to 100 to 150°F (38 to 66°C) is used to break the emulsion. This results in the oil being free from the water and therefore separable by the other means. Emulsion breaking is highly specific to the particular wastewater and should be carefully tested in the laboratory prior to system design.

Heavy Metals Removal

Treatment of wastewater may involve the removal of soluble or particulate heavy metals such as lead, copper, mercury, and chromium. These materials are used in a large variety of industries and are hazardous to

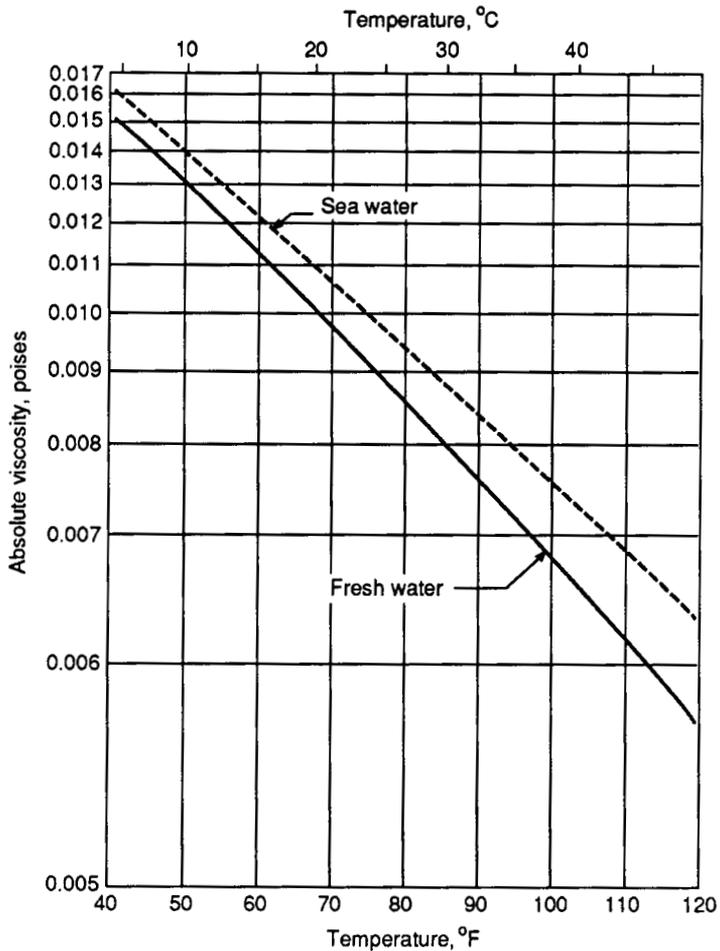


FIGURE 6.87 Absolute viscosity of clean water (74).

the environment because they do not naturally degrade and are toxic even at relatively low concentrations. Fortunately, heavy metals are generally not very soluble in water (depending on the pH, specific metal, and other ions present) and can therefore be removed by physical means, such as settling or filtration. Removal normally requires adjustment of the pH to the proper level to precipitate the metal (alkaline pH for hydroxide precipitation and acidic pH for sulfide precipitation) and then removal of the precipitated solid by coagulation and gravity settling and/or filtration. Removal of metals to levels approaching their solubility concentration for a given pH is theoretically possible. These concentrations are shown for common metals in Figure 6.91 and 6.92. The assumption is that the metals are present as free metallic ions and that no competing reactions prevent the precipitation of the particular ion.

Some metals have unique characteristics that make their removal somewhat more difficult. For example, chromium can be present in either a hexavalent form (Cr^{+6}) or a trivalent form (Cr^{+3}). Hexavalent chromium is very toxic and will not precipitate at alkaline pH as chrome hydroxide. However, Cr^{+3} will precipitate at

TABLE 6.90 Gravity Oil–Water Separator Design

1. Rise velocity

$$V_r = \frac{g(\rho_w - \rho_o)d_p^2}{18\mu}$$

- where V_r = rise velocity of oil droplet
 g = acceleration due to gravity
 ρ_w = density of water, mass/volume
 ρ_o = density of oil, mass/volume
 d_p = oil particle diameter
 μ = viscosity of water

2. Horizontal velocity

$$\text{Maximum } V_h = 15V_r \text{ or } 3 \text{ ft/min (60 m/s)}$$

3. Flow factor. Obtain flow factor F from Fig. 6.88

4. Cross-sectional area

$$A_c = Q/V_r$$

- where A_c = cross-sectional area
 Q = maximum flow

5. Separator depth. Range of separator depth d is 3 ft (0.9 m) to 8 ft (2.4 m)6. Separator width. Range of separator width B is 6 ft (1.8 m) to 20 ft (6.1 m)7. Depth-to-width ratio. Range of d/B is 0.3 to 0.88. Number of channels. Number of channels n is greater than 1 if A_c is more than 160 ft² (15 m²)

9. Horizontal area

$$\text{Minimum } A_s = f(Q/V_r)$$

10. Separator length

$$L = A_s/B = F(V_h/V_r)d$$

alkaline pH, and therefore, in order to remove Cr^{+6} , a first step is to reduce it to Cr^{+3} . This can be done with acid and a strong reducing agent, such as sulfur dioxide or sodium thiosulfate. Once this reduction is completed (generally done in batches in less than 1 h), the chromium can be removed along with the other metals present.

Another feature of certain metals (aluminum and iron particularly) is that they become highly soluble if the pH is raised too high. For example, aluminum forms an insoluble precipitate when the pH is between 6 and 8, but the precipitate redissolves above pH 8, making it impossible to remove at that pH by physical means. These metals are said to be amphoteric, and treatment must take into account this characteristic. Discussions of the individual characteristics of various metal ions and their removal from water are presented in several EPA publications (75–77).

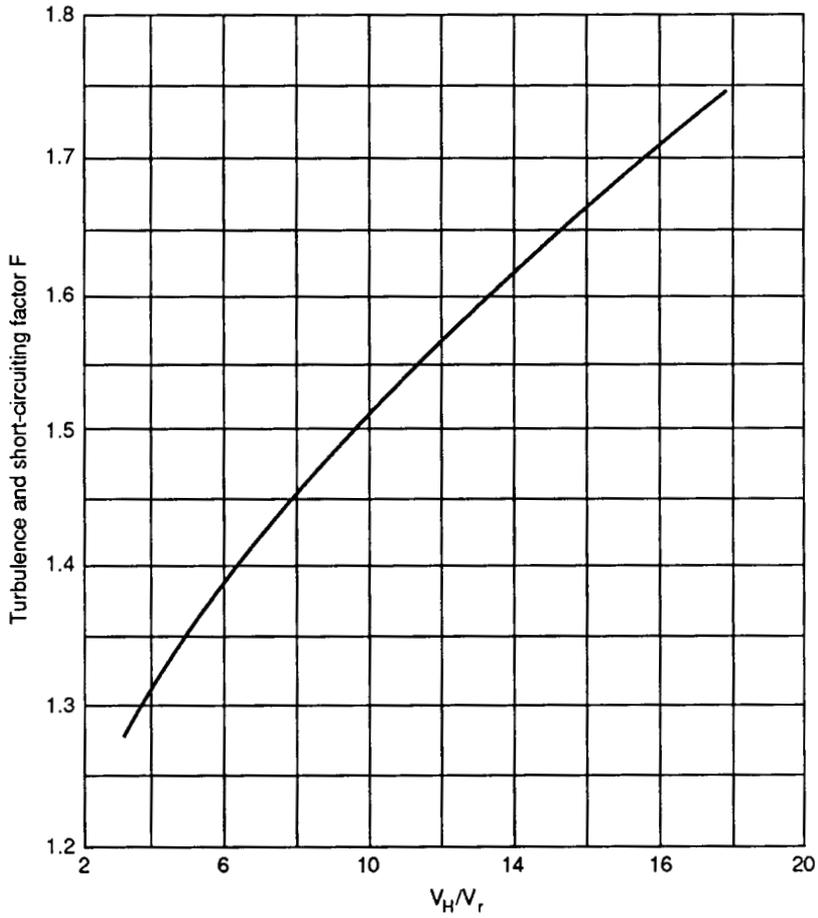


FIGURE 6.88 Turbulence and short-circuiting factor (74).

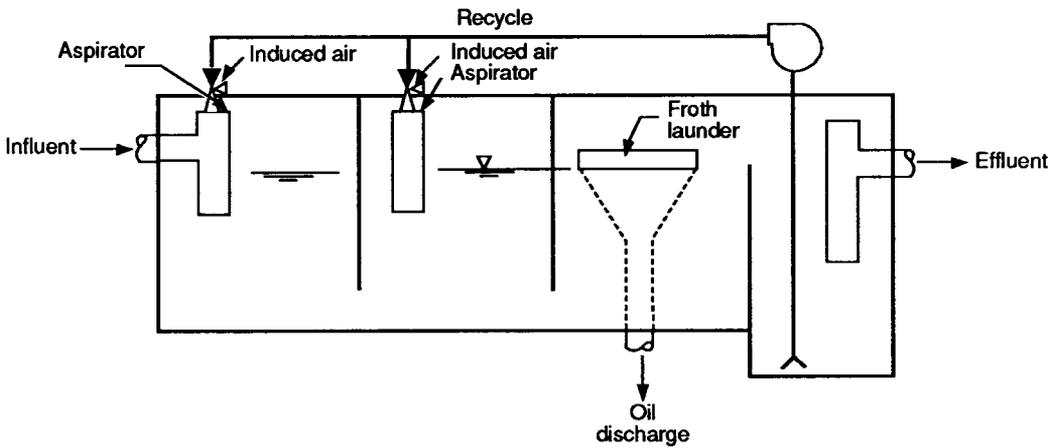


FIGURE 6.89 Induced air flotation.

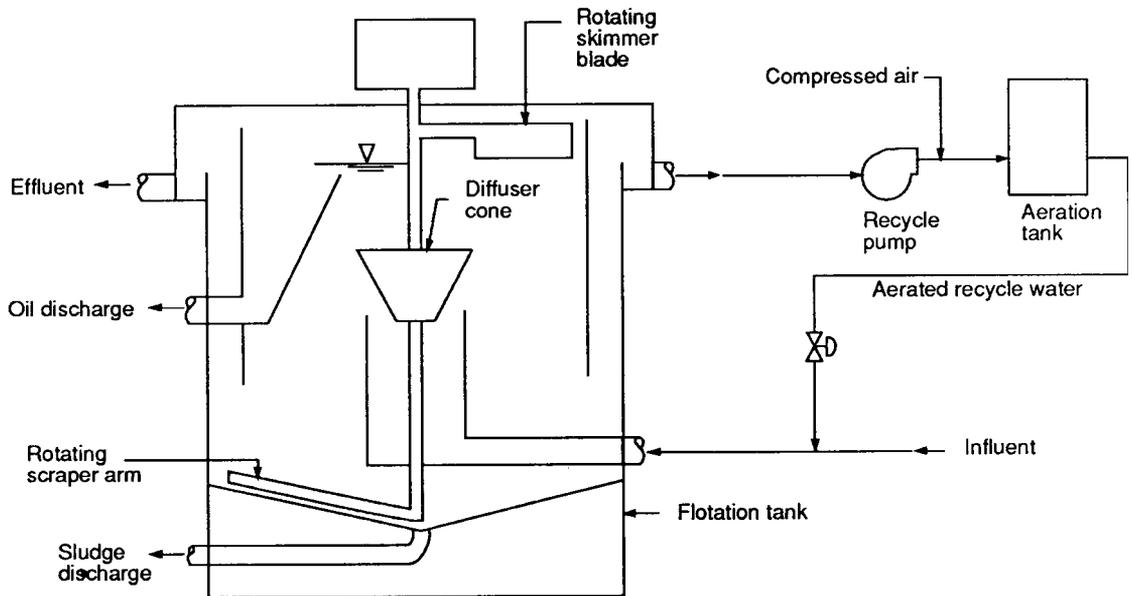


FIGURE 6.90 Dissolved air flotation.

Once the metal precipitate (generally a hydroxide or sulfide precipitate) is formed, chemical coagulation with lime and/or anionic polyelectrolytes is generally used to enhance solids removal by gravity in a clarifier. Typical design criteria for this chemical addition and clarification process are noted in Table 6.91. It should be noted that sludge recycle is often used to promote flocculation and improve solids separation.

It is very important to recognize that the level of treatment of the metals is limited by their solubility in water, and therefore, if a mixture of metals is present, it is likely that the required effluent levels may not be possible at a single pH value. When that is the case, multistate pH adjustment and clarification are required to remove the metals near the optimum pH (minimum solubility). Two stages are generally sufficient, but a third stage composed of final pH adjustment and polishing filters can be used where very stringent requirements must be met.

Another totally different treatment process not depending on precipitation can also be used to remove heavy metals. This is ion exchange, which takes advantage of the ionic nature of the metals and involves the attachment of the ions (mostly cations) to a synthetic resin through charge neutralization.

Post-aeration

The purpose of post-aeration is to provide additional oxygen to the receiving water. Post-aeration can increase plant effluent dissolved oxygen concentration levels to 4 to 6 mg/L from the normal concentrations around 0.5 to 2 mg/L. This additional dissolved oxygen provides for maintenance of stream standards in cases where normal dissolved oxygen discharge levels are not adequate.

The four principal post-aeration techniques are diffused, mechanical, cascade, and U-tube. The techniques differ in structural, mechanical, and operation and maintenance requirements, and in the methods of oxygen transfer (see Figure 6.93). Since post-aeration may create a detergent-type foam, the design should include facilities for retaining and destroying the foam prior to discharge.

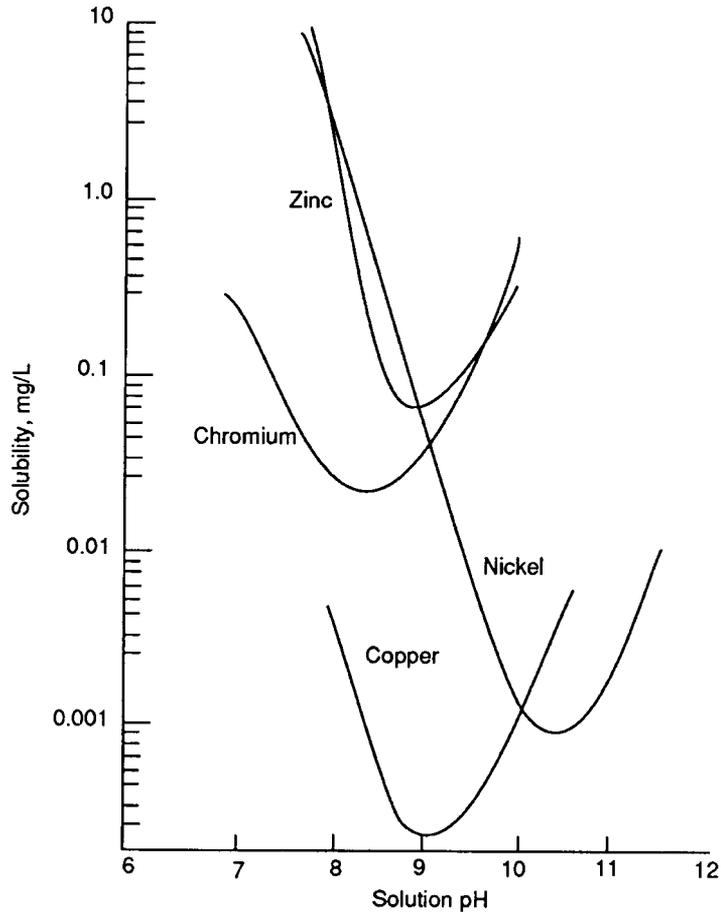


FIGURE 6.91 Solubility of chromium, copper, nickel, and zinc hydroxides.

TABLE 6.91 Heavy Metal Precipitation Data

Parameter	Typical range
pH adjustment (lime, caustic, CO ₃ or HCO ₃)	Selected based on specific metal ion
Anionic polyelectrolytes	0.5–5 mg/L
Rapid mix	1–2 min
Flocculation	2–5 min
Sedimentation	500–1000 gpd/ft ² (20–40 m ³ /m ² ·d)

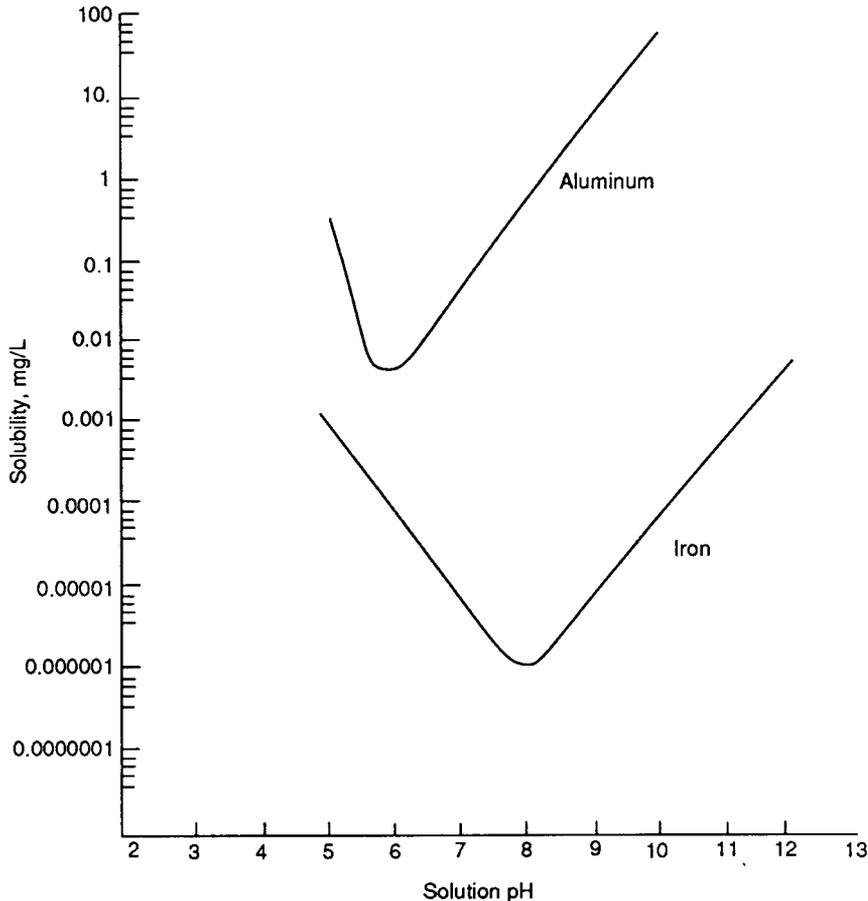


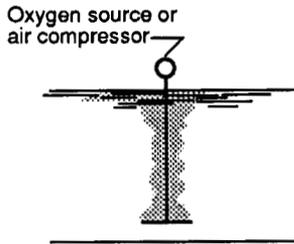
FIGURE 6.92 Solubility of aluminum and iron hydroxides.

Diffused Aeration. The diffused air system relies on the use of diffusers to supply oxygen to the effluent while it is retained in a basin for 10 to 30 min. The amount of oxygen transferred to the water is dependent on a number of factors, with the main ones being the temperature and oxygen level in the wastewater. Table 6.92 contains typical design data for this system.

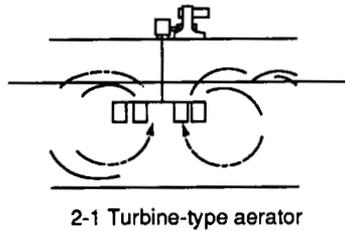
Mechanical Aeration. Mechanical post-aeration can be accomplished using either turbine or pump aerators. Oxygen transfer occurs by the vortexing action imparted by the aerator and by diffusion through the exposure of large volumes of wastewater to the atmosphere. The system design is a function of the aerator horsepower and impeller configuration, which determine the minimum area and the maximum spacing and depth. Normal horsepower ratings can range from 0.03 to 0.1 hp/100 gal (0.006 to 0.02 kW/m³) of tank capacity.

Cascade Aeration. Cascade aeration utilizes flow in thin layers over steps or weirs to maximize turbulence and thus maximize oxygen transfer from the atmosphere to the wastewater. A head of 3 to 10 ft (1 to 3 m) is

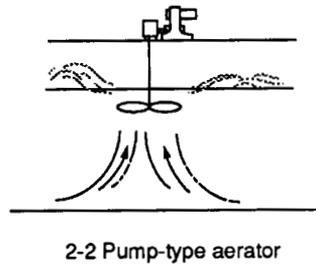
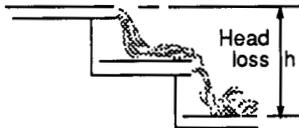
1. Diffused aeration



2. Mechanical aeration



3. Cascade aeration



4. U-tube aeration

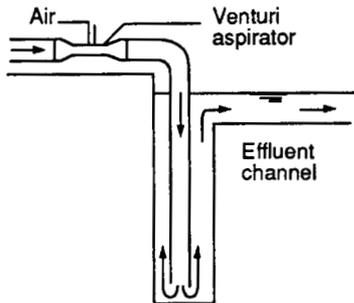


FIGURE 6.93 Post-aeration systems (71).

TABLE 6.92 Diffused Post-aeration Design Data (71)

Parameter	Value
Basin depth	9–15 ft (2.7–4.6 m)
Basin width	10–30 ft (3–9 m)
Ratio of width to depth	1.5 desired, 2.0 maximum
Detention time	10–30 min
Air requirements	0.5–5 ft ³ /min/ft ² (1.5–15 m ³ /min/m ²) of tank area

required, depending on the initial level and desired increase of the dissolved oxygen. If the necessary head is not available, pumping will be required, offsetting the low energy–costs benefits of this method.

U-Tube Aeration. An aspirator or compressor and diffuser may be used to entrain air for a U-tube aerator. The name comes from the use of a U-shaped downturn in the conduit carrying the wastewater; the head requirement for a medium-size plant is about 5 ft (1.5 m). Design considerations also include air-to-water ratio and tube cross-sectional area. This system is not widely used for post-aeration but has been used more often for aeration in force mains.

Disinfection

Disinfection involves the destruction of disease-causing organisms (also see Chapter 5, Water Supply). The most common means of wastewater disinfection is chlorination. Concerns for side effects and/or undesirable by-product formation have increased attention to dechlorination and alternative disinfectants for some discharges.

Recent technology has brought forth new techniques for disinfecting treated sewage, the more popular being ozonation and ultraviolet light. Other available techniques include other chemical disinfectants and their compounds, such as bromine, fluorine, iodine, peroxide, permanganate, phenols, alcohols, surfactants, heavy metals, and acids and alkalis. Thermal and other radiation processes may also be used.

While many systems show promise for the future, chlorination is expected to continue as the primary disinfection process. This is due to its proven performance, relatively low cost, availability in large quantities, toxicity to pathogens, and other factors, such as ability to control odor and to maintain a residual.

Chlorination. Chlorination is the most widely used method of disinfection and is accomplished by application of elemental chlorine, hypochlorites, or chlorine dioxide. Common to each disinfectant is the need for good mixing and contact time, and the effectiveness is monitored by the quantity of residual chlorine.

Rapid dispersion of the disinfectant at the point of addition increases contact time and improves disinfection efficiency. Over-and-under and end-around baffling may be spaced to provide turbulent mixing at the application point and plug flow throughout the remainder of the chlorine contact chamber. If plant hydraulics or other restraints make baffling impractical, mechanical mixing or air agitation can be used effectively. Horizontal velocities of 0.1 to 0.25 ft/s (2 to 4.5 m/min) at minimum flow should be provided to lessen deposition of solids.

After mixing, a contact time of 15 to 30 min at peak flow should be provided. The designer should consult with governing regulatory agencies for their requirements on contact time and effluent residual chlorine. Generally, chlorine residuals of 0.5 mg/L are required.

The sum of the combined and free chlorine concentrations remaining after a specified period of time is the total chlorine residual. The residual should be measured immediately after sampling.

Chlorine demand is the difference between the chlorine applied and the residual chlorine and provides a measure of disinfection and all other chlorine-demanding reactions for the contact period. Thus, the chlorine dosage must satisfy both the demand and residual requirements. Typical ranges of chlorine dosage are presented in Table 6.93; actual chlorine requirements for a given plant should be determined by appropriate field studies. Chlorination equipment should be sized to feed approximately 50 to 100 lb Cl₂ per 10⁶ gal (85 to 170 kg Cl₂ per 10³ m³) treated.

Chlorine. Elemental chlorine combines with water to form hypochlorous (HOCl) and hydrochloric (HCl) acids. The hydrolysis goes virtually to completion at pH values and concentrations normal to municipal waste waters. Hypochlorous acid and hypochlorite (OCl⁻) ions are known as free available chlorine, and hypochlorous acid is an extremely active disinfectant at pHs of 6.5 to 7.5. By maintaining a pH below 7.5, the tendency of hypochlorous acid to dissociate to hypochlorite ion can be discouraged. Also, chlorine combines with ammonia in wastewater to form monochloramine, which is a persistent but relatively slow-acting disinfectant.

TABLE 6.93 Chlorine Dosages for Disinfection (51)

Wastewater to be disinfected	Chlorine dosage, mg/L
Raw sewage	6–12
Raw sewage (septic)	12–25
Settled sewage	5–10
Settled sewage (septic)	12–40
Chemical precipitation effluent	3–10
Trickling filter effluent	3–10
Activated sludge effluent	2–8
Sand filter effluent	1–5

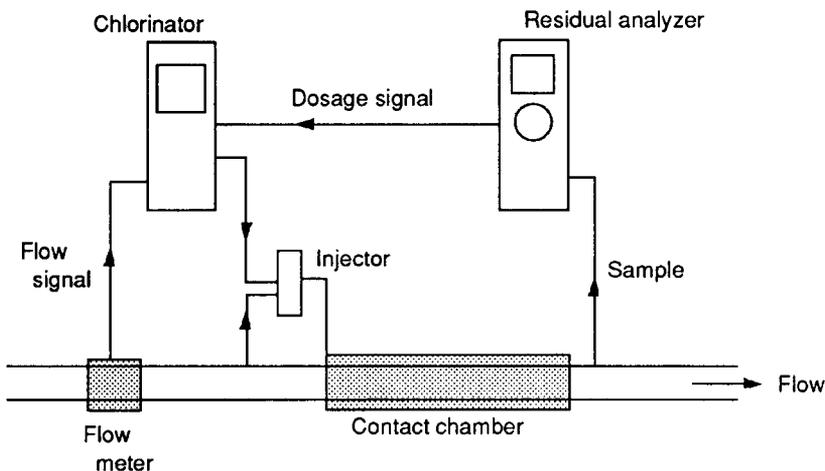
There are numerous configuration of chlorine control systems. The one illustrated in Figure 6.94 shows a chlorinator paced by the wastewater flow rate and the effluent chlorine residual. The amount of chlorine added is determined by container weight loss.

Typically, liquefied chlorine gas is delivered to the plant in pressurized containers. Standard container sizes in the United States are 100-lb (45.4-kg), 150-lb (68.1-kg), and 1-ton (0.9-Mg) cylinders, and 16-ton (14.5-Mg), 30-ton (27.2-Mg), and 55-ton (49.9-Mg) tank cars.

In most cases, gaseous chlorine is released from the top of the container and is drawn through the chlorinator by a vacuum created by the injector. (Separate evaporators are required in larger plants where the evaporation rate within the container cannot meet the demand.) The chlorine gas is then mixed with treated effluent or potable water in the injector to form a chlorine solution, which is control-fed into the mixing portion of the chlorine contact chamber.

Hypochlorites. The two most common hypochlorite salts are calcium and sodium hypochlorite, which are commercially available in dry and liquid form, respectively. In terms of available chlorine, hypochlorites have the same oxidizing and disinfecting potential as chlorine gas. Also, with exception of the feeder, storage, and some piping, a hypochlorination system is very similar to that using chlorine.

The choice between chlorine and hypochlorites depends mostly on safety considerations and economics.

**FIGURE 6.94** Chlorine control system.

Hypochlorites are safer and preclude the danger of storing liquefied chlorine gas at plants with close proximity to populated areas. Capital costs are similar, but operating costs for hypochlorites are higher.

The most popular form of hypochlorite is sodium hypochlorite, which can be produced on-site (limited applications) or delivered in 500 to 5000 gal (1,900 to 19,000 L) tank cars or trucks. The concentration delivered on-site is typically 12 to 15% by weight of available chlorine. The tank cars or trucks may be unloaded to storage tanks from which chemical feeders regulate the quantity applied to the wastewater.

Chlorine Dioxide. Chlorine dioxide (ClO_2) is an unstable and extremely corrosive gas. It is usually produced at the site by mixing a solution of sodium chlorite with strong chlorine in contact with water to assure the gas remains in solution and to avoid an explosion hazard.

Like chlorine and hypochlorites, chlorine dioxide is a strong oxidizing agent and an excellent disinfectant. Chlorine dioxide has the advantage of being effective over a much broader pH range. Also, it does not react with ammonia to form less effective chloramines. Drawbacks to the broad use of chlorine dioxide are its relatively high costs and the lack of a suitable test for low residual measurements.

Dechlorination. There is increased awareness of the potential toxic effects to humans and aquatic life by the formation of halogenated organic compounds through the chlorination of water supplies and wastewater effluents. Thus, dechlorination systems are becoming more and more common. Dechlorination may be accomplished by using reducing agents, such as sulfur dioxide or sodium sulfite, activated carbon, sunlight, prolonged storage, or aeration for certain volatile forms of chlorine.

Sulfur Dioxide. Of the alternatives for dechlorinating larger flows, sulfur dioxide (SO_2) has the most developed technology, the most effective proven performance, and the least expense. The process involves dissolving sulfur dioxide into water, where it quickly forms sulfurous acid which reacts almost instantaneously with free and combined chlorine. The reaction yields small amounts of sulfuric and hydrochloric acids, which are neutralized by the wastewater's buffering capacity.

Sulfur dioxide is fed as a gas with equipment similar to that used for chlorine. As shown in Table 6.94, feed rates of 1 mg/L of sulfur dioxide to residual chlorine may be used for design. Contact times of up to 5 min in a mixed contact tank are desirable, but in-line mixing systems are often adequate, due to the practically instantaneous reaction.

Depending on the degree of dechlorination required, two types of sulfur dioxide systems may be used. One is a closed-loop feedback system that dechlorinates to a fixed residual level above zero, say 0.1 mg/L. The other type is an open-loop feedforward system that dechlorinates to essentially a zero residual level. A schematic of these systems appears in Figure 6.95.

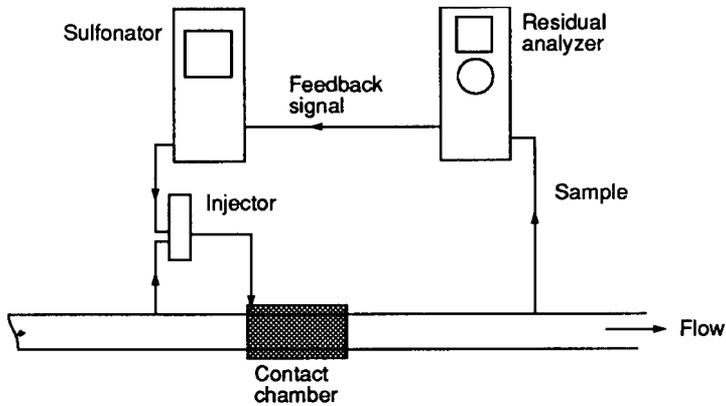
Since sulfur dioxide is a reducing agent, overdose can lead to reduced dissolved oxygen in the treated effluent. As a result, post-aeration and/or reaeration may be required to meet effluent dissolved oxygen requirements. In such cases, the capital and operating costs are increased four to six and two to four times, respectively.

Sulfites. For treatment facilities with flows less than 1 mgd ($3800 \text{ m}^3/\text{day}$), sulfite salts are practical alter-

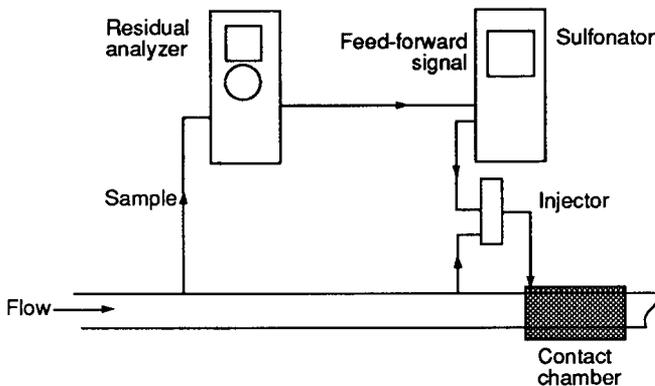
TABLE 6.94 Dechlorination Chemical Feed Rates (50)

Chemical	Typical feed rates, lb/lb (kg/kg) of residual chlorine
Sulfur dioxide	1.1
Sodium sulfite	0.57
Sodium bisulfite	0.68
Sodium thiosulfate	1.43

Note: Contact time, 1 to 5 mm.



(a)



(b)

FIGURE 6.95 Sulfur dioxide dechlorination systems. (a) Closed-loop feed-back system; (b) Open-loop feed-forward system.

natives for dechlorination. The more common forms are sodium sulfite (Na_2SO_3), sodium bisulfite (NaHSO_3), and sodium metabisulfite ($\text{Na}_2\text{S}_2\text{O}_5$). As with sulfur dioxide, these chemicals react almost instantaneously with available chlorine.

The sulfite salts may be applied as a solid or liquid via manual or automatic feed systems. Provisions for mixing and for testing residual chlorine should be provided. Table 6.94 provides typical feed rates for sulfite salts.

Activated Carbon. Dechlorination with activated carbon is a physical process wherein sorption is used to remove free chlorine, chlorinated amines, and chlorinated organics. This approach has the added benefit of

removal of residual refractory organics and possibly some of the toxic chlorinated organics. This process is very effective and reliable and is easy to operate.

The equipment and design principles of advanced wastewater treatment applies to systems for dechlorination. A typical activated carbon dechlorination bed is designed for a wastewater application of 3 gpm/ft² (2.9 m³/m²min) and a contact time of 15 to 20 min.

Since the capital and operating costs of dechlorination with activated carbon are relatively very high, this process has limited potential when chemical processes may be used. It is likely to be most feasible when used after wastewater treatment using carbon or other processes for high-level removal of organics.

Ozonation. Because of its many years of use in Europe to disinfect water supplies and its high oxidation potential, ozone has received much attention as an alternative to chlorine. Ozone equals chlorine in ability to kill bacteria and may be superior in ability to inactivate viruses. Also, ozone reduces wastewater color and odor, does not produce dissolved solids, is not affected by pH, and provides an oxygenated effluent.

There are results available from numerous pilot plant studies and a few full-scale operations that use ozone for disinfections. For best results, filtration or other tertiary treatment processes should precede the ozonation process, since oxidizable organics may consume ozone faster than the rate of disinfection. Current performance data should be sought by the design engineer.

Since ozone is a relatively unstable gas and breaks down into elemental oxygen relatively quickly, it is generated on site with an ozonator which applies an electrical charge across air or oxygen. Automatic devices are available to control voltage, frequency, gas flow, and moisture.

Wastewater ozonation may be accomplished by mechanical mixing, countercurrent or concurrent flow columns, diffusers, or injectors. Dosage requirements and contact time will vary with the wastewater characteristics, but common ranges are 1 to 10 mg/L and less than 15 min, respectively.

Typical flow diagrams for the three ozone disinfection systems are presented in Figure 6.96. Since about half the power is required to generate ozone from oxygen as from air, covered contact chambers with oxygen recycle should be investigated.

Deterrants to the use of ozone are the relatively high capital and operating costs and the lack of residual protection. Manufacturers and researchers are working to bring down the costs, and postchlorination may be used, or required by regulatory agencies, to provide residual protection, especially with respect to water supplies.

Ultraviolet Irradiation. Application of ultraviolet irradiation for wastewater disinfection has advanced to become technically and economically competitive (high capital but low operating costs) with chlorine-based systems. A proper dosage of ultraviolet radiation is an effective bactericide and virucide and does not contribute to the formation of toxic compounds (50, 115, 116).

At present, the low-pressure mercury arc lamp is the principal means of generating ultraviolet energy used for disinfection. The mercury lamp is favored because about 85% of the light output is monochromatic at a wavelength (λ) of 253.7 nm, which is within the optimum range (250 to 270 nm) for germicidal effects. To produce ultraviolet energy, the lamp, which contains mercury vapor, is charged by striking an electric arc. The energy generated by the excitation of the mercury vapor contained in the lamp results in the emission of ultraviolet light. When first used, multiple lamps were grouped horizontally in modular units; now, a vertical configuration is also available.

Ultraviolet light is a physical rather than a chemical disinfecting agent. The radiation penetrates the cell wall of the microorganism and is absorbed by cellular materials, including DNA and RNA. This either prevents replication or causes death of the cell to occur. Cellular damage proceeds via a series of photochemical reactions, which require that light energy of the proper wavelength(s) is available and is absorbed by the reacting species.

The ultraviolet disinfection system design must consider wastewater characteristics and system hydraulics. In particular, upstream treatment process performance is essential to reliable service. Because the

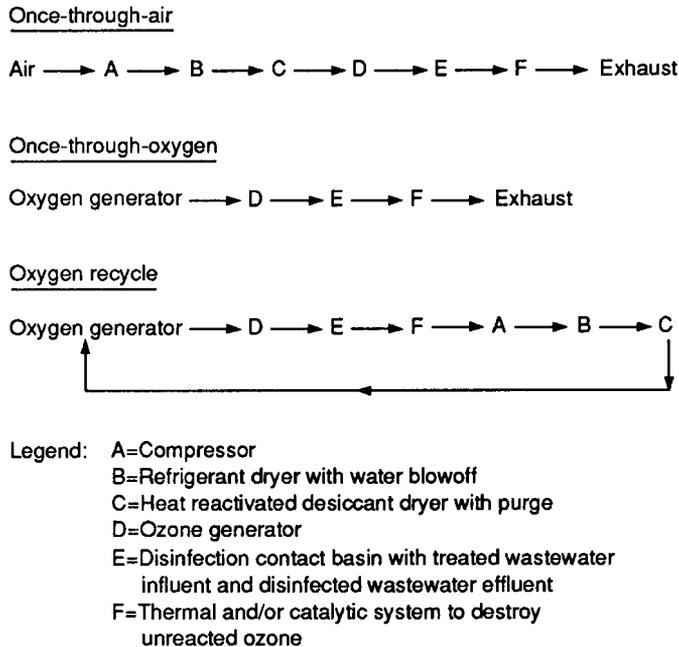


FIGURE 6.96 Ozone disinfection processes.

only ultraviolet radiation effective in destroying bacteria is that which reaches the bacteria, the water must be relatively free from turbidity that would absorb the ultraviolet energy and shield the bacteria. Also, since the distance over which ultraviolet light is effective is very limited, most effective disinfection occurs when the flow passes by as a thin film. To limit the liquid thickness the ultraviolet light must penetrate, most ultraviolet units are constructed with an array of ultraviolet lamps through which the wastewater is passed.

All wastewaters will attenuate ultraviolet radiation to some degree. The attenuation of ultraviolet radiation by the wastewater itself contributes to the decrease in the intensity of ultraviolet radiation with distance away from an ultraviolet lamp. Therefore, microorganisms that pass near an ultraviolet lamp are more likely to be killed or inactivated than those that do not come near a lamp. As the ultraviolet transmittance (an analytical measure of the ability of a medium to transmit light of a given wavelength through a known path length) decreases, "available" ultraviolet intensity decreases, resulting in poorer disinfection efficacy. Empirical observations have revealed that ultraviolet systems are able to achieve adequate disinfection with common lamp geometries when ultraviolet transmittance measures 65% or more, ($\lambda = 254 \text{ nm}$, path length = 1.0 cm), although adequate disinfection has been achieved for transmittances below this level.

The advantages of ultraviolet irradiation are that no chemicals require handling or are induced into the flow and that there is no potential for development of taste and odor problems. Practical applications include special-purpose potable water disinfection, such as shipboard or isolated resorts, and wet industrial-type operations, such as breweries.

A significant shortcoming of ultraviolet irradiation, especially for disinfecting municipal wastewaters, is that it provides no residual to inhibit recontamination. Thus, regulatory agencies may require standby backup systems. Other disadvantages are that turbidity, color, and/or organic compounds affect the effectiveness of this process.

The economics of ultraviolet irradiation show a relatively high initial cost but a very low operation and

maintenance cost. Cost evaluations must consider availability of capital versus operating funds and of costs for extra facilities, such as prefiltration and standby disinfection. Thus, future proven performance will affect requirements and costs before ultraviolet irradiation is more widely applied.

ADVANCED TREATMENT

Treatment processes are available for control of specific organic and inorganic pollutants. They may be used as independent systems or as advanced wastewater treatment systems. The more common processes are for removal of nutrients, nitrogen and phosphorus, and of suspended solids by filtration. Other more special-purpose processes include carbon adsorption, ion exchange, reverse osmosis, and deep-well injection.

Nitrogen Control

Excess ammonia and/or nitrate in aquatic systems accelerates the onset of eutrophication. Microbial activity requires 4.57 mg of oxygen to oxidize 1 mg of ammonia; the effect may be detrimental to water quality. Also, health problems can be caused by high levels of nitrate in drinking water supplies. Thus, removal of nitrogenous compounds may be a necessity to maintain water quality standards.

The control of nitrogen in wastewater begins with nitrification for oxidization of ammonia-nitrogen and, if required, ends with denitrification for reduction of nitrates and nitrites to nitrogen gas (50, 78–80). The more common control of nitrification may be accomplished biologically or by such processes as ammonia stripping, breakpoint chlorination, or ion exchange. Denitrification is typically achieved through biological treatment. The more common treatment stages for biological nitrogen removal are illustrated in Figure 6.97.

Wastewater temperature and pH are important design considerations. As illustrated in Figure 6.98, these parameters affect the distribution of ammonia and ammonium ions. Biological processes typically function on ammonium conversion at moderate pHs, while an ammonia air-stripping system requires higher pHs. Temperature influences loading and performance criteria.

Nitrification. Biological nitrification can be accomplished by single-stage or separate-stage suspended-growth systems or by attached-growth systems. The two-step process begins with ammonium conversion to nitrite by *Nitrosomonas* bacteria, which is followed by nitrite conversion to nitrate by *Nitrobacter* bacteria.

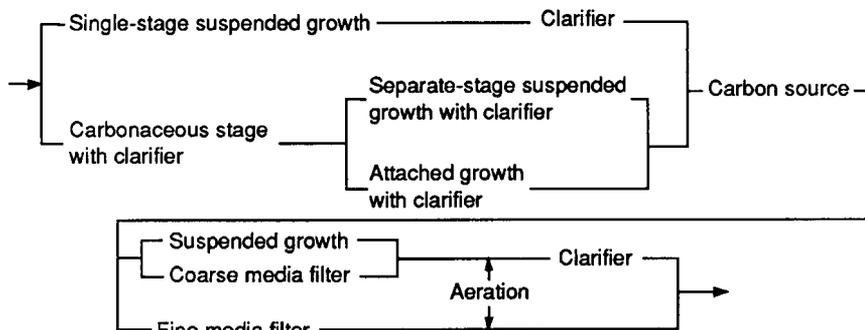


FIGURE 6.97 Biological nitrogen control.

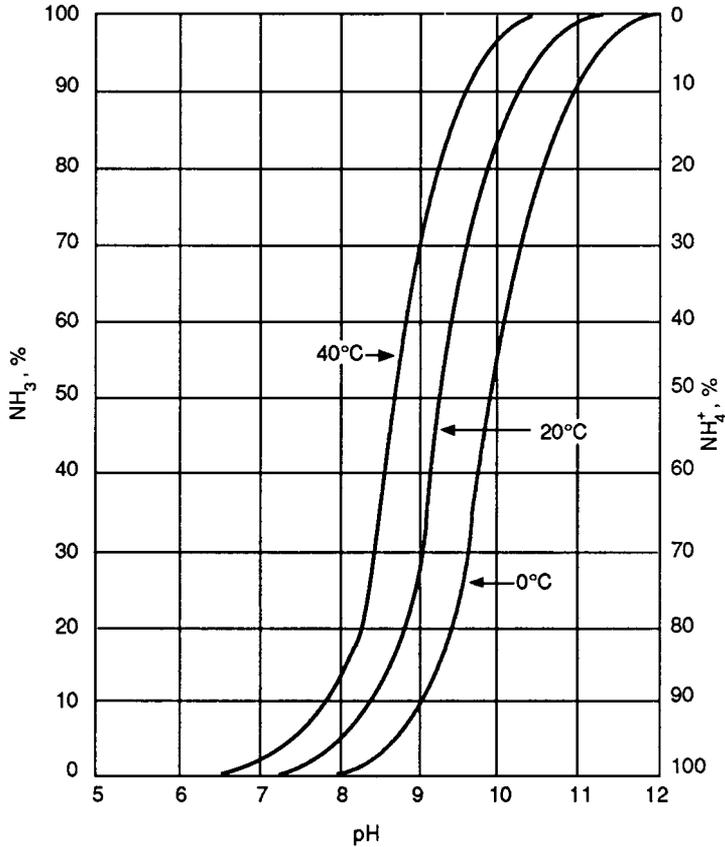


FIGURE 6.98 Influence of pH and temperature on ammonia and ammonium distribution (78).

The relatively slow growth rate of *Nitrosomonas* bacteria limits the nitrification process. Both organisms are most efficient at temperatures of 85 to 95°F (30 to 36°C), pHs of 8.0 to 8.5, and dissolved oxygen of near 2.0 mg/L.

Activated sludge systems are designed to nitrify by assuring the following:

1. The sludge age is sufficient to allow maintenance of the nitrifying organisms. This will be at least five days and substantially more under conditions of reduced temperatures, such as 50°F (10°C). For industrial wastes, the sludge age should be experimentally determined.
2. No toxic or inhibitory materials are present that prevent or suppress the maintenance or growth of nitrifying organisms.
3. Proper pH, dissolved oxygen, and other environmental conditions to satisfy the organism's basic metabolic requirements.

Under these conditions, activated sludge systems are often preferred over attached-growth systems, which generally have poorer carbonaceous BOD removals and a less-controllable process.

An attached-growth system is more difficult to design because of a lack of knowledge and control of sludge age. Empirical approaches are possible, based on experience in pilot and full-scale facilities for domestic wastewaters. For nontypical domestic or industrial wastewaters, an experimental approach to determine the design basis should be used.

As a second-stage process, attached-growth systems (e.g., packed column, RBC, or fluidized bed) following activated sludge for carbonaceous BOD removal are often feasible and offer the advantage of maintaining nitrification without the need to clarify by gravity and recycle biological solids. Second-stage suspended-growth systems are difficult to control with respect to solids separation and sludge age.

Single-Stage Suspended Growth. Single-stage nitrification uses a single aeration basin for oxidation of ammonia and carbonaceous material. The design conditions associated with this activated sludge modification are presented in Table 6.95.

The sludge retention time is especially important due to the relatively slow growth rate of the nitrifying organisms. The other key factors in the degree of nitrification are temperature and the dissolved oxygen concentration.

Since nitrification requires about 7.5 mg/L of alkalinity per 1 mg/L of ammonia nitrogen oxidized, lime additions may be required to supplement natural alkalinity. This reaction is also temperature-dependent, and the dosage requirements should be determined on a case-by-case basis.

Separate-Stage Suspended Growth. In separate-stage nitrification, different processes are used for removal of carbonaceous material and nitrification. The primary advantage of this system is that it provides an opportunity to optimize the hydraulic and organic loading of both processes. The tankage requirements for this system are illustrated in Figure 6.99, and typical design data are presented in Table 6.96.

Separate-stage nitrification has similar sensitivities, such as temperature and dissolved oxygen conditions, to the single-stage process. However, separate-stage nitrification operates under the favorable condition of lower BOD₅ concentrations and does not require as great a sludge age (10 to 20 days). Also, plug flow is preferable for the aeration basin flow scheme.

Attached Growth. The use of trickling filters, rotating biological contactors, or other attached-growth systems for nitrification is limited. A major concern is process control in general, and varying seasonal temperatures in particular. Often, at least two-stage treatment is required.

Design data are not well defined for attached-growth systems. However, some general findings for trickling filters are that nitrification occurs best at low organic loadings, 6 lb BOD/1000 ft³/day (0.1 kg BOD/m³/day) or less and at hydraulic loadings around 50 gpd/ft² (2 m³/day/m²).

Denitrification. Facultative heterotrophic organisms perform denitrification by reducing nitrates and nitrites to nitrogen gas. Since the wastewater is usually low in carbonaceous material, a supplemental carbon

TABLE 6.95 Single-Stage Nitrification Design Data (50)

Parameter	Typical value
Temperature	86°F (30°C) or higher desired
pH	8.0–8.5
F/M ratio	0.05–0.15
Aeration detention time	18–24 h
Sludge retention time	20–30 days
MLSS	3000–6000 mg/L
Volumetric loading	5–15 lb BOD ₅ /1000 ft ³ (0.08–0.24 kg BOD ₅ /m ³)
Dissolved oxygen	1–2 mg/L

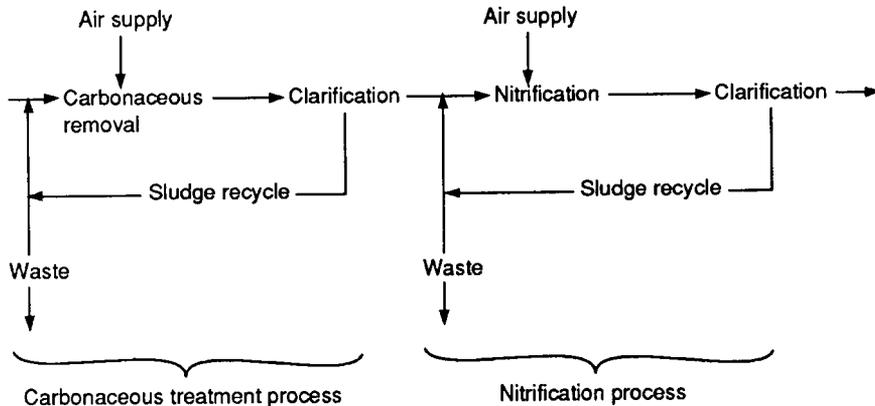


FIGURE 6.99 Separate-stage nitrification system.

source, commonly methanol, is required. Methanol is usually applied at 2 to 4 lb (1 to 2 kg) per pound (kg) of nitrate removed, and the dosage should be carefully adjusted with respect to the nitrate concentration and temperature. Raw or partially treated wastes may be used as the carbon source, but they yield a less efficient process.

Suspended- or attached-growth systems may be used for denitrification. Due to the anaerobic conditions for denitrification, effluent aeration for 5 to 10 min is recommended.

Suspended Growth. Denitrification by a suspended-growth system is comprised of an anaerobic reactor, which may be covered and which is equipped with flocculating-type mixers, and a settling tank with provisions for sludge recycle. Table 6.97 lists typical design data. Also, temperature and a carbon source must be incorporated into the design.

Coarse-Media Attached Growth. Denitrification filters using coarse natural or synthetic media is an attached-growth process using anaerobic conditions in a submerged filter bed (i.e., water plant type). A wide variety of media, including gravel, stone, and plastic, may be used as long as the bed contains a high void volume and low specific volume. In addition to temperature and carbon source considerations, typical design data for a downflow filter (packed column) are presented in Table 6.98. An alternate upflow (fluidized bed) scheme may be used. The contact time must be greater, and the hydraulic loading rate less.

The features of a coarse-media filter will permit sloughing biomass to increase suspended solids in the

TABLE 6.96 Separate-Stage Nitrification Design Data (50)

Parameter	Typical value
Aeration	
pH	7.6–7.8
MLVSS	1500–2000 mg/L
Dissolved oxygen	1.0 mg/L minimum
Return sludge	50–100%
Clarification	
Surface loading rate	400–600 gpd/ft ² (16–24 m ³ /day/m ²)
Solids loading rate	20–30 lb/day/ft ² (100–150 kg/day/m ²)
Detention time	0.5–3 h

TABLE 6.97 Suspended-Growth Denitrification Design Data (50)

Parameter	Typical value
Reactor	
pH	6.5–7.5
MLVSS	1000–3000 mg/L
Mixer power requirements	0.25–0.5 hp/1000 ft ³ (6.5–13 kW/1000 in ³)
Clarifier	
Depth	12–15 ft (3.75–4.7 m)
Surface loading rate	400–600 gpd/ft ² (16–24 m ³ /day/m ²)
Solids loading rate	20–30 lb/day/ft ² (100–145 kg/day/m ²)
Return sludge rate	50–100%
Detention time	Around 2 h

effluent sufficiently so that some additional clarification is required. Backwashing of the filter may be required to control suspended solids.

Fine-Media Attached Growth. Fine natural, synthetic, or multimedia may be used to provide an attached-growth denitrification filter. Anaerobic conditions must be maintained in the pressurized, submerged bed. The process requires a carbon supplement and is temperature-sensitive. Table 6.99 provides typical design data for a downflow filter (packed column). Upflow filters (fluidized bed) may be considered.

The fine media retain biomass slough and provide a low suspended-solids effluent concentration. However, periodic backwashing is required to remove slough and to maintain an acceptable head loss through the filter. The finer the media, the more frequent backwashing is necessary.

TABLE 6.98 Coarse-Media Denitrification Design Data (50, 78)

Parameter	Typical value
Media diameter	Over 15 mm (0.6 in)
Voids	70–95%
Specific surface	65–275 ft ² /ft ³ (215–900 m ² /m ³)
Surface loading rate	2.5–4.0 gpd/ft ² (100–165 L/day/m ²)
Loading rate (NO ₃ -N rem/packing surface)	0.5–1.3 × 10 ⁻⁴ lb/ft ² (2.4–6.3 kg/m ²)
Backwash	Infrequent

TABLE 6.99 Fine-Media Denitrification Design Data (50, 78)

Parameter	Typical value
Medium diameter	0.08–0.6 in (2–15 mm)
Voids	40–50%
Specific surface	85–300 ft ² /ft ³ (280–985 m ² /m ³)
Surface loading rate	720–10,000 gpd/ft ² (30–410 m ³ /day/m ²)
Backwash frequency	0.5–4 days
Backwash rate	8–25 gpm/ft ² (470–1465 m ³ /day/m ²)

TABLE 6.100 Air Stripping Design Data (50)

Parameter	Typical value
Wastewater loading	1–2 gal/min/ft ² (60–120 m ³ /day/m ²)
Airflow rate	300–500 ft ³ /gal (2245–3740 m ³ /m ³)
Wastewater pH	10.8–11.5
Air pressure drop	0.015–0.019 in H ₂ O per foot (0.0125–0.0158 cm H ₂ O per meter)
Tower packing	
Material	Plastic or wood
Spacing	2 in (0.8 cm) horizontal and vertical

Air Stripping. Ammonia can be removed from wastewater by converting ammonium ions to ammonia, by raising the pH, and then freeing the ammonia by passing the wastewater through an air-stripping tower. A packed tower is used with a countercurrent of air drawn through bottom openings. Typical design criteria are presented in Table 6.100.

This process is very air temperature sensitive. For example, ammonia removals of 90 to 95% may be obtained at 68°F (20°C), but at 50°F (10°C), the efficiency drops to around 75% (50). Also, efficiency is poor when ammonia concentrations are below 10 mg/L. Other forms of nitrogen are not removed by this process.

Breakpoint Chlorination. Ammonium ions can be oxidized by chlorine to produce nitrogen gas. At the same time, hydrochloric acid is also produced; this requires the addition of lime or caustic soda for neutralization. The acidic coproduct reaction can be avoided by use of sodium hypochlorite in lieu of chlorine.

The process is named with respect to the level (breakpoint) of chlorine needed for ammonium oxidation. The chlorine dosage control is most important in order to minimize by-product formation, such as chlorinated hydrocarbons, and production of residual chlorine and excessive chloride concentrations. Also, pH must be closely controlled. Typical design data are presented in Table 6.101.

If chlorine residuals are excessive, a downstream dechlorination or chemical change to hypochlorite is needed. Sulfur dioxide or activated carbon are common additives for dechlorination.

Breakpoint chlorination is relatively low in capital costs but can be very high in operating cost, especially for higher (above 15 mg/L) ammonium concentrations.

TABLE 6.101 Breakpoint Chlorination Design Data (50, 78, 79)

Parameter	Typical value
pH	6–7
Contact time	30 min
Oxidizing chemical dosage	
Chlorine/ammonium ion	8–13
Sodium hypochlorite/ammonium ion	9–14
Neutralization chemical dosage	
Lime/chlorine	0.9–1.1
Caustic/chlorine	1.5
Base/sodium hypochlorite	Not required

Ion Exchange. Selective ion exchange can be used with clinoptilolite medium to remove ammonium ions. This application is best suited for use in very cold locations or when higher removal rates are required than available from other methods.

The medium must be regenerated periodically by air stripping, electrolyte (sodium chloride) treatment, or strong acids, such as sulfuric or hydrochloric. Removal and disposal of spent regenerant is an important design consideration.

Ion exchange typically achieves ammonia removal rates of 93 to 97% at flow rates up to 6 gpm/ft² (350 m³/day/m²). More particulars on ion exchange design are presented in a separate section.

Phosphorus Control

Unlike carbon and nitrogen, which may enter a waterway from the atmosphere, phosphorus enters a waterway principally from erosion and wastewaters. As a result, phosphorus contributions may be controlled. Improved farming practices and land use planning, in conjunction with zoning and sediment control ordinances, can be used to reduce erosion and, thus, controlling phosphorus. Numerous approaches may be used to control phosphorus in municipal and industrial wastewater discharges (16, 50, 81).

The selection of a wastewater phosphorus control program begins with determination of phosphorus reduction requirements or effluent limitations, characterization of wastewater conditions, and cost-effective analyses of alternatives.

Phosphorus Limitations. Municipal wastewaters generally have phosphorus concentrations between 10 and 30 mg/L. The effluent control requirements may be expressed as a percentage reduction of phosphorus, or in terms of a specific effluent limitation, such as 1.0 mg/L.

Extensive field studies are necessary to determine the amount of phosphorus that is significantly contributing to the acceleration of the rate of eutrophication of a body of water. Studies are complicated because of the contribution of phosphorus from erosion, as well as the constant exchange of phosphorus between the living and nonliving components of the water body. As technology is refined, the effluent limitations can be better defined for phosphorus control.

Characterizations. The three principal forms of phosphorus in wastewater are orthophosphate, polyphosphate, and organic phosphorus. Significant changes occur to the phosphorus forms throughout a biological treatment process; however, in a secondary effluent, most of the phosphorus is present as orthophosphate. The orthophosphate form is the easiest to remove by chemical precipitation.

The water quality of the raw municipal wastewater, as well as the quality of the wastewater at various locations within an existing treatment facility, should be determined. Of the many constituents which must be considered, the common parameters examined are

- Alkalinity
- Carbon dioxide
- Dissolved oxygen
- Hardness
- pH
- Orthophosphate
- Total phosphorus
- Temperature

In addition, the ionic constituents of the wastewater should be examined to identify those that may influence a subsequent treatment program.

Conventional sedimentation processes are effective in the removal of suspended (insoluble) forms of

phosphorus; however, only a portion (cell uptake) of dissolved (soluble) phosphorus is removed in a conventional biological treatment process. Thus, the dissolved phosphorus compounds are those which must receive special attention and which must be removed in order to meet effluent limitations. Phosphorus reduction may be achieved by influent, treatment, and/or effluent controls.

Influent Controls. Social acceptance is a major factor to be considered in any control alternatives dealing with influent controls of phosphorus. Normally, in-depth investigations of influent phosphorus control will be dependent upon circumstances where the phosphorus level of the wastewater may not be sufficiently reduced with available funds or raw wastewater contains unusually high concentrations of phosphorus. For cases such as these, the influent control of phosphorus may be by control of phosphorus in detergents or by control of major phosphorus point sources, such as industrial connections. The detergent industry has made contributions to influent controls by the production of detergents with little or no phosphates.

Treatment Process Controls. The quantity of phosphorus in the effluent from a wastewater treatment plant can be reduced by changes in operation of existing treatment processes, plant modifications to add new (supplemental) treatment processes, and/or expansion of the treatment facility. Evaluation of each method should include special attention to waste solids handling and ultimate disposal of solid and liquid residues.

Treatment process controls for phosphorus removal have minimal or significant impacts on a plant's normal operation and maintenance program. In larger plants, much of the system may be automated and require only minimal attention. The standard operations for chemical precipitation would include the refilling of chemical tanks, adjusting of pumps, changing of recorder charts and monitoring of wastewater quality in the treatment process and effluent. Regardless of the treatment process control approach, additional sampling and testing must be conducted and additional time must be allotted for review of general operating conditions for the entire treatment facility.

The existing plant safety program should be carefully reviewed as part of the implementation of a phosphorus control program. If hazardous chemicals are to be used in a process, special care must be taken to protect plant personnel, as well as provide a spill contingency plan.

Wastewater treatment plant operations, such as air supply, solids loading, and flow pattern, can be modified to enhance reduction of phosphorus. For example, a low sludge age permits removal of chemically bound phosphorus as well as insoluble phosphorus removed by settling.

Aerobic conditions favor the conversion of the soluble phosphorus to an insoluble state and should be maintained for high-phosphorus-content wastewater. On the other hand, if waste activated sludge is returned to a primary clarifier (low dissolved oxygen), phosphorus will be released from the sludge to the liquid phase.

The operations approach to phosphorus control may not satisfy effluent requirements, but because of its relative ease of implementation and low costs, it should be considered in conjunction with other alternatives.

Frequently, existing wastewater treatment plants will be modified by the addition of specific treatment processes, such as chemical precipitation, sorption, and ion exchange. One or more of these processes should be considered in the analysis of alternatives for phosphorus control.

Expansion of existing wastewater treatment plants may be considered as an alternative with the operations and/or modifications approach. The treatment and removal concepts discussed earlier are considered again with this alternative. The major difference is that posteffluent treatment is examined and new construction will be considered.

Generally, the expansion alternative will be most desirable in cases where effluent limitations require more efficient removal of biochemical oxygen demand and suspended solids, as well as phosphorus. Also, there may be cases where space is limited and the wastewater may need to be transported to a new treatment site for additional treatment. As with the other alternatives, solids handling must be examined.

The expansion alternative may consider the diversion of all or part of a plant effluent to a new drainage basin where the phosphorus limitation does not exist. Also, the alternative should consider land application of the effluent or discharge to another treatment system.

Chemical Phosphorus Removal. Chemical precipitation is the most common treatment process for phosphorus control. Following a review of wastewater characteristics, equipment requirements, economics, and other factors, a choice of coagulant(s) and point(s) of application for a chemical precipitation process can be made. Lime, aluminum and iron salts, and polymers are used frequently. These chemicals may be applied at the influent or effluent of a primary clarifier, the influent or effluent of an aeration basin; the influent or effluent of a final clarifier, and/or a point to enter the liquors from the sludge-handling system.

To assist local governments, a phosphorus removal model (REMOVE) was developed for the EPA to provide planning-level cost estimates for achieving a designated level of phosphorus removal. There are 22 treatment schemes listed in Table 6.102 that can be selected and evaluated by the model.

The model also considered four levels of phosphorus reduction. The levels of effluent (total unfiltered) phosphorus concentrations were assumed to be achievable depending upon the type of treatment provided, as shown in Table 6.103. The treatment scheme numbers refer to those in Table 6.102. This planning tool may be used as a first step in development of alternatives for chemical phosphorus treatment. The new or retrofit conditions, availability and cost of chemicals, and sludge disposal options also are important factors in alternative development.

The quantity of chemical coagulant varies with several factors, such as phosphorus concentration, alkalinity, and/or pH. Typical coagulant dosages are shown in Table 6.104. For metal salts, the usual dosage range is a 1:3 metal ion–phosphorus molar ratio. More project-specific dosage rates should be determined by laboratory jar tests and confirmed by plant scale trials.

The pH range for best precipitation of phosphorus varies with the chemical aid used. For example, the range for alum effectiveness is 5.5 to 6.5. For iron salts, the pH range is 4.5 to 5.0 and 7.0 to 8.0 for ferric and ferrous ions, respectively. With lime, the typical pH range is 9.0 to 10.0.

Each point of chemical application requires provisions for chemical storage, solution tanks, mixers, and feeders. As a result of the chemical precipitants and an increase in precipitation of biomass, the require-

TABLE 6.102 Chemical Precipitation Schemes for Phosphorus Removal (81, 82)

-
1. Alum and polymer addition to primary settler
 2. Ferric chloride and polymer addition to primary settler
 3. Ferrous chloride and lime addition to primary settler
 4. Lime addition to primary settler
 5. Alum and polymer addition to flocculation basin
 6. Ferric chloride and polymer addition to flocculation basin
 7. Ferrous chloride and lime addition to flocculation basin
 8. Lime addition to flocculation basin
 9. Alum addition to aeration basin
 10. Ferric chloride addition to aeration basin
 11. Sodium aluminate addition to aeration basin
 12. Alum addition to aeration basin and multimedia filtration
 13. Ferric chloride addition to aeration basin and multimedia filtration
 14. Sodium aluminate addition to aeration basin and multimedia filtration
 15. Alum addition after trickling filter
 16. Ferric chloride addition after trickling filter
 17. Alum addition after trickling filter and multimedia filtration
 18. Ferric chloride addition after trickling filter and multimedia filtration
 19. Alum addition to flocculation basin after conventional secondary treatment
 20. Ferric chloride addition to flocculation after conventional secondary treatment
 21. Lime (one-stage) addition to flocculation after conventional secondary treatment
 22. Lime (two-stage) addition to flocculation basin after conventional secondary treatment
-

TABLE 6.103 Chemical Phosphorus Removal Treatment Options (81, 82)

Effluent concentration	Treatment option	Treatment scheme
2.0 mg/L	Option 1. Chemical addition to the primary clarifier.	Schemes 1–4
	Option 2. Chemical addition to a flocculation basin prior to the primary clarifier.	Schemes 5–8
	Option 3. Chemical addition to the aeration basin.	Schemes 9–11
	Option 4. Chemical addition after the trickling filter.	Schemes 15–16
0.5 mg/L	Option 5. Chemical addition to the aeration basin plus multimedia filtration or Chemical addition after the trickling filter plus multi-media filtration.	Schemes 12–14
	Option 6. Addition of lime to a flocculation basin following conventional secondary treatment.	Schemes 17–18 Scheme 21
0.3 mg/L	Option 7. Addition of alum or ferric chloride to a flocculation basin following conventional secondary treatment—one stage.	Schemes 19–20
0.1 mg/L	Option 8. Addition of lime to a flocculation basin following conventional secondary treatment—two stage.	Scheme 22

ments for solids handling facilities should be a major consideration in evaluation of the chemical precipitation approach.

Biological Phosphorus Removal. Biological phosphorus removal (BPR) technologies are modifications of the activated sludge process that convert soluble phosphorus to its insoluble form. Most BPR systems can reliably achieve effluent concentrations below 2.0 mg/L and frequently approach 1.0 mg/L. New technology and process modifications show promise of improved performance. Factors common to good BPR performance are high soluble BOD to soluble phosphorus ratio, low dissolved oxygen and nitrate concentrations in the anaerobic/anoxic zone, and high simple organic compound concentrations in the anaerobic zone. High sludge yield and effective phosphorus control in solids handling processes also are necessary (81, 117, 118).

Influent wastewater characteristics and effluent limitations are the first issues considered in the BPR process selection. If not in place, a phosphate detergent ban should be considered as it not only reduces the phosphorus load but also improves BPR performance by increasing the BOD:P ratio. Common complications are that most applications involve retrofitting existing facilities. Uncertainties in planning for BPR are the impacts of patented processes and licensing fees and the definition of “proven performance.”

To meet stringent treatment objectives required for water quality protection, chemical or biological phosphorus removal processes can be used. The choice of process depends on many factors, but biological phosphorus removal is becoming increasingly popular due to cost and safety issues and requirements on residuals disposal. Table 6.105 compares advantages and disadvantages of BPR with respect to chemical phosphorus removal systems.

TABLE 6.104 Coagulant Dosages for Phosphorus Removal (51)

Coagulant	Dosage, mg/L
Aluminum sulfate	75–250
Ferric chloride	45–90
Lime	200–400
Sodium aluminate	75–150

TABLE 6.105 Advantages and Disadvantages of BPR Processes

Advantages	Disadvantages
Reduced operating costs	Difficult to achieve phosphorus levels <1 mg/L
Limited chemical dependence	Sensitive to shock loadings and upsets
Avoids risks of on-site chemicals	Requires higher operations skills
Reduced sludge production	May release phosphorus during sludge handling
Greater biosolids fertilizer value	Not well suited for retrofits

Biological phosphorus removal can be achieved by producing environmental conditions suitable for the proper microorganisms to grow and metabolize. To develop the proper culture of organisms and maintain them, the mixed liquor must be exposed to a sequence of anaerobic and aerobic conditions.

In the anaerobic zone, the microorganisms exist in a harsh environment with limited available food. Available substrate is utilized by the organisms for maintenance or stored inside the microorganisms while phosphorus is released. Anaerobic conditions and simple organic compounds are necessary for phosphorus release to occur.

The primary microorganism responsible for phosphorus release, *Acinetobacter*, is sensitive to the presence of chemical oxygen (nitrates) and dissolved oxygen. To reduce the concentration of these compounds in the process stream, anoxic (no dissolved oxygen present, but nitrates are present) zones may be encouraged. Under these conditions, nitrates are removed by conversion to nitrogen through denitrification.

Following anaerobic conditions, the mixed liquor passes to aerobic zones. Under these favorable conditions, the *Acinetobacter* actively grows and metabolizes. Because the microorganisms contain large quantities of stored substrate and the mixed liquor contains substrate, the energy requirements for growth and metabolism are high. As a result of the energy needs, phosphorus uptake in excess of that released earlier occurs. The microorganisms now store phosphorus—luxury uptake—which is then removed from the system as waste sludge.

Under stressed conditions, *Acinetobacter* prefer simple substrates to complex ones. Simple compounds, such as acetate, are easier to utilize and require less energy for metabolism. Volatile fatty acids (VFAs) are simple compounds, easy to metabolize, and stimulate these organisms to release phosphorus. VFAs, sometimes called short-chain fatty acids, are low molecular weight organic acids. They are a product of the first stages of anaerobic digestion.

In the design of biological phosphorus removal systems, one of the key parameters is the ratio of soluble BOD to phosphorus in the wastewater. If this ratio exceeds 10 to 1, effluent soluble phosphorus levels of 1 mg/L or less are achievable. With low-strength wastewaters, the ability to meet this criterion is marginal, but high-rated (1000 gal/day/ft² (40 m³/day/m²)) primary clarifiers may be used to provide the necessary substrate to the secondary process with or without acetate or VFA addition.

Fermentation of primary sludge for the express purpose of promoting BPR is increasing in popularity. By anaerobically treating solids from the primary clarifier, a VFA-rich mixture can be generated. Primary sludge fermentation (PSF) systems may utilize a complete-mix fermenter followed by gravity thickeners with fermented sludge recycle. Existing primary clarifier or anaerobic digester structures may be converted for use as the basic tankage with equipment and piping modifications to function as PSF units. A PSF system will produce 0.2 lb VFA/lb TSS (0.2 kg VFA/kg TSS). This level of production can be achieved with a hydraulic residence time of 12 to 15 hours and a solids residence time of 2 to 6 days.

There are a number of biological phosphorus/nutrient removal processes. The more common BPR processes include:

- A/O process—anaerobic/oxic process
- Phostrip process—sidestream process for low BOD:P ratios
- UNC process—University of North Carolina process

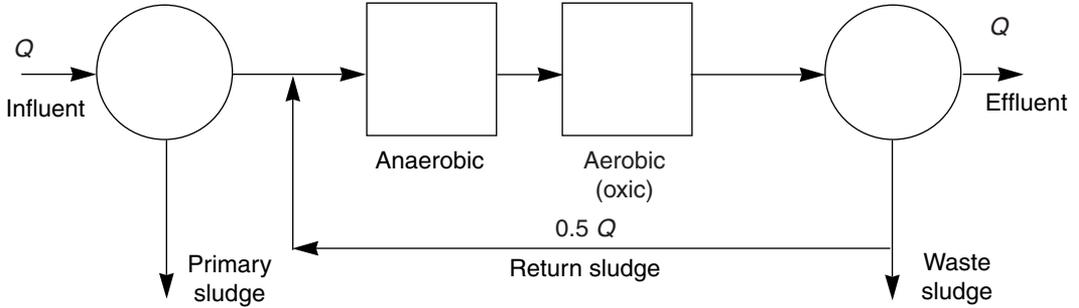


FIGURE 6.100 A/O process flow diagram.

In the A/O process, secondary influent and return activated sludge (RAS) pass through the anaerobic zone where phosphorus release occurs. Phosphorus uptake follows in the aerobic zone. During secondary clarification, phosphorus-rich sludge is collected for recycle (RAS) or wasted (WAS) to sludge handling processes. The A/O process is shown in Figure 6.100 and is characterized by high organic loadings and relatively short solids retention time (SRT).

The Phostrip process compliments a mainline activated sludge process as a sidestream process incorporat-

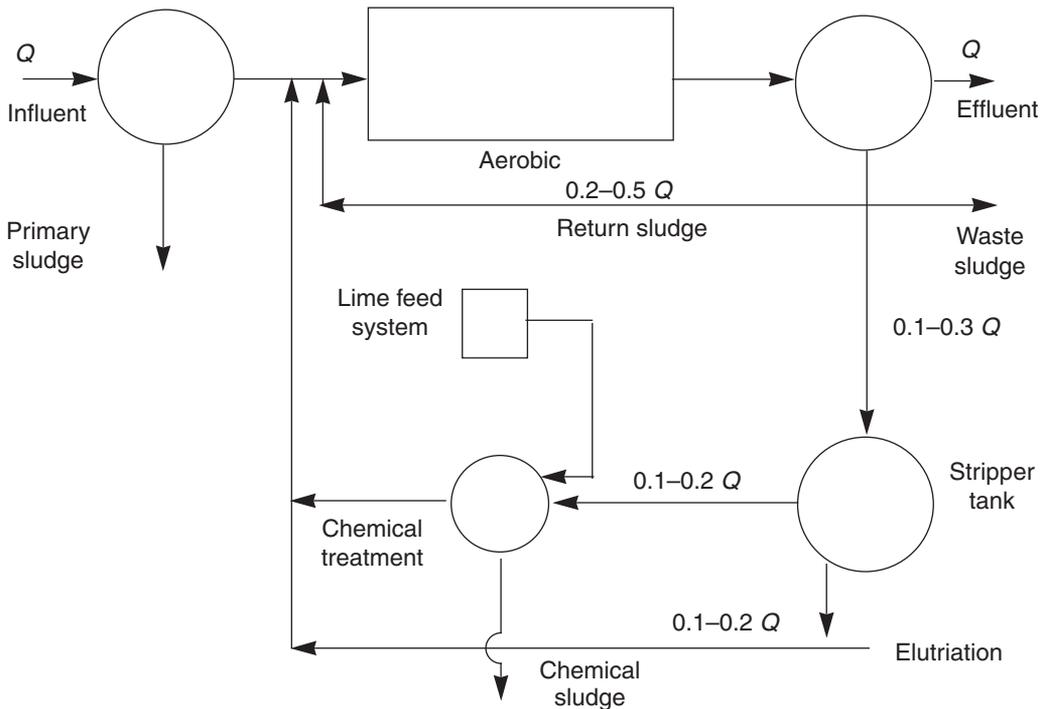


FIGURE 6.101 Phostrip process flow diagram.

ing both biological and chemical phosphorus removal methods. A process flow diagram is presented in Figure 6.101. A portion of the RAS is diverted to a stripper tank where phosphorus is released under anaerobic conditions. Elutriation is used to wash soluble phosphorus from the RAS solids before they are returned ahead of the aerobic zone. The stripper overflow is treated for chemical phosphorus removal with lime in separate tanks or primary clarifiers. Phosphorus removal also is achieved by wasting activated sludge.

The UNC process configuration, Figure 6.102, returns activated sludge from the secondary clarifier to an anoxic zone for nitrate conversion. Following this zone, the RAS enters an anaerobic zone for phosphorus release. The UNC process is capable of producing the desired results, even for low-strength domestic wastewaters. The soluble carbon in the RAS can serve as an organic source for the microorganisms. If adequate phosphorus release does not occur, acetate or VEAs can be added to the anaerobic zone.

A review of biological phosphorus removal processes shows that most of the processes have similar hydraulic residence time (HRT) criteria. This means that a design could encompass several processes to account for changes in wastewater characteristics or regulatory requirements. The typical range of HRTs by process zone is:

- Anoxic zone 1–2 hours
- Anaerobic zone 1–2 hours
- Aerobic zone 4–8 hours

For the mainline processes (A/O and UNC), the solids retention times and recycles rates vary. The SRT and recycles for the A/O process are 2–25 days and 25–40%, respectively. For the UNC process the SRT and recycles are 8–12 days and 50–100%, respectively.

As a safeguard against biological upsets or mechanical failures, standby/backup chemical feed systems typically are provided. Chemicals also may be used to trim BPR effluents to meet permit conditions. With permit limits below 1 mg/L, chemical trim and/or tertiary filtration are provided to ensure effluent quality control.

Nutrient Control

Because of its direct relation to in-stream oxygen demand, nitrification is widely practiced for conversion of ammonia nitrogen to nitrates. In sensitive watersheds, denitrification is required for complete removal of the nutrient nitrogen. In the United States, phosphorus removal also has become a high priority. Initially, phosphorus was deemed the limiting nutrient on discharges impacting impoundments. More recently, regulators

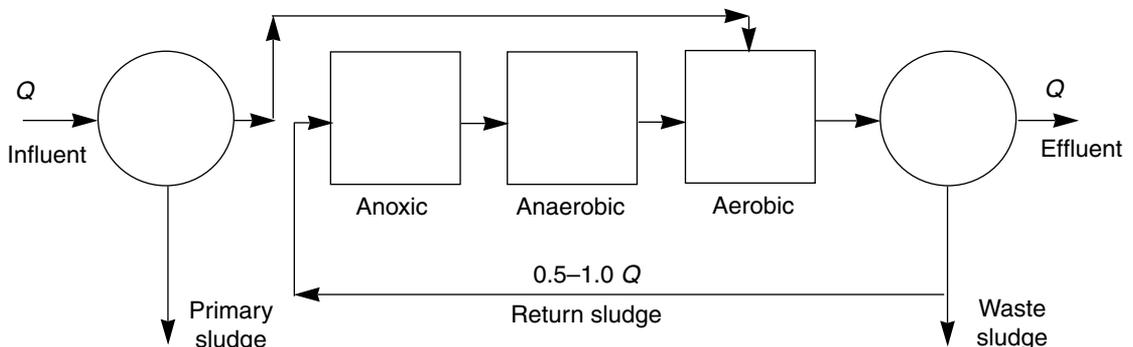


FIGURE 6.102 UNC process flow diagram.

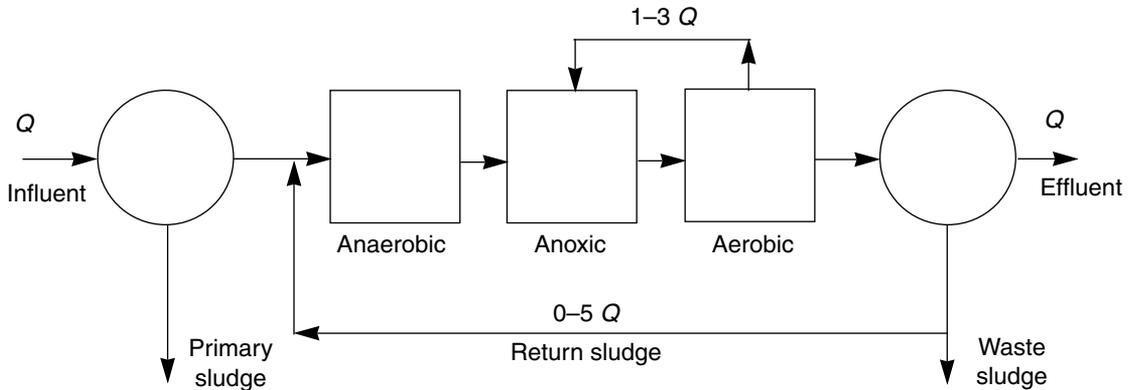


FIGURE 6.103 A²/O process flow diagram.

are looking at all nutrient discharges to sensitive waterways. Effluent phosphorus limits of 1.0 mg/L or less are becoming common (78, 80, 81, 117, 118).

When both nitrogen and phosphorus reductions are required, biological nutrient removal (BNR) utilizes combinations of the nitrogen and phosphorus control technologies discussed previously. Using anaerobic, anoxic, and aerobic zones, the principal BNR processes are:

- A²/O process—anaerobic/anoxic/oxic
- Five-stage Bardenpho process
- UCT process—University of Cape Town process
- VIP process—Virginia Initiative Project process

The A²/O process expands the capability of the AIG process used for BPR by adding an anoxic zone and internal recycles from the aerobic zone. Ammonia removal is achieved in the aerobic zone. The anoxic zone reduces the nitrate loading to the anaerobic zone for enhanced phosphorus removal. Denitrification occurs by internally recycling mixed liquor between the aerobic and anoxic zones. A process flow diagram is shown in Figure 6.103.

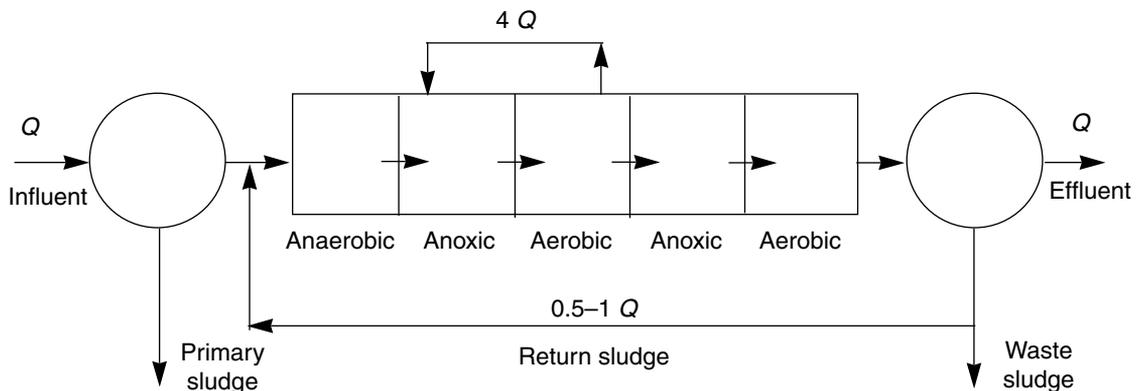


FIGURE 6.104 Five-stage Bardenpho process flow diagram.

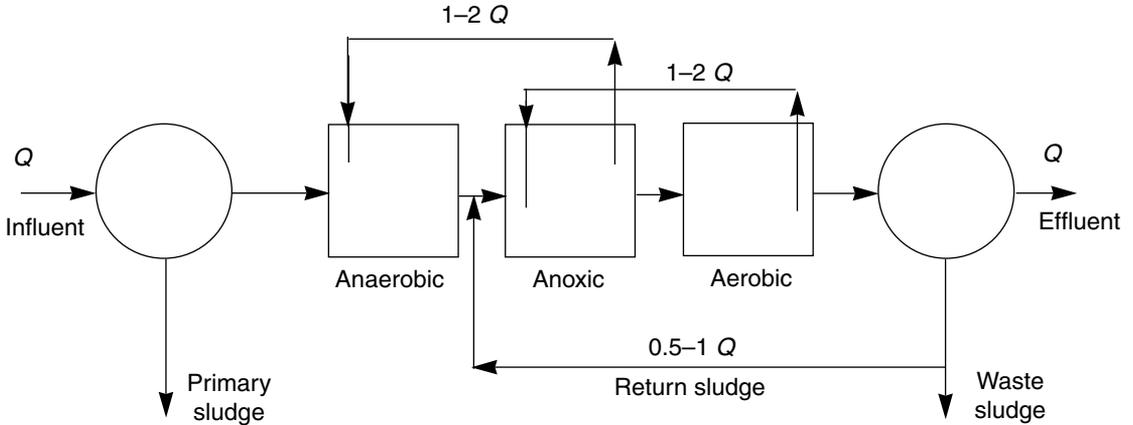


FIGURE 6.105 UCT process flow diagram.

As shown in Figure 6.104, the five-stage Bardenpho process includes anaerobic–anoxic–aerobic–anoxic–aerobic zones. The process design uses relatively low loading rates for increased nitrogen removal.

The UCT process returns activated sludge to ahead of the anoxic zone and thereby reduces the nitrate loadings to the anaerobic zone. As a result, phosphorus removal is increased in the anaerobic zone. This arrangement to maximize phosphorus removal does reduce nitrogen removal potential. Figure 6.105 illustrates the UCT process components.

The VIP process is similar in concept to the UCT process. The VIP process flow diagram is presented in Figure 6.106.

A comparison of BNR process design parameters is presented in Table 6.106. As with BPR processes, standby/backup chemical feed systems typically are provided as a safeguard against biological upsets or mechanical failures. Chemicals also may be used to trim BNR effluents to meet permit conditions. With phos-

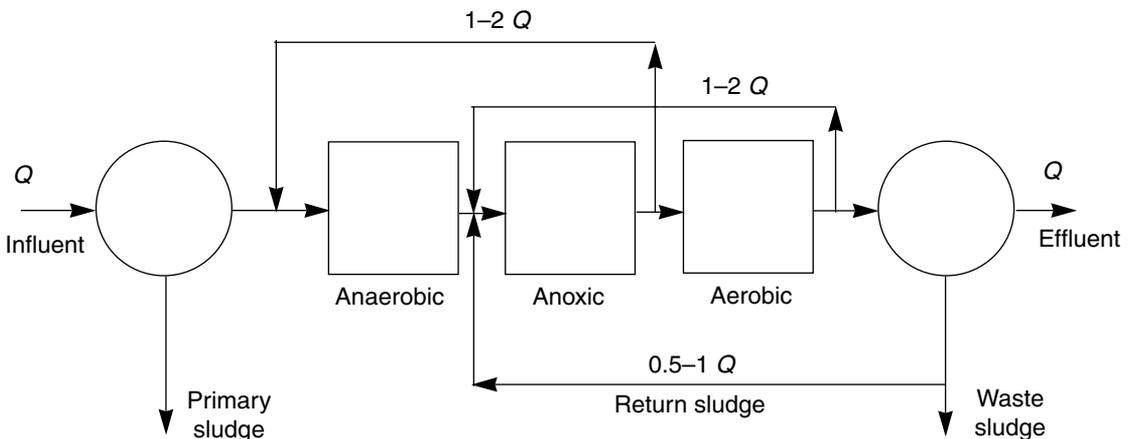


FIGURE 6.106 VIP process flow diagram.

TABLE 6.106 Comparison of BNR Process Design Parameters

Design parameter	BNR processes			
	A ² /O	Five-stage Bardenpho	UCT	VIP
F:M Ratio	0.15–0.25	0.1–0.2	0.1–0.2	0.1–0.2
SRT, days	4–27	10–40	10–30	5–10
MISS-mg/L	3000–5000	2000–4000	2000–4000	1500–3000
RAS,%	50	50–100	50–100	50–100
Internal recycle, %	100–300	400	200–400	200–400

phorus permit limits below 1 mg/L, chemical trim and/or tertiary filtration are provided to ensure effluent quality control.

Granular Media Filtration

Granular media filtration is a viable alternative for suspended solids reduction as a pretreatment before certain treatment processes, such as carbon columns; to upgrade a marginal secondary effluent to concentrations of 30 mg/L or less; to polish a good secondary effluent to levels of 10 mg/L; or, with chemicals, to reduce suspended solids to below 10 mg/L. The process for wastewater treatment is similar to that used historically for potable water treatment.

Design Considerations. The technology of granular media filtration is changing to satisfy the increasing applications in wastewater treatment. Table 6.107 illustrates most of the key elements considered for the design approach. Schematic diagrams for typical filter configurations are shown in Figure 6.107.

This section focuses on the most common system, which is a plug flow–downflow–gravity–continuous flow filter using stratified dual media. Figure 6.109 illustrates this system, and Table 6.108 presents typical design data.

The dual-media filter (Figure 6.108) uses a layer of anthracite over a layer of sand (see Table 6.109 for characteristics). This approach provides better suspended solids removal with longer filter runs at higher

TABLE 6.107 Granular Media Filter Characteristics

Item	Characteristic
Flow	
Scheme	Plug flow
Direction	Upflow, downflow, or biflow
Type	Gravity or pressure
Mode	Continuous or batch
Medium	
Type	Natural most common
Interface	Stratified or unstratified
Single	Fine to coarse sand
Dual	Anthracite and sand
Multiple	Anthracite, sand, and garnet

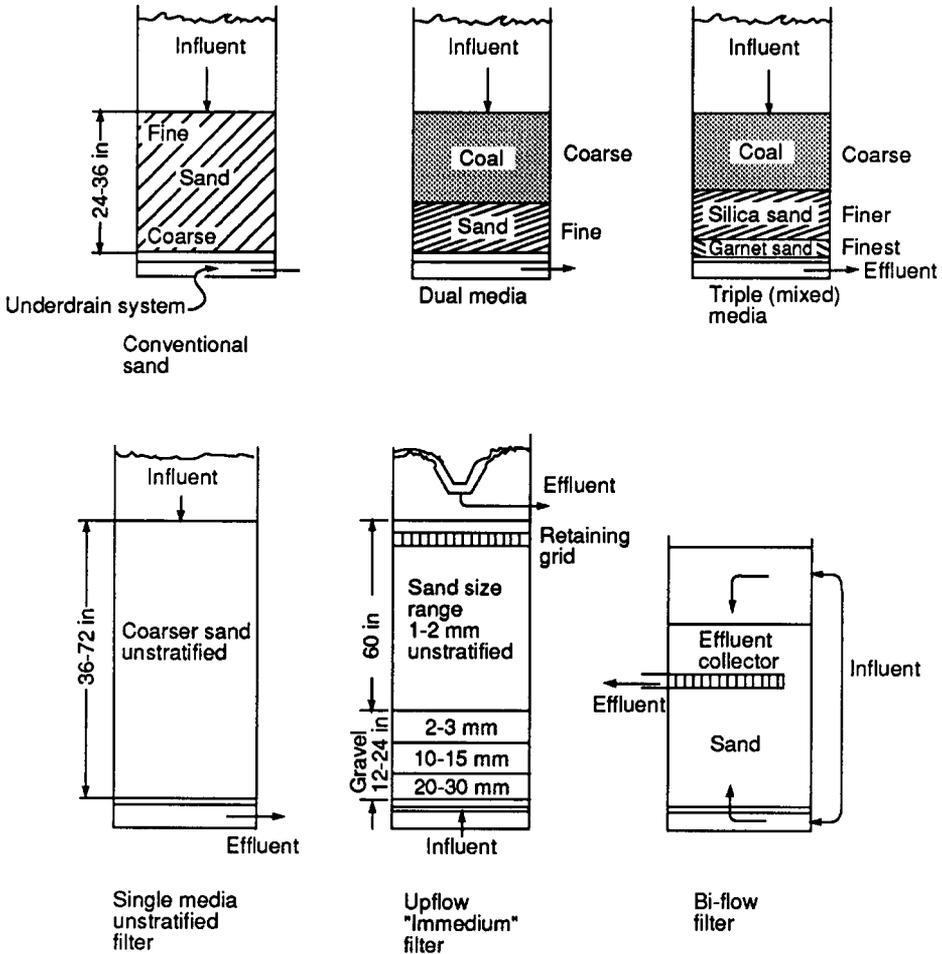


FIGURE 6.107 Granular media filter configurations (83). (Note: 1 in = 2.54 cm.)

flow rates than the more conventional single-medium filter. Minimum depths for anthracite begin at 38 to 46 cm (15 to 18 in) and are increased when heavy solids loads are expected. Typical sand layer depths are 8 to 12 in (20 to 30 cm). The backwash rate, at the end of the cleaning cycle, is adjusted to ensure hydraulic reclassification of the filter media.

Filter Backwashing. Periodically, a reverse flow or backwash is required to remove collected materials from the media. Maximum terminal head losses of 10 to 15 ft (3 to 4.5 m) are allowed before cleaning. Typical backwash flow rates are 15 to 25 gpm/ft² (10 to 17 L/m²/s) for 8 to 10 minutes.

The backwash system includes a source of water, which may be filtered wastewater or subsequent process effluents with few suspended solids. The backwash water is pumped from a storage tank sized for the number of filters, the duration of filter runs, and the backwash design conditions. Spent backwash water is routed to the plant's headworks or to an intermediate process providing settling. The system instrumenta-

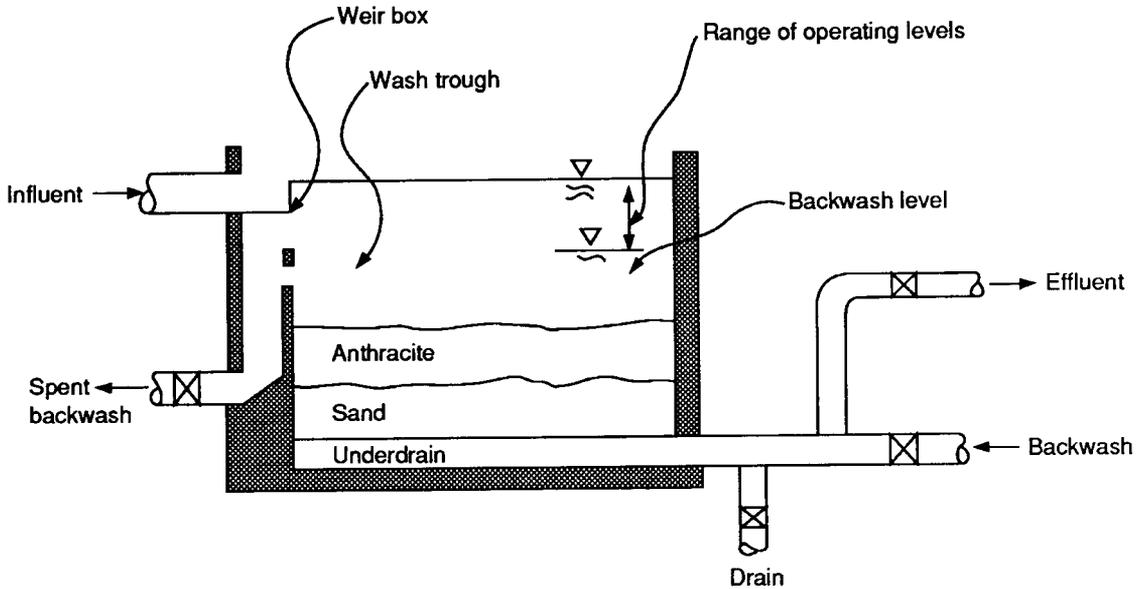


FIGURE 6.108 Dual media filter system.

TABLE 6.109 Dual-Media Filtration Design Data (49, 50, 83)

Parameter	Typical value
Number of filters	Minimum of two
Filtration rate	2–8 gal/min/ft ² (1.35–5.4 l/m ²)
Bed depth	24–48 in (60–120 cm)
Sand to anthracite ratio	1: 1–4:1
Filter run	8–48 h
Suspended solids removal	75%

TABLE 6.109 Anthracite and Sand Media Characteristics

Parameter	Typical value	
	Anthracite	Sand
Specific gravity	1.4–1.7	2.65
Effective size, mm	0.8–1.2	0.4–0.6
Uniformity coefficient	1.4–1.7	1.2–1.6

TABLE 6.110 Surface Wash System Operation (49, 50, 83)

Step 1	Take filter out of service.
Step 2	All filter water level to drain to top of wash trough.
Step 3	Apply surface wash water at 1–3 gpm/ft ² (0.7–2 L/m ² /s) and 50 to 100 lb/in ² (350–700 kN/m ²) for 1 to 3 mm.
Step 4	Begin water backwash cycle and continue both washings for 5 to 10 mm, as needed.
Step 5	Stop surface wash.
Step 6	Complete water backwash cycle for classification of filter media.
Step 7	Return filter to service.

tion and indicator controls are needed to ensure flexibility and control of the process by the operator. Even though most systems operate on a batch basis, continuous backwash systems are available.

The basic backwash system is often complemented by surface wash or air scour systems to improve cleaning, especially of organic particulates.

Surface Wash. A supplemental surface wash system can be used to loosen and remove deposits from upper levels of the medium. A sequence of surface washing steps and typical design data are presented in Table 6.110.

Air Scour. An air scour system has two objectives: (1) reduce washwater requirements and (2) clear deeper portions of the filter bed. The system must be designed and operated so that airflow can be controlled in order to prevent flushing out finer media and disrupting media placement and gradation. Sequencing and typical design data for use of air scour are shown in Table 6.111.

Carbon Adsorption

After long and successful use in the drinking water treatment industry, carbon adsorption continues to gain popularity in the wastewater treatment field. To date, most applications are for greater than secondary treatment and use granular rather than powdered activated carbon. The systems are designed based on the ability of specially prepared carbon to adsorb and remove a specific pollutant from the liquid portion of a wastewater flow. A secondary benefit to adsorption systems is the filtering effect of fixed beds of granular carbon.

Adsorption. The process by which a substance is taken up and becomes attached to the surface of a solid is known as adsorption. In this context, the substance to be removed is small concentrations of organic con-

TABLE 6.111 Air Scour System Operation (49, 50, 83)

Step 1	Take filter out of service.
Step 2	Lower water level in filter to a few inches above the media.
Step 3	Apply air at 3 to 5 cfm/ft ² (0.015–0.025 m ³ /m ² /s) alone for 3 to 10 min.
Step 4	Begin water backwash at 2 to 5 gpm/ft ² (1.4–3.4 L/m ² /s) until the water is around 10 in (25 cm) of the wash trough.
Step 5	Stop air supply.
Step 6	Continue water backwash through normal cycle for cleaning and classification of the filter media.
Step 7	Return filter to service.

stituents from wastewater, and the solid is activated carbon. Carbon is used because its porous structure provides a relatively large surface area per unit volume. Laboratory (and perhaps pilot plant) studies should be included in the system design phase.

The most important variables in selection and design of carbon treatment systems are contact time and carbon exhaustion, which are also known as usage rate and breakthrough characteristics. Contact time is especially important in that there must be adequate time for the pollutants to react with the carbon exterior and to enter and react with the surface of interior pores.

For example, the adsorption of the organic pollutants in a fixed-bed, granular carbon contactor is a non-steady-state process in which the constituent is removed in an increasing amount as the quantity of wastewater is passed through the bed. Rapid adsorption occurs as the wastewater passes through the upper layers of carbon, and the amount adsorbed is in equilibrium with the influent concentration. The adsorption zone, which is in equilibrium with the influent, moves downward in the bed as flow continues to be applied to the bed. This is illustrated in Figure 6.109.

As the adsorption zone approaches the bottom of the bed, the concentration increases. When the effluent concentration reaches the predetermined maximum, the pollutant is considered to have broken through. The breakthrough is defined as the volume of wastewater that has passed through the bed before the maximum allowable concentration appears in the effluent. The time to breakthrough increases with an increase in depth of carbon; a decrease in effective carbon particle size; a decrease in the flow rate; and a decrease in the influent concentration. The breakthrough curve is also useful to determine the frequency and rate of regeneration needed.

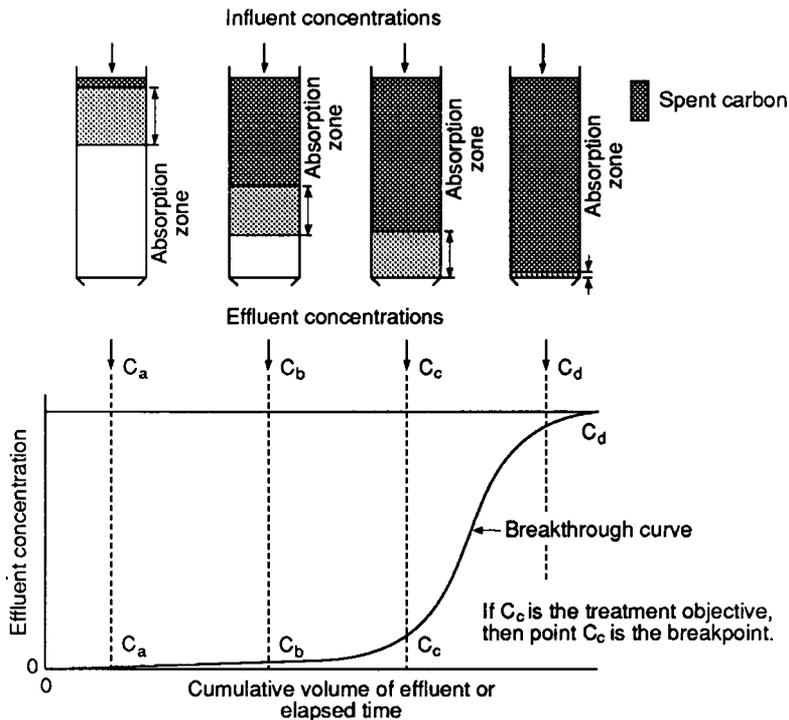


FIGURE 6.109 Fixed-bed adsorption and breakthrough (84).

Regeneration. When the adsorption capacity of the carbon is exhausted, the supply must be replenished. In larger plants, this is accomplished by a regeneration facility on site. For smaller installation, the spent carbon is either wasted or regenerated at an off-site multiuser facility.

Thermal regeneration systems are used at wastewater treatment plants and usually include multiple-hearth furnaces. Different types of furnaces or a rotary kiln may be considered as alternatives. Other system components include separate holding tanks for spent and/or regenerated carbon, dewatering screws, quench tanks, and air pollution control equipment (see Figure 6.110).

Typical design data for thermal carbon regeneration are presented in Table 6.112. The thermal process includes the following steps:

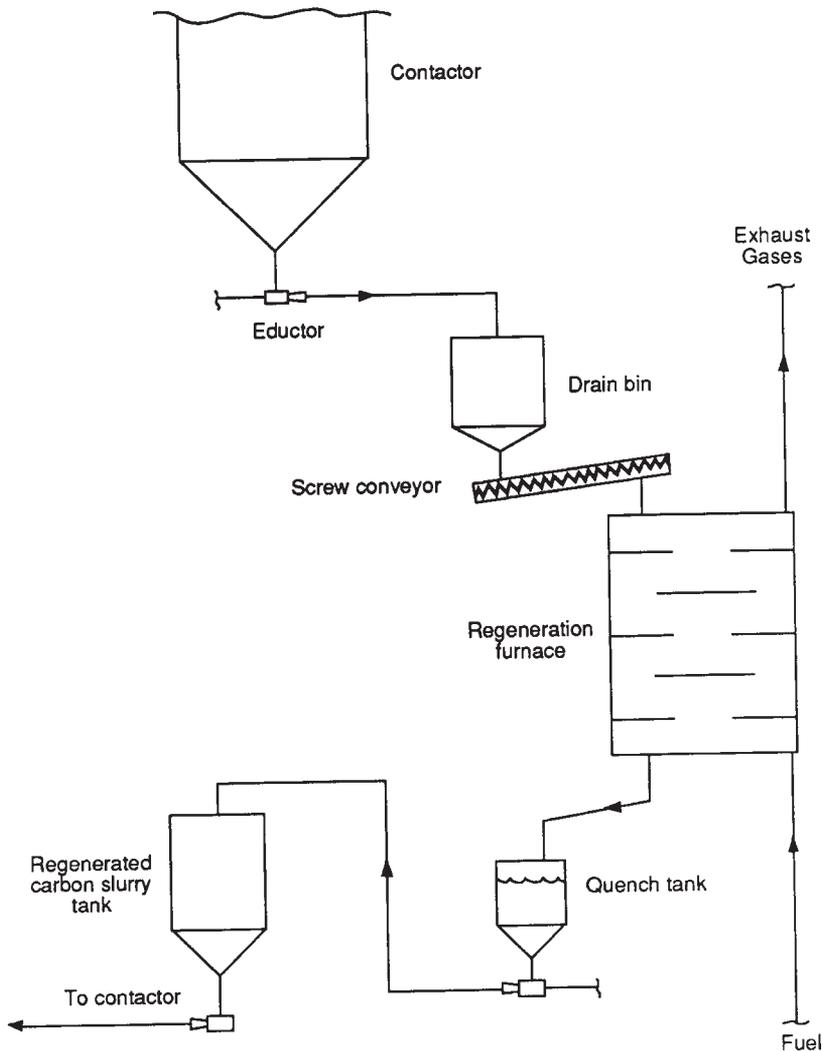


FIGURE 6.110 Thermal carbon regeneration system (85).

TABLE 6.112 Thermal Carbon Regeneration Design Data (84, 85)

Parameter	Typical value
Multiple-hearth furnace loading	80–i 10 lb/day/ft ² (55–75 kg/m ³ /h)
Residence time	30 min
Natural gas fuel	3.0–5.5 scf/lb C (0.19–0.34 m ³ /kg C)
Steam	1–3 lb/lb C (1–3 kg/kg C)
Energy requirements	
Carbon	3200 Btu/lb C (7435 kJ/kg C)
Steam	1250 Btu/lb (2900 kJ/kg)
Heat loss	15%
Carbon loss	5–10%

1. Drying by evaporation at temperatures of 212 to 300°F (100 to 150°C).
2. Volatilization of light organics at temperatures of 300 to 600°F (150 to 315°C).
3. Pyrolysis of heavy organics of temperatures of 600 to 1500°F (315 to 815°C).
4. Reactivation by removal of char at temperatures of 1700 to 1900°F (925 to 1040°C) and gaseous products from the pores.

The regenerated carbon normally has different characteristics than virgin carbon, in part due to enlarged pore sizes from the pyrolysis step.

Granular Activated Carbon. In wastewater treatment, carbon adsorption systems most often use granular activated carbon (GAC). The carbon is available from a number of suppliers, and the common particle sizes are 8 × 30 and 12 × 40 mesh.

The principal component of a GAC system is the contactor(s), where the bed of activated carbon is housed. Two other key components are systems for carbon regeneration and for carbon storage and transfer. These components interact as shown in Figure 6.111, along with the optional components of pumping facilities and bed backwash. A process flow diagram for a downflow–fixed-bed system with regeneration is shown in Figure 6.112.

The design of the adsorption contactor is based on flow rate, hydraulic loading, and contact time, which determine the volume, cross-sectional area, and height needs. The type of vessel is typically cylindrical steel, open top for a gravity or closed for a pressure system, or rectangular concrete for a gravity system. Combining the geometry requirements and type of vessel, the number of parallel and series contactors is determined.

Modes of Operation. There are several flow options for GAC systems. The fundamental types are fixed bed and moving bed. The direction of flow may be down or up through the column. The downflow units function by gravity or under pressure. Pressure systems are used for upflow units. Except in special cases, multiple beds or contactors are used.

Typical GAC contactor configurations include series or parallel operation of

- Downflow—fixed beds
- Upflow—expanded beds

These systems are illustrated in Figure 6.113.

The series use of fixed beds provides very efficient use of carbon in that a bed may be taken to complete exhaustion before regeneration while effluent quality is protected by subsequent beds. This condition may be approached with parallel beds if blending of effluents is acceptable practice. It is noted that startup of parallel beds should be staggered to permit bed-by-bed regeneration without reducing effluent quality. The par-

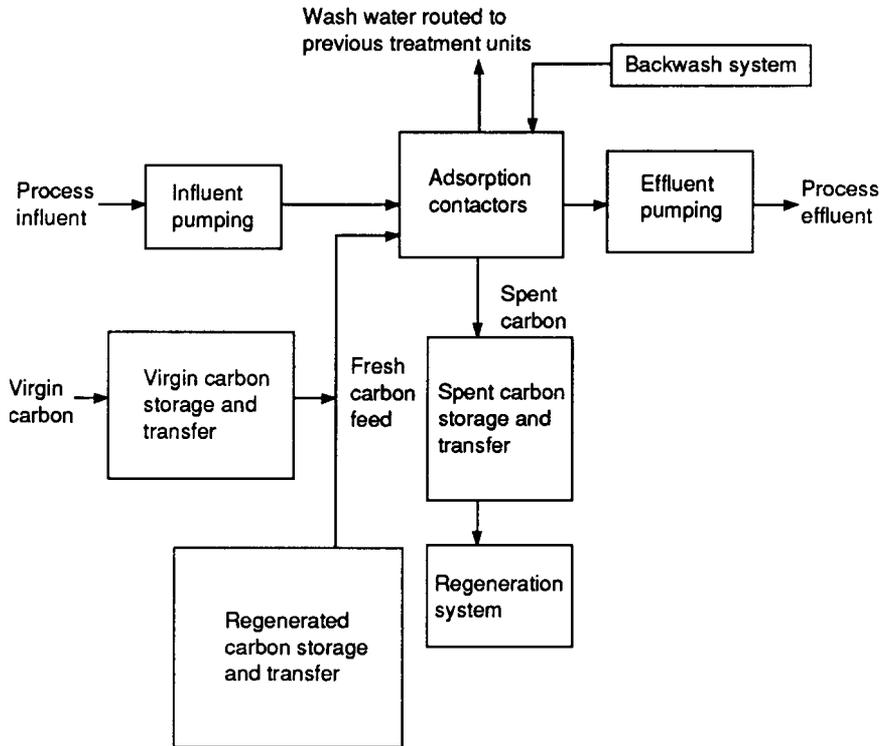


FIGURE 6.111 Granular activated carbon process schematic.

allel-bed approach is preferred at plants with a large total flow rate. The downflow units will serve as a suspended solids filter and, if the loading is very significant, periodic backwashing will be required.

The upflow expanded beds can tolerate higher suspended solids loadings than the downflow beds, but their effluent suspended solids concentrations may be excessive, requiring filtration before subsequent treatment or discharge. The upflow moving bed contactor makes very efficient use of the carbon since fully exhausted carbon can be removed from the bottom of the bed while fresh carbon is added to the top.

Design Consideration. The design of the adsorption system is based on the influent and effluent wastewater characteristics. The former affects the choice of flow configuration and the size of equipment. Trade-offs may be made between adsorption and regeneration systems with respect to operations and economics. The effluent characteristics are the result of treatment objectives and performance requirements. Also, the design should consider the requirements for carbon handling and storage.

After the needed contact time is established, the cross-sectional area of carbon is selected to provide a permissible hydraulic loading. Superficial bed velocity should be considered, but the primary consideration in selecting the hydraulic loading is head loss. The head loss depends on flow rate, suspended solids loading, carbon size and type, and gravity or pressure flow conditions. Typical carbon bed hydraulic loading rates for reasonable head losses and common systems are shown in Figure 6.114.

For downflow beds, backwashing is another design consideration. The quantity and type of accumulated solids and head loss (hydraulic loading) interact to determine the frequency and quantity of backwash. Also, air scouring and surface washing may be appropriate for a specific application.

Backwash frequency may vary from every few hours to more than a day; commonly, several backwashes

WASTEWATER DISPOSAL

Notes

1. Tank A is operating as primary adsorber.
2. Tank B is operating as secondary adsorber.
3. Tank C is operating on backwash.
4. Tank D is idle.
5. Tank E is operating on a cycle to remove carbon.
6. Tank F is operating on a cycle to refill with activated carbon.

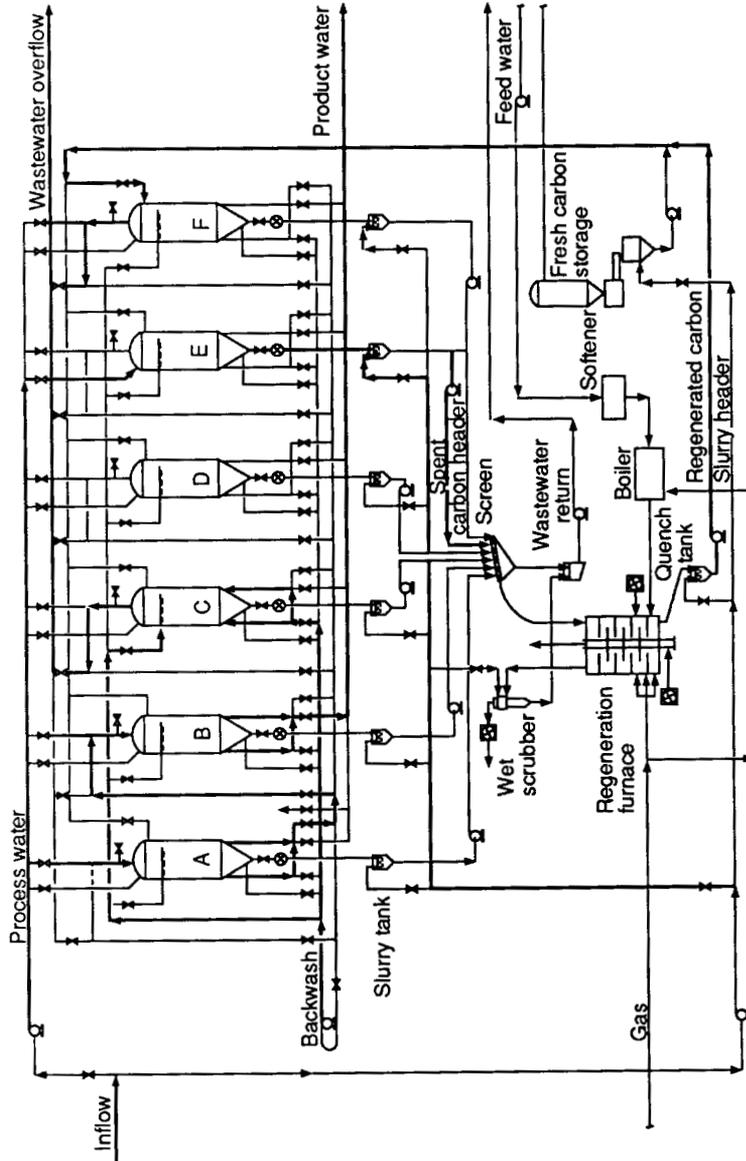


FIGURE 6.112 Granular activated carbon process flow diagram.

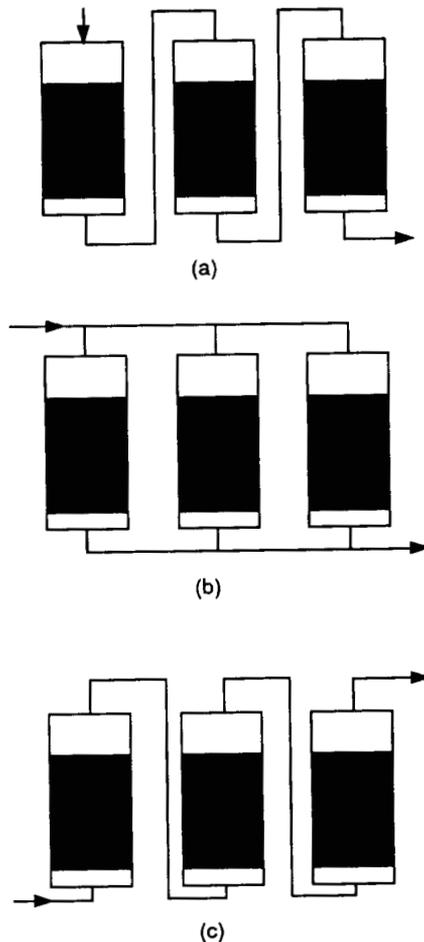


FIGURE 6.113 Modes of operation for granular activated carbon systems. (a) Downflow in series fixed bed; (b) downflow in parallel fixed bed; (c) upflow in series expanded bed (85).

are needed each day. Typical backwash rates are 15 to 20 gpm/ft² (600 to 800 L/m²/min) for a duration of 10 to 20 min.

Powdered Activated Carbon. Powdered carbon is being used in wastewater treatment to remove soluble organics and to aid clarification. The carbon can be used dry or in a slurry form and is fed into the existing conventional treatment works. Coagulation or effluent filters may need to be added to capture residual carbon particles for discharge suspended solids control. Also, there is a need for increased sludge-handling capability. Finally, regeneration may or may not be cost effective. The following are examples of powdered activated carbon applications:

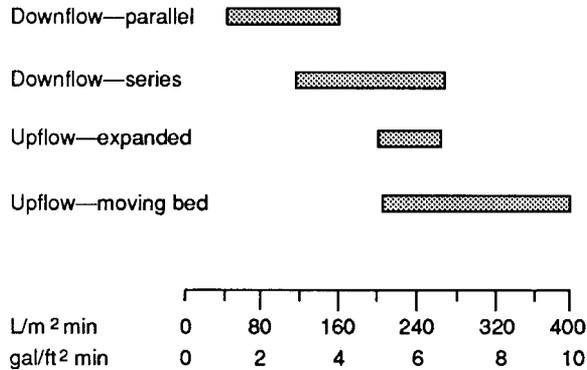


FIGURE 6.114 Typical hydraulic loadings for granular activated carbon systems.

1. Addition to primary clarifiers to reduce load of BOD and solids to downstream processes
2. Addition to activated sludge system to control organic loading
3. Addition to secondary clarifiers to achieve advanced levels of treatment

Powdered activated carbon is receiving more and more attention as a viable wastewater treatment alternative after many years of successful performance in the water supply industry.

Ion Exchange

In some industrial wastewater discharges, there are certain waste streams that have expensive chemicals in them that can be recovered for reuse. An example is in the exotic heavy metals plating industry, where there is a possibility of removing gold, silver, cadmium, and other metals for reuse. It is also necessary to remove small traces of certain chemicals from the waste streams, such as copper, zinc, phenols, cyanides, and chromates. The use of ion exchange for this type of treatment should be considered as a possible economic alternative.

Ion exchange differs from other systems of waste treatment as it is a process for substituting other ions for certain ions in the waste stream. Innocuous cations, such as hydrogen (H^+), can be substituted for toxic heavy metals (Cu^{2+} , Cd^{2+} , etc.), and innocuous anions, such as hydroxide (OH^-), can be substituted for toxic anion ions, such as phenols and cyanides.

Ion Exchange Resins. Ion exchange resins are relatively insoluble granular materials that have acid or basic radicals exposed on parts of their surface that can be exchanged for ions of the same positive or negative sign in the solution in contact with the granular materials. This exchange is done without any physical change in the granular material other than a possible increase in volume. The same number of ions will be removed from the ion exchange resin as are removed from the solution and added to the resin. Therefore, the capacity of the resin to exchange charged radicals is based on the number of ion radicals available on the exposed granular surface of the resin. Because the number of exposed radicals is proportional to the surface area of the resin granular, the resins are furnished with a wide range of individual particle sizes for each type of resin. Thus, to determine the capacity for ion exchange, the type of resin and the size or graduation of the resin being furnished must be known.

There are basically four types of ion exchange resins. As shown in Figure 6.115, there are either strong

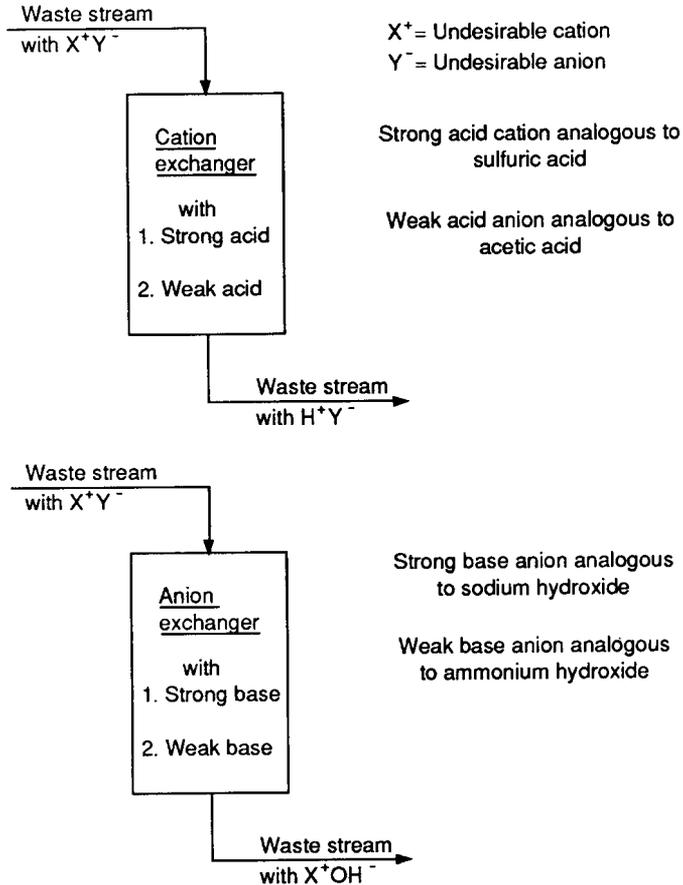


FIGURE 6.115 Ion exchange system schematics.

acid or weak acid cation resins, and, depending on the waste to be treated, either one or both may be required. Strong acids are effective in removing heavy metals, such as calcium, magnesium, iron, cadmium, silver, gold, and mercury. Weak acids are useful in separation of simple and complex organic bases. Strong acid cation exchanges are more useful in wastewater treatment. Weak acid cation exchanges are more useful in special chemical process work, such as the isolations of alkaloids and antibiotics.

As also shown in Figure 6.115, there are strong base anion resins or weak base resins, and either one or both may be required for treatment of certain wastewaters. The strong base anion resins are in the hydroxide form, which splits neutral salts by converting the salt into its corresponding base and converting the ion exchange resin into the corresponding salt. The strong base anion resin removes all mineral acids, including the weak acids such as boric and silicic. A weak base anion resin will remove only strong mineral acids such as hydrochloric, sulfuric, and nitric, and permit weak mineral acids such as carbonic and silicic acid to pass through.

Special selective resins have been developed by certain manufacturers that can be used for removal of specific ions. This applies to both cations and anions of inorganic nature and some organics.

Depending on the application, a cation exchanger with a strong or weak acid resin may be all that is re-

quired, or a single strong or weak base resin ion exchanger may be suitable, as shown in Figure 6.115. If both cations and anions are to be removed, a cation exchanger followed by an anion exchanger will be needed. A simplified flow sheet is shown in Figure 6.116. Other combinations are listed below

- Strong acid/weak base (two-train unit)
- Weak acid/strong base (two-train unit)
- Weak acid/weak base (two-train unit)
- Strong acid/weak acid/weak or strong base or both bases (three- or four-train unit)
- Strong acid/weak and strong base (three-train unit)

The cation and anion exchange beds, when combined into a single demineralizing system, are called multibed units or trains. Frequently, the two types of resins, cation and anion, are mixed in the same treatment tank, and this results in a demineralizer system known as a mixed-bed unit. The mixed-bed units are generally installed following a multibed train when ultrapure water is required.

Regeneration. An ion exchange is the substitution of one ion for another; the process is reversed during regeneration. Regeneration consists of filling the cation unit with a concentrated acid solution, letting the unit stand for a considerable period of time, then discharging all of the waste cations which have been collected. The same process is duplicated in the anion unit except that caustic or another strong alkali is used instead of an acid. If there is recovery of either the cation or anions at this point, then the regenerative waste from that particular unit can be discharged back to the process tank or into a separate unit for holding for any special treatment prior to reuse. If the regenerative waste is not to be used, then special provisions for handling or treatment of the concentrated waste will be required.

Operating Modes. The common modes of operation are batch, column, and moving bed. The selection of method is based primarily on the ion or ions to be removed and the batch or continuous quantity of flow.

Batch Operation. The batch operation consists of putting the exchange resin and wastewater in a tank, mixing, and allowing equilibrium to be established. Then, the treated wastewater is filtered off, and the resin is prepared for the next cycle. The efficiency of the process is a function of the selectivity of the ion exchange resin at equilibrium; generally, only a portion of the resin's capacity is utilized. Also, it is often impractical to regenerate the resin for batch reuse.

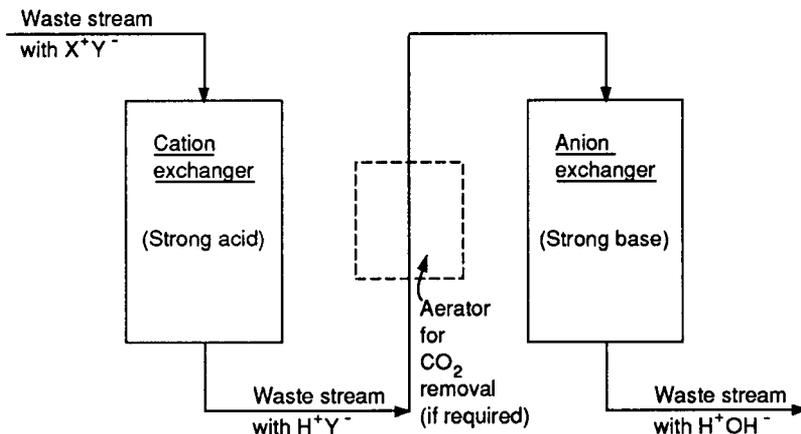


FIGURE 6.116 Multibed ion exchange system schematic.

Column Operation. The column operation can be considered as many batch operations in series. The wastewater flows through the column, which is a vertical cylinder filled with the ion exchange resin. The flow may be down or up through the resin. Most fixed-bed units use downflow operation. Also, the ion exchange column may be operated with countercurrent flows of wastewater and regenerant.

Moving-Bed Operation. The moving-bed or continuous countercurrent ion exchange system is another modification of the ion exchange process. This system is designed to continuously remove and regenerate part of the ion exchange resin and then return the resin to the treatment system. This has the advantage of eliminating the long periods of downtime for regeneration. This system has another advantage, which is maintaining a consistent effluent water quality. A normal multibed demineralizer will start to discharge or leak undesirable ions into the effluent as the exchange positions on the granular medium becomes saturated. Therefore, the quality of the effluent will gradually deteriorate. A conductivity meter is used to detect the deterioration.

Design Considerations. The actual system to be used will depend entirely on the type of cations and/or anions to be removed, the degree of purity desired, the concentration of the constituents being removed, and the presence of other cations or anions in the waste stream that will also be removed and/or will hinder the removal of the desired constituents. Therefore, it is essential that a complete laboratory analysis of the waste stream be made prior to any detailed design of the system.

Laboratory testing can be used to develop a variety of operating curves, such as the one presented in Figure 6.117, which illustrate the active exchange and regeneration cycles for a column operation. The slope of the curve is steeper for low flow rates than for high flow rates.

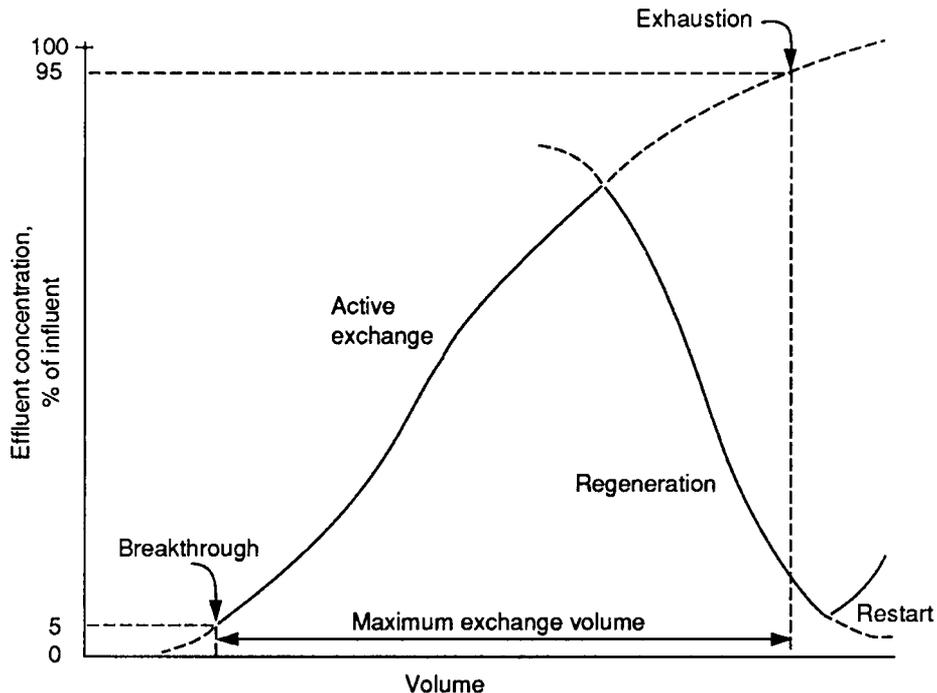


FIGURE 6.117 Ion exchange operating curve.

The exchange zone descends through the resin bed a distance equal to its height during the time it takes for a volume of effluent to pass through the column. It is noted that breakthrough occurs at the limit of the exhaustion cycle and the beginning of the regeneration cycle and that exhaustion is when the resin exchange capacity is deleted.

The pH and temperature of the waste stream under operating conditions are essential for the design. Some common ion exchange resins are dissolved at elevated temperature. Other resins are very sensitive to certain pH ranges. None of the ion exchange equipment is designed to handle suspended solids loading, because this material will rapidly clog the unit and reduce the flow through it. Some oily organics will coat the resin and prevent the ion exchange from taking place. Other organics can easily be selectively removed in special ion exchange units using special resins. The past experience of the various ion exchange manufacturers is very helpful in sizing equipment units and selecting the most economical resin for the particular waste problem.

In the design of a wastewater treatment process, there are several factors that should be considered when evaluating the possible use of ion exchanges (see Table 6.113). Based on these items, an evaluation of the total cost on an annual basis can be developed for comparison with other methods of treating the waste stream.

In selection of the necessary equipment, there are several factors, such as those in Table 6.114, which should be considered. A typical ion exchange train with controllers, valves, pumps, and chemical feed system is shown in Figure 6.118; dual trains are required for continuous operation. Normally, these systems are purchased as a complete package from any one of a large number of manufacturers.

A practical example use of ion exchange is for the removal of ammonium ions from wastewater where cold weather limits use of other processes, reduction from a feed with low concentrations is needed, potential for the reduction of emissions to the atmosphere exists, and/or limited increase in total dissolved solids is permitted. Typical design criteria for a column system with regenerant recovery appears in Table 6.115.

Reverse Osmosis

While most current applications are for treating brackish water and seawater for drinking supplies, reverse osmosis systems are commercially available for wastewater treatment. The process permits the removal of dissolved solids from the principal wastewater flow; the dissolved solids are concentrated into a much smaller volume for separate treatment or disposal.

Even though reverse osmosis technically has many applications, such as treatment of raw sewage or sewage reuse, its most practical applications are for industrial water pollution control as a concentrating process. Applications include liquid volume reduction, by-product recovery, process water recycle or reuse, and pollutant concentration.

TABLE 6.113 Evaluations in Ion Exchange Process Selection

-
1. Will exchange permit the recovery and reuse of a process chemical?
 2. Is it possible to remove only the undesirable waste constituents that make it impossible to treat in other units, such as a municipal sewage treatment plant?
 3. Are there other processes that can do this same removal more economically?
 4. Will there be a way to handle or treat the concentrated regenerative waste which is periodically removed from the ion exchange units?
 5. Was the cost of handling and purchasing the regenerative chemicals included in the process cost evaluation?
 6. Was the cost of handling and treating the regenerative wastes included in the process cost evaluation?
 7. Based on the selected resin, what is the annual cost for additions to and periodic replacement of ion exchange resin? (The actual expected life varies greatly with the resin selected and the waste characteristics.)
 8. What is the estimated cost for the necessary equipment to provide this ion exchange service?
-

TABLE 6.114 Evaluations in Ion Exchange Equipment Selection

1. Based on the waste constituents to be removed, what is the best sequence for the operating units?
2. Based on the waste constituents to be removed, what are the best types of resins to be used?
3. Based on possible savings in labor cost, should the unit be regenerated manually or automatically?
4. Should a storage tank be provided with sufficient storage ahead of the demineralizer to permit the several hours of downtime required for regeneration or should two separate ion exchange trains be furnished? Can the ion exchange train be regenerated during nonproduction hours at the industrial plant?
5. Have the size and the type of resin in the cation and anion units of the exchange train been balanced to permit economic regeneration of the total train at the same time? (A strong acid cation exchange resin may have twice the absorption capacity of a strong base anion exchange resin.)
6. In the space requirements for the ion exchange units, has consideration been given to the accessibility of all control valves, piping, chemical feed pumps, chemical feed storage and forklift truck access for regeneration chemicals?
7. Have provisions been made for chemical spills in the area of the ion exchange units?
8. Where is the control panel to be located in relation to the operating units?
9. Is the regenerative waste sump of adequate size?
10. Have the proper materials been selected for the ion exchange resin tanks, chemical storage tanks, piping, valves, pumps, controllers, and sump linings?
11. Is it possible to remove any single piece of the equipment out of the space without dismantling everything else first?
12. Is all equipment properly labeled with name tags, valve numbers, pipes coded, and pumps numbered in accordance with the manufacturer's operating and maintenance instructions?

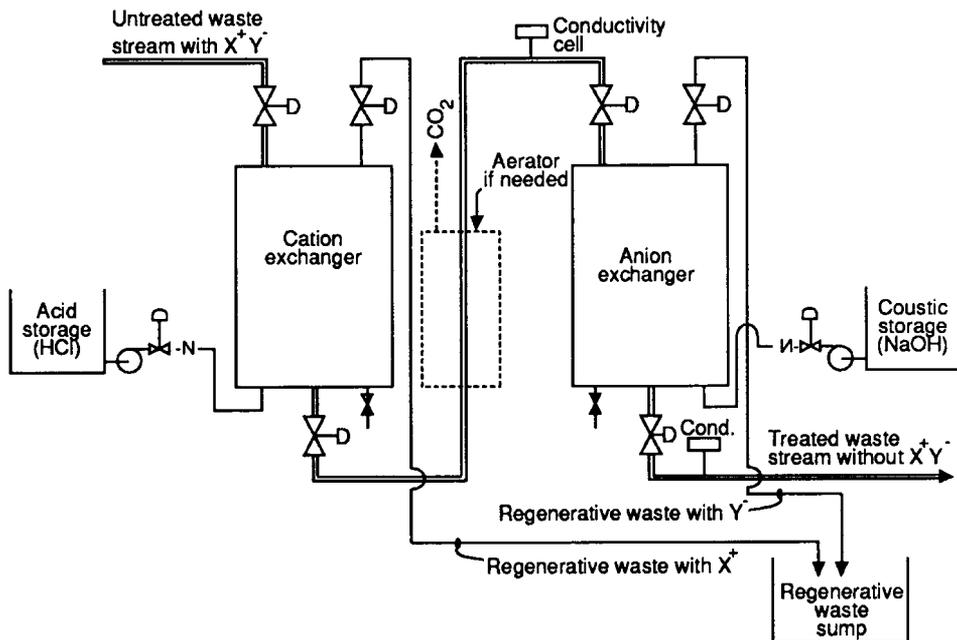
**FIGURE 6.118** Single cation–anion exchanger train.

TABLE 6.115 Ion Exchange Design Criteria for Ammonia Removal (50)

Ion Exchange Operation	
Clinoptilolite medium, size = 20 × 50 mesh	
Bed height = 4–6 ft (1.2–1.8 m)	
Wastewater suspended solids = 35 mg/L maximum	
Wastewater loading rate = 7.5 to 20 bed volumes per hour	
Pressure drop = 8.4 in (21 cm) of water per foot (meter)	
Cycle time = 100–150 bed volumes for one 6-ft (1.8-m) bed = 200–250 bed volumes for two 6-ft (1.8-m) beds in series	
Regeneration	
Solution = 2% NaCl	
Solution flow rate = 4–10 bed volumes per hour = 4–8 gpm/ft ² (163–326 L/m ² min)	
Total solution volume = 10 bed volumes = 2.5 to 5% of treated wastewater	
Cycle time = 1–3 h	
Backwash = 8 gpm/ft ² (326 L/m ² /min)	

Design considerations include

1. Feedwater characteristics, especially total dissolved solids
2. Quality requirements of treated water
3. Presence of particulate and colloidal matter that may clog membrane
4. Membrane type and configuration
5. Operating conditions (typical):
 - Pressure: 400–500 lb/in² (2755–3450 kN/m²)
 - Temperature: 50–75°F (10–24°C)
 - pH: 6 to 8 units
6. Single or multiple storage systems
7. Cost effectiveness of system with respect to other alternatives

Currently, it is the last item, cost, that significantly limits the use of reverse osmosis for wastewater treatment.

Deep-Well Disposal

Disposal by deep-well injection has been utilized for a variety of wastewaters, especially those that are not economically treated by conventional methods. The increased emphasis on groundwater protection and hazardous waste control has significantly limited the use of this disposal method.

The design objectives are to locate a previous stratum below an impervious stratum that protects groundwater supplies. The wastewater should not cause deterioration of the stratum and there should be no geological faults. The well depth may range from several hundred feet to a few miles. The development of the well and selection of operating equipment is of utmost importance to the success of this disposal method.

SLUDGE HANDLING

The procedures for treatment and disposal of sludge generated at the wastewater treatment plant are a function of the raw wastewater characteristics, individual treatment processes, chemicals usage, local regula-

tions, and numerous site-specific conditions. Furthermore, the capital costs and often demanding operational requirements of sludge-handling systems may equal or exceed those of the preceding liquid treatment facilities. Thus, the design engineer must thoroughly evaluate alternatives to determine the cost and operational effectiveness of a sludge-handling system.

Sludge-handling processes are categorized as

- Stabilization
- Conditioning
- Concentration
- Dewatering
- Drying
- Residual disposal

Sections follow that discuss design of each process. The interrelation of processes and sludge-handling methods are illustrated in Figure 6.119, along with key words defining functions of each process.

Stabilization

Sludge stabilization is provided to eliminate nuisance conditions and to reduce the threat to human health. Also, volume and weight reduction and by-product (gas) production are stabilization functions.

To eliminate nuisance conditions, the putrescibility of the sludge must be controlled. This may be achieved by either eliminating the readily degradable organic matter through further biological treatment or by physical or chemical action to discourage microbial growth. To minimize the threat to human health, microorganism levels must be reduced. Such reductions may be made by biological, physical, or chemical means or by natural decay.

Processes that may be used to stabilize sludges include the following:

- Aerobic digestion
- Anaerobic digestion
- Chemical stabilization
- Composting
- Heat treatment
- Irradiation
- Lagooning

The selection of a stabilization method or combination of methods depends upon the method of sludge treatment and dewatering, and upon the final disposal method chosen. Stabilization methods, such as aerobic and anaerobic digestion, also reduce the sludge mass and may significantly alter dewatering properties, so these changes must also be considered in selection and design of stabilization processes.

Aerobic Digestion. Stabilization of sludge by aerobic digestion is well suited for use at smaller treatment facilities. For larger facilities, energy costs usually favor other stabilization methods. Advantages and disadvantages of the aerobic digestion process are listed in Table 6.116. Typical design data are presented in Table 6.117.

Aerobic digestion may be designed for batch or continuous-flow operation. When operated as a batch process, sludge is added to the reaction tank while the contents are continuously aerated. Once the tank is filled, aeration continues for another two or three weeks. Aeration is then discontinued and solid-liquid separation occurs. Solids at 2 to 4% are removed, and clarified liquid, supernatant, is decanted. The continuous-

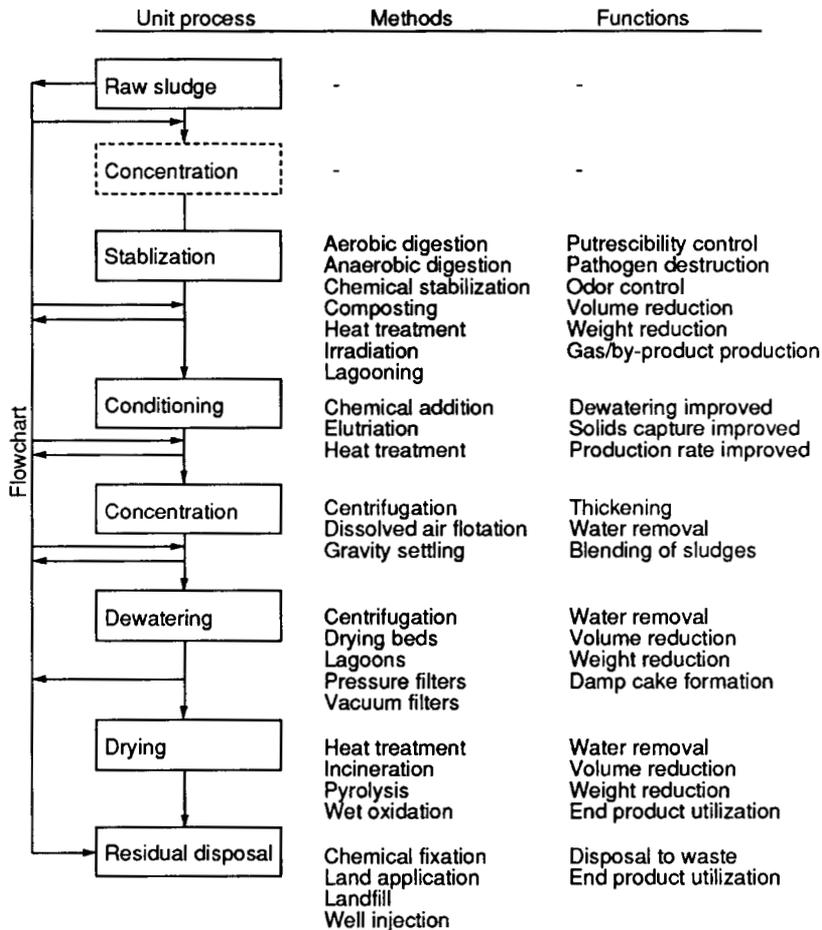


FIGURE 6.119 Sludge-handling processes, methods, and functions.

flow operation is similar to conventional aeration systems with an aeration tank followed by settling, as shown in Figure 6.120.

Important process design parameters (see Table 6.117) are very temperature sensitive. For example, destruction of volatile solids nearly ceases below 50°F (10°C). Similarly, destruction of a given microorganism requires several times the retention period at 50°F (10°C) as at 68°F (20°C). Thus, operation at temperatures below 59°F (15°C) is undesirable and may require excessive retention times to ensure stabilization. If heating is warranted, aerobic digestion may not be cost effective.

Anaerobic Digestion. Biological degradation of organic matter in the absence of free oxygen is termed anaerobic digestion. Anaerobic digestion of sewage sludge results in conversion of readily degradable organic matter into carbon dioxide (20 to 30% by volume), methane (60 to 70% by volume), hydrogen sulfide, other gases, and water, leaving a biologically stable residue. In addition, a considerable reduction in indicator bacteria occurs. The advantages and disadvantages of anaerobic digestion are generally the reverse of

TABLE 6.116 Advantages and Disadvantages of Aerobic Digestion (16, 50, 86)

Advantages	
•	Simplicity of operation and maintenance
•	Lower capital costs
•	Lower BOD and total phosphorous in supernatant
•	Few effects from loading, pH, and toxic interference
•	Insignificant odors
•	Nonexplosive
•	Reduces grease and hexane solubles
•	Better sludge fertilizer value
•	Shorter retention periods
•	Excellent small-plant alternative
Disadvantages	
•	Higher operating costs
•	Highly sensitive to ambient temperature
•	No useful by-products
•	Variable dewaterability
•	Lower volatile solids reduction
•	Questionable economics for larger plants
•	Reduces pH and alkalinity
•	May experience foaming
•	Potential for pathogen spread through aerosol drift

TABLE 6.117 Aerobic Digester Design Data (4, 16, 46, 71, 86)

Parameter	Typical value
Number of tanks	Multiple desired
Solids retention time @20°C	
Primary sludge	15–20 days
Primary and activated sludge or trickling filter sludge	15–20 days
Waste activated sludge only	10–16 days
Waste activated sludge without primary settling	12–18 days
Volume allowance	3–4 ft ³ /capita/day (0.8–0.11 m ³ /capita/day)
Volatile suspended solids loading	0.02–0.14 lb/ft ³ /day (0.32–2.24 kg/m ³ /day)
Minimum dissolved oxygen	1–2 mg/L
Oxygen requirements	
Destroy cell tissue	2:1
Reduce primary sludge BOD	1.6: 1–1 .9:1
Diffused aeration	
Waste activated sludge only	20–30 ft ³ /min/1000 ft ³ (20–30 m ³ /min/1000 in ³)
Mixed or other sludges	Over 60 ft ³ /min/1000 ft ³ (over 60 m ³ /min/1000/m ³)
Mechanical aeration	0.75–1.5 hp/1000 ft ³ (20–40 kW/1000 m ³)

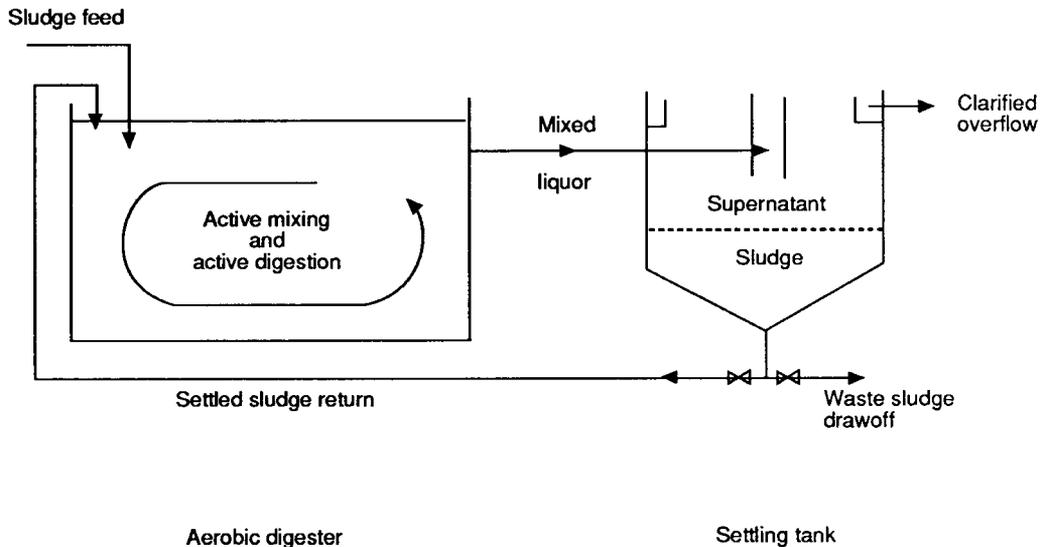


FIGURE 6.120 Continuous-flow aerobic digestion system.

those for aerobic digestion (Table 6.116). General design data for anaerobic sludge digestion processes appear in Table 6.118.

The design of anaerobic digestion systems is influenced by sludge chemical factors such as alkalinity, pH, volatile acids, nutrients, and toxins, and by digester physical factors such as detention time, temperature, mixing, solids loading, and solids concentration. Temperature, mixing, and detention time are the primary operational factors affecting design.

Conventional digester tanks are cylindrical with a sloped bottom. Minimum bottom slopes are 1:4 and 1:12 for gravity and mechanical removal systems, respectively. Egg-shaped digesters are an alternate configuration. Digester design features include digester cover, facilities for mixing, sludge heating, gas collection and regulation, control valves, and odor control.

TABLE 6.118 Anaerobic Digester Design Data (16, 46, 50, 86)

Parameter	Typical value
Number of tanks	Multiple desired
Solids retention time	
Mesophilic	20–60 days
Thermophilic	10–20 days
Temperature	
Mesophilic	50–95°F (10–35°C)
Thermophilic	100–140°F (38–60°C)
Volume allowance	0.04–0.2 ft ³ /capita/day (0.001–0.01 m ³ /capita/day)
Volatile suspended solids loading	0.04–0.4 lb/ft ³ /day (0.64–6.4 kg/m ³ /day)
pH range	6.6–7.6
Digested sludge solids concentration	4–6%

In order to utilize digester gas, a proper gas collection and storage system must be designed. Components of such a system include covers for the tanks and supplemental storage tanks. The two principal types of covers for cylindrical digesters are fixed and floating.

Fixed covers provide maximum use of digester volume, while simplifying the design and operation of the mixing system. There is a free space between the roof of the digester and the liquid surface for gas storage. With fixed covers, less flexibility is provided for accommodating changes in liquid volume and gas pressure, requiring provisions for minimizing pressure fluctuation. Fixed covers have a lower capital cost than floating covers; however, they usually require a second vessel or a dedicated tank to store gas.

Floating covers fit the surface of the digesters and allow the volume of the digester to change without allowing air to enter the digester. Air and digester gas must not be allowed to mix, since the mixture may create an explosive environment. Floating covers complicate the design of mixing equipment but provide gas storage capacity.

A gas-holding floating cover with an external launder liquid seal often is recommended for secondary digester/storage reactors. This arrangement allows flexibility in digester operations and virtually eliminates the need for odor control.

With both types of covers, pressure valves, vacuum relief valves, and flame traps are required. A slight positive pressure (100 to 200 mm) is maintained under the covers to prevent suctioning air into the tank. Observation ports and manholes are commonly provided in the cover design.

Proper digester mixing is intended to ensure process uniformity and also to prevent scum formation and grit accumulation. Energy requirements for prevention of grit and scum are much larger than for mixing alone, and therefore are controlling factors. Conventional mixing power design requirements are from 0.2 to 0.3 hp/1000 ft³ (6 to 8 kw/1000m³) and digester mixing systems broadly fall into two categories: mechanical systems and gas systems.

Mechanical digester mixing systems include recirculation pumping, confined draft tube type mixers, and unconfined mechanical mixers. Recirculation pumping commonly is used. For the recirculation system to be effective, the pumps need to be adequately sized to provide complete turnover every 20 to 30 minutes. Mechanical mixing using draft tubes provides good top-to-bottom mixing and helps prevent scum buildup; however, draft tubes are sensitive to liquid level. Maintenance issues for mechanical systems relate to impeller, gear box, bearings, and plugging by rags.

Mixing with digester gas often is used in conventional design of first- and second-stage anaerobic digesters. Gas mixing systems include unconfined gas mixing, gas injection, and confined gas lifting. Unconfined gas mixing systems introduce gas to the digester through spargers or diffusers mounted near the bottom of the digester. Gas injection systems supply gas to the digester through tubes, or lances, which are typically located several feet above the floor. Confined gas lifting systems supply gas to draft tubes located in the digester, either through injection tubes or through a large bubble generator.

An advantage of the confined gas lifting system over mechanical mixers is the absence of moving parts and good mixing efficiency. However, gas injection systems also are more prone to plugging and maintenance problems because of submergence. A dual mixing system, such as a combination of recirculation pumping and a gas injection system, may be used to take advantages of each type's special features.

Maintaining constant digester temperatures is essential in order to obtain optimal anaerobic process performance. To maintain this constant temperature, supplemental heat is provided to the incoming sludge. The supplemental heat must also compensate for heat losses from the walls, roofs, and floor of the tanks.

Internal and external heat exchangers are the two most commonly used heating methods. External heat exchangers are preferred due to ease of maintenance and operation. Water baths, jacketed pipe, and spiral pipe design are the most common external heat exchanger designs. The goal of any heat exchanger design is to limit temperature variations to less than 1°C/day, since changes greater than that can cause process upsets.

Another method of heating the sludge is hot water jackets on gas pistons or draft tubes. Internally installed pistons and tubes offer an excellent heat transfer efficiency. The most important source of energy for anaerobic digesters is the digester gas. Digester gas can be used to fuel a boiler system that heats water. The

hot water is then used in the heat exchangers or water jackets to maintain sludge temperature. Exhaust gas may be collected, compressed, and sparged into the recarbonation units to recover the carbon dioxide.

The two basic types of anaerobic single-stage digestion processes are standard-rate and high-rate. Standard-rate digestion is illustrated by Figure 6.121 and is typically an unheated system operating in the mesophilic range. Detention times may be up to 30 to 60 days. Volatile suspended solids loading rates are low at 0.04 to 0.1 lb/ft³/day (0.6 to 1.6 kg/m³/day). There is intermittent feeding and withdrawal of sludge from the stratified digester.

The basic high-rate digester (see Figure 6.122) is heated and operates in the higher mesophilic range, 85 to 95°F (29 to 35°C). As a result, detention times approach 20 days. The range of volatile suspended solids loading rate is 0.1 to 0.4 lb/ft³/day (1.6 to 6.4 kg/m³/day). The mixed reactor (digester) may be continuously or intermittently feeding sludge with the associated withdrawals.

As shown in Figure 6.123, the features of both basic systems may be combined for two-stage digestion. The two-stage process may operate at various loading rates. The first digester is used for digestion, and the second concentrates and stores digested sludge. When sludge from the second digester is recirculated to the first, it provides seed for stabilization of the raw sludge. This modification should be considered as a viable alternative to single-stage digestion.

Egg-shaped digesters (ESDs) have found wide acceptance within Europe and are gaining popularity in the United States for applications in medium to large facilities. An ESD has a smaller digester volume requirement, because it can be operated at a maximum solids loading.

A single-stage, conventional digester design normally includes gas storage. Since the ESD system provides only minimal gas storage volume, a second vessel or dedicated storage tank is usually required. The energy required for mixing and scum resuspension for an ESD is less than that of conventional digesters. ESDs usually are mixed with a single internal draft tube and a backup gas sparging system.

Conventional digesters require periodic cleaning, typically every 3 to 8 years, which requires shutdown and dewatering. The sides of an ESD form a cone so steep at the bottom that, with appropriate mixing, grit

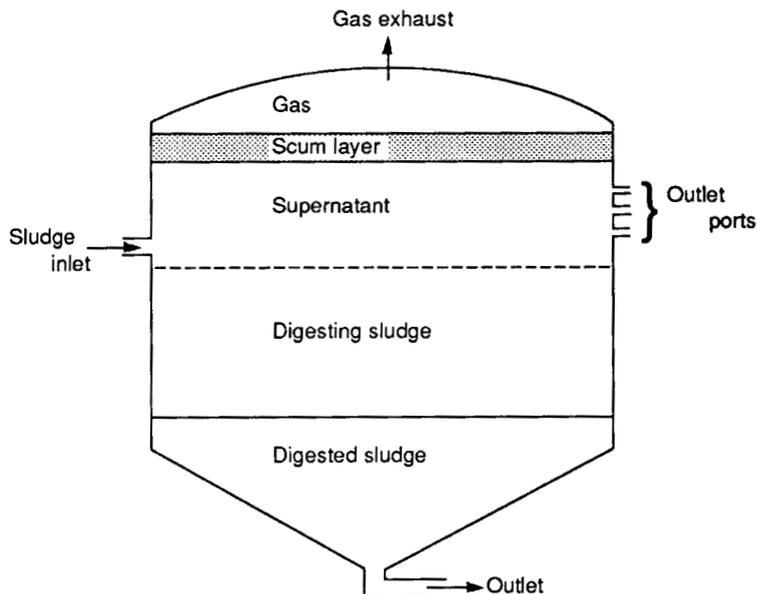


FIGURE 6.121 Standard-rate anaerobic digestion system.

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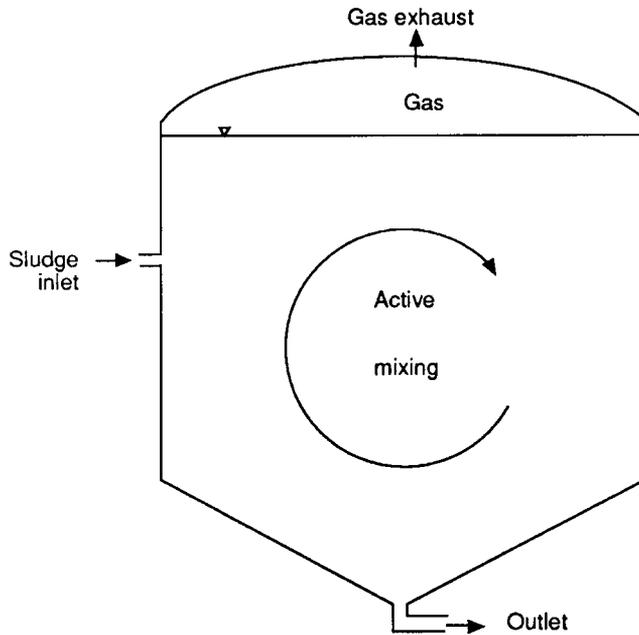


FIGURE 6.122 High-rate anaerobic digestion system.

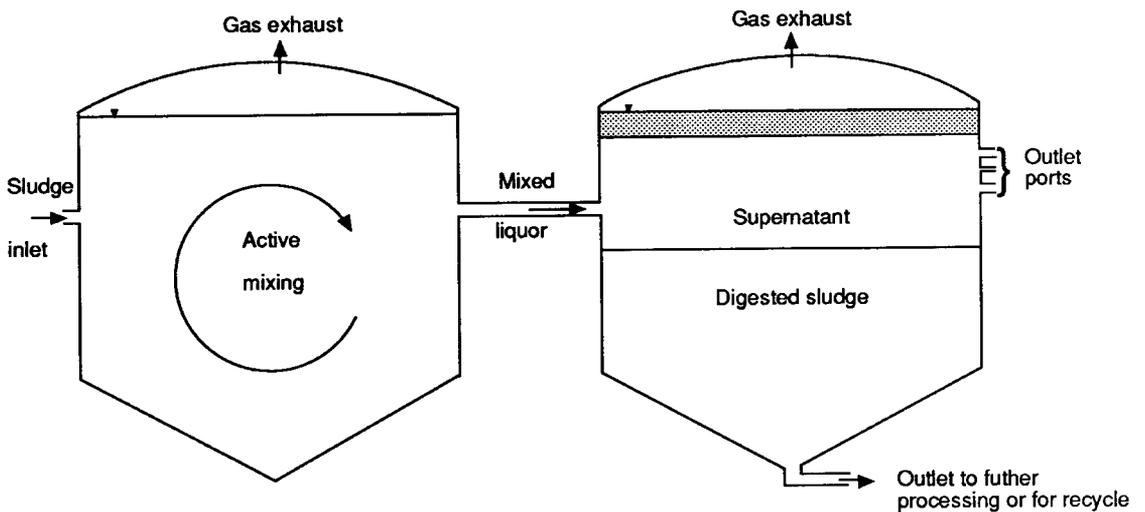


FIGURE 6.123 Two-stage anaerobic digestion system.

will not accumulate. Scum generation is much lower due to the steep upper cone and the smaller surface area. The draft tube mechanical mixers or a small surface mixer helps break up any scum that forms.

ESDs have a smaller footprint and also provide some space below the egg's curvature for additional equipment and piping. Since the ESD is a gas-tight design, the release of odors and/or flammable gases is prevented.

A cost comparison of conventional and egg-shaped digesters is needed to determine the feasibility for medium to large plant applications. The comparison should consider digesters with similar mixing capability, recirculation pumping, heating, external heat exchangers, as well as gas utilization/wasting considerations.

In thermophilic systems, higher temperatures are used to speed reaction rates and increase organism destruction rates. This approach should be considered for expanding existing digestion system capacity and for layer installations. The alternative analysis will be comparing increased energy costs for heating versus capital costs for conventional digesters.

Regardless of the selected process, the design should consider recovery and reuse of digester gas. For larger systems, recovered methane can become a valuable energy source. This consideration also comes into effect when analyzing mesophilic versus thermophilic systems. The fuel value of digester gas is approximately 600 Btu/ft³ (22 kJ/L).

Supernatant, with its high concentration of solids and BOD, is usually withdrawn in small quantities and is recycled through the treatment works (by discharge to primary clarifier influent). Mixed (high-rate) systems must be allowed to settle before decanting supernatant unless additional tankage is provided for the separation process.

Chemical Stabilization. Chlorine oxidation and lime treatment are chemical methods used to stabilize and disinfect sludges (4, 50, 86). The cost of chemicals and secondary effects limit broad use of these stabilization processes.

Chlorine Oxidation. Application of high doses (in excess of 2000 mg/L) of chlorine gas directly to sludge in an enclosed reactor produces a stabilized sludge suitable for dewatering. Beds and filter presses are often used for dewatering. Chemical conditioning is required for all mechanical dewatering systems.

The sludges and supernatants have a low pH, which may require adjustment. Also, chlorinated compounds and undissolved heavy metals may be undesirable by-products of chlorine oxidation.

Lime Treatment. In the lime stabilization process, lime is added to untreated sludge in sufficient quantity to raise the pH to 12 or higher. Either hydrated lime, Ca(OH)₂, or quick lime, CaO, may be used for the lime treatment. The high pH and high temperature environment reduces pathogens, eliminates offensive odors, and reduces potential for putrefaction. Results of an EPA study show that the pH must be maintained between 12.2 and 12.4 to ensure that the sludge is stabilized and the pH should remain above 11.0 for better than 2 weeks (87).

Lime stabilization requires more lime than conditioning processes and does not cause destruction of organics. However, it does provide a single-treatment process that can be easily controlled and produces a sludge suitable for land application. Representative lime dosage requirements as a function of percent sludge solids are shown in Table 6.119.

Composting. Composting is an environmentally acceptable means of sludge stabilization. As the organic material decomposes, the compost heats to temperatures in the pasteurization range of 120 to 160° and pathogens are destroyed. Continued biological activity produces a stable end product. Approximately 20 to 30% of the volatile solids are converted to carbon dioxide and water. The composted material is used as a soil conditioner in agricultural or horticultural applications. Even though there are many variations, composting methods are generally contour or windrow spreading or mechanical (rotating drum or tower) systems.

TABLE 6.119 Lime Requirements in Chemical Stabilization (50)

Lime dosage to obtain desired pH	Sludge, % solids				
	1	2	3	3.5	4.4
pH = 11 Ca(OH) ₂ , mg/L	1400	2500	3700	6000	8200
pH = 12 Ca(OH) ₂ , mg/L	2600	4300	5000	9000	9500

The sequential steps usually involved in composting are preparation, digestion, curing, and finishing. Bulking materials, such as municipal solid waste, wood chips, or soil, are mixed with the sludge during the preparation stage.

The digestion stage requires several days (mechanical) or several weeks (spreading) to be completed. The exothermic reaction is increased with mixing (aeration) of the compost. Curing follows for a couple of weeks as the decomposition rate is decreased. The final stage, finishing, is when solid fractions are removed from the composted material.

A more detailed discussion of composting of municipal solid wastes and sludge is presented in Chapter 8. Also, data are available in numerous references (4, 16, 50, 86).

Heat Treatment. Sludge stabilization by heat treatment may be achieved by pasteurization or low-pressure oxidation (4, 50). The process is used to destroy pathogenic organisms, including disinfection of sludge, and is widely used in Europe. With increased interest in land application of sludges, this process has special interest, especially when applied to pastures or other food-chain crops.

Pasteurization systems require temperatures of 115–170°F (50–75°C) with exposure times of 30 to 60 min. High-range combinations are required to destroy viruses and coliform bacteria. Similar results may be achieved by low-pressure oxidation at temperatures of 350–400°F (175 to 205°C), pressures of 180 to 210 lb/in² (1250 to 1450 kN/m²), and retention times of 20 to 30 min. This process is often considered in conjunction with sludge-conditioning systems.

Irradiation. The future may find that irradiation is a cost-effective alternative for stabilizing sludge by destroying pathogens. However, irradiation is currently curtailed due to high costs, practical technology, and safety problems.

Lagooning. Lagoons can be designed as biological digesters for raw sludge stabilization. The limitations on this practice are the costs of land, potential aesthetic (odor) problems, and lack of operational flexibility. Also, several years may be required for the biological reactions to be complete. Thus, lagoons are most suitable for emergency applications or for rural locations. Also, lagoons may be specially designed for sludge dewatering.

Conditioning

Conditioning is a treatment process specifically designed to improve the sludge's dewatering characteristics. Chemical conditioning and heat treatment are the most common methods. Elutriation is a washing process that can be used to reduce chemical conditioner requirements. Special applications may consider conditioning by ash addition, freezing, or other methods. A summary of common conditioning methods and purposes is shown in Table 6.120.

TABLE 6.120 Sludge Conditioning Methods and Purposes (86)

Conditioning method	Affected unit process	Purpose
Polymer addition	Thickening	Improve loading rate, degree of concentration, and solids capture.
Polymer addition	Dewatering	Improve production rate, cake solids content, and solids capture.
Inorganic chemical addition	Dewatering	Improve production rate, cake solids content, and solids capture.
Elutriation	Dewatering	Decrease acidic chemical conditioner demand and increase degree of concentration.
Heat treatment	Dewatering	Eliminate or decrease chemical use, improve production rate, cake solids content, and stabilization. Some conversion may also occur.
Ash addition	Dewatering	Provide improved cake release from belt-type vacuum filters and facilitate filter pressing. It can also result in higher filter yields and reduced chemical requirements.

Chemical Conditioning. The use of chemical flocculants to coagulate sludge solids and release sorbed water is practiced frequently. The accompanying changes in the chemical and physical nature of the sludge significantly improve dewatering. Field and/or laboratory tests should be used to identify the most effective chemical type and dosage for the specific application.

Inorganic chemical conditioners most commonly used are ferric chloride, aluminum sulfate, and lime. Ferrous salts have had limited application. Organic conditioners include organic polyelectrolytes or polymers. Cationic polymers have the most application; however, anionic and nonionic polymers can be used effectively in conjunction with other chemicals. Ranges of requirements for the principal chemical conditioners are shown in Table 6.121.

Elutriation. Prior to chemical conditioning, elutriation has been widely used to remove components from anaerobically digested sludges that may interfere with subsequent dewatering operations or require larger quantities of chemicals for conditioning. Elutriation is a countercurrent or concurrent washing of soluble compounds and fine solids. An advantage of the system is that chemical costs may be significantly lower; however, the use of the operation is often restricted by the buildup of undesirable solids in the treatment plant.

One or more tanks can be used to perform single- or multistage washing. Plant effluent or potable water is mixed with sludge for 10 to 15 min. The mixing is achieved either mechanically or by use of diffused air. Ratios of sludge to wash water vary, but a typical ratio is 1:2.

Heat Treatment. At temperatures of 300 to 500°F (150 to 260°C) and pressures of 150 to 400 lb/in² (1035 to 2760 kN/m²), heat treatment conditioning may occur (86). The cellular composition of biological sludges breaks down, and the solid fraction is composed of mineral matter and cellular residue. Also, the process reduces water affinity of the residual solids. All of which produces a sludge conditioned for easier dewatering.

Concentration

Sludge concentration frequently precedes dewatering (and in some cases stabilization) processes for the purpose of thickening the sludge and withdrawing the supernatant. Since the thickened sludge has less total vol-

TABLE 6.121 Chemical Conditioner Requirements (*I6*)

Type of sludge	Chemical dosage, % dry sludge solids		
	FeCl ₃	Ca(OH) ₂	Polymer
Raw primary	1–3	0–5	0.15–0.25
Raw (primary and trickling filter)	3–6	0–15	0.2–0.5
Raw (primary and activated sludge)	4–8	0–15	0.3–0.75
Activated sludge	6–10	5–15	0.4–1.25
Digested primary	2–3	3–8	0.15–0.4
Digested (primary and trickling filter)	4–8	5–15	0.3–0.75
Digested (primary and activated sludge)	6–10	5–15	0.3–1

ume than the unthickened sludge, the time and costs involved with sludge dewatering are decreased. Thickening processes commonly are used when two sludges from separate stabilization processes are combined for dewatering.

The design of sludge thickening units should include consideration of the

- Type of sludge, e.g., waste activated sludge
- Concentration of sludge to be thickened and the desired resultant concentration
- Other processes used for sludge treatment and handling
- Sludge's stability
- Needs for chemical treatment
- Capital and operating costs
- Pumping and piping of the concentrated sludge
- Operating mode, i.e., batch or continuous

The methods used for sludge thickening are physical processes and most often include one of the following:

- Gravity settling
- Dissolved air flotation
- Centrifugation

A comparison of the use of these methods, as a function of sludge type, is presented in Table 6.122.

Gravity Settling. Gravity settling or thickening is a process wherein the sludge settles simply by the force of gravity and supernatant is removed by a basin overflow. The process is characterized by Class III and IV sedimentation, as discussed in the section on suspended solids treatment.

The physical characteristics of most gravity thickeners are similar to a conventional circular settling basin except that the sludge collection scrapers have greater capacity, and the bottom slope is steeper. Some of the common physical features are (*I6*)

Depth	10 to 12 ft (3.3 to 3.7 m)
Diameter	Up to 70 to 80 ft (21 to 24 m)
Bottom slope	1:6 to 1:4

Some agitation is desirable to dislodge entrapped liquid and gas bubbles, prevent bridging of sludge solids, and keep the thickened sludge moving toward the center collection well from which it is removed.

TABLE 6.122 Comparison of Sludge Thickening Methods (86)

Method	Type sludge	Other pertinent considerations	Frequency of usage and relative success
Gravity settling	Primary	Separate waste activated sludge thickening	Increasing, excellent results
Gravity settling	Digested primary	Separate waste activated sludge thickening	Infrequent now; but feasible
Gravity settling	Primary and waste activated	Recirculation of waste activated sludge to primaries	Decreasing, usually poor results
Gravity settling	Primary and waste activated	Direct mixed sludge thickening	Some new installations, marginal results
Gravity settling	Air waste activated	Separate primary sludge thickening	Essentially never used, poor results
Gravity (elutriation)	Digested primary and waste activated mixture	Secondary digesters or elutriation	Many plants built, requires flocculants
Dissolved air flotation	Primary and waste activated	Direct mixed sludge thickening	Rarely used
Dissolved air	Air waste activated	Separate gravity thickening of primary sludge	Increasing, good results
Solid bowl conveyor type	Waste activated	Separate gravity thickening of primary sludge	Some limited use, solid capture problem
Disk-type centrifuge	Waste activated	Separate gravity thickening of primary sludge	Some limited use, data now being accumulated

This mixing is usually mechanical and is provided in conjunction with the sludge collection mechanism. The characteristics of the sludge must be assessed as part of the design process.

The principal design consideration for the gravity thickener is the area required to achieve the desired concentration of withdrawn sludge (see Class IV sedimentation). This design parameter is expressed as the solids surface loading rate, measured in pounds per square foot per day (kilograms per square meter per day). Typical design and performance data for gravity thickeners for different types of sludges are shown in Table 6.123. Chemical coagulants can be used to increase loading rates and to improve settling and thickening characteristics.

TABLE 6.123 Gravity Thickener Design Data (50)

Type of sludge	Solids surface loading rate, lb/ft ² /day (kg/m ² /day)	Thickened sludge solids concentration, %
Primary	20–30 (98–146)	8–10
Primary with lime addition	20–25 (98–122)	7–12
Primary and trickling filter	10–12 (49–59)	7–9
Primary and waste activated	6–10 (29–49)	4–7
Trickling filter	8–10 (39–49)	7–9
Waste activated	5–6 (24–29)	2.5–3
Tertiary with lime addition	60 (293)	12–15

The hydraulic loading to the gravity thickener is typically 200–800 gpd/ft² (8 to 33 m³/day/m²). The rate should be low enough to permit effective settling and thickening, but high enough to control odors and the formation of septic conditions. The latter may also be controlled by the application of chlorine; a residual of around 1 mg/L should be sufficient.

The sludge volume ratio (SVR) is another important design parameter and is defined as the ratio of the sludge blanket volume to the daily volume of removed sludge. The SVR has units of days, which provides a relative approximation of the sludge retention time or the time required for thickening. A typical range of SVR values is 0.5 to 2 days, with less time required during warm weather. Settling column tests may be used to develop an SVR for a specific sludge (see Class IV sedimentation).

Dissolved Air Flotation. For sludges that resist compaction by gravity settling and require long settling periods, dissolved air flotation (DAF) may be used. Also, a flotation thickener system provides a high solids concentration and lower capital cost than a gravity thickener.

In this process, air is injected into the sludge under pressure. The resulting small air bubbles lift the particles of sludge and carry them to the surface for removal. The principles of flotation were discussed in the section on suspended solids treatment; the physical features of circular and rectangular units were illustrated in Figures 6.43 and 6.44, respectively. These data should be reviewed in conjunction with the following.

Design data for flotation thickeners is presented in Table 6.124. Other design considerations are detention time, typically 30 min, and feed concentration, which varies significantly as a function of the sludge being treated. The variability of sludges suggests that at least laboratory-scale tests be conducted during the design process.

The equipment required for conducting a laboratory flotation test includes a compressed air source, pressure gauge, pressure tank, and flotation cell. Since the surface area is limited by the equipment, the detention time is fixed. However, the laboratory flotation system is useful in estimating the air to solids ratio for a desired thickened sludge (float) solids concentration. Also, the laboratory test may be used to evaluate the effects of chemical additives. Equipment manufacturers and their literature provide more detailed test procedures.

The float is scraped from the DAF thickener to a storage hopper to permit the release of entrained air before the next treatment process or final disposal. The initial density of the float, which is commonly about 6 lb/gal (700 kg/m³), is used to size the storage hopper. Also, it is noted that provisions should be made for collection and removal of solids that may settle in the flotation thickener.

TABLE 6.124 Flotation Thickener Design Data (16, 50)

Parameter	Typical value	Range
Pressure	40–60 lb/in ² (270–410 kN/m ²)	30–80 psig (205–540 kN/m ²)
Air-to-solids ratio	0.02	0.02–0.04
Solids loading without chemicals	10–20 lb/ft ² /day (50–100 kg/m ² /day)	8–48 lb/ft ² /day (40–240 kg/m ² /day)
Solids loading with chemicals	25–50 lb/ft ² /day (125–250 kg/m ² /day)	25–60 lb/day/ft ² (125–300 kg/m ² /day)
Hydraulic loading	1150 gal/ft ² /day (460 m ³ /m ² /day)	575–3000 gal/ft ² /day (230–1200 m ³ /m ² /day)
Float thickness	8–12 in (20–30 cm)	6–24 in (15–60 cm)
Recycle	100%	30–200%
Float solids concentration*	3–5%	2–10%
Solids removal	95–99%	75–99%

*Chemical treatment increases float solids concentration by about 1%.

Centrifugation. In theory, centrifuges can be used for sludge thickening; however, high costs and maintenance requirements have limited their use for this purpose. General information on centrifugation was presented in the section on suspended solids treatment.

Basket, disk, and solid bowl centrifuges have been evaluated for use in sludge thickening, and the respective solids concentrations were 9 to 10%, 4 to 6%, and 5 to 13% (86). These units will be most feasible when space is limited or other thickening methods are ineffective.

The disk centrifuge is generally best suited for handling large flows with relatively low concentrations of fine solids. The machine is illustrated in Figure 6.124.

Dewatering

Dewatering is a sludge-handling process that removes sufficient water so that the sludge transforms from a fluid state to that of a damp solid or sludge cake. The more common dewatering devices are centrifuges, drying beds, lagoons, vacuum filters, pressure filters, and belt filter presses.

The design engineer should carefully consider each method in the alternative analysis for a specific sludge. Some of the relationships between dewatering and pre- and postprocess are shown in Table 6.125.

Centrifugation. By using centrifugal force to increase the sedimentation rate of sludge solids, centrifuges have their best application in dewatering sludges. The basic variables affecting design and operation of centrifuges are sludge feed rate, sludge solids characteristics, sludge consistency, temperature, and chemical conditioning.

Solid bowl, basket, and disk are the most common centrifuge types. The disk type has application for concentrating solids but is less effective for sludge dewatering. The auxiliary equipment for centrifuge dewatering includes

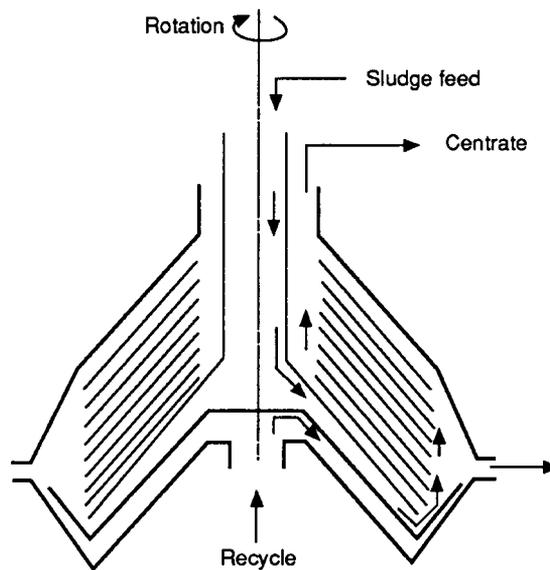


FIGURE 6.124 Disk centrifuge (86).

TABLE 6.125 Relationship of Dewatering to Other Sludge-Handling Processes (86)

Method	Pretreatment normally provided		Normal use of dewatered cake			
	Concentration	Conditioning	Landfill	Land spread	Heat drying	Incineration
Centrifuge (solid bowl)	Yes	Yes	Yes	Yes	Yes	Yes
Centrifuge (basket)	Variable	Variable	No	Yes	No	No
Drying beds	Variable	Not usually	Yes	Yes	No	No
Lagoons	No	No	Yes	Yes	No	No
Filter presses	Yes	Yes	Yes	Variable	Not usually	Yes
Horizontal belt filters	Yes	Yes	Yes	Yes	Yes	Yes
Rotary vacuum filter	Yes	Yes	Yes	Yes	Yes	Yes

1. Chemical conditioning process equipment
2. Sludge feed pumps
3. Central pump and piping
4. Washwater system
5. Sludge cake handling equipment

Solid Bowl Centrifuge. The solid bowl centrifuge is a continuously operating unit and may be a counter-current (Figure 6.125) or concurrent (Figure 6.126) model. The principal machine variables that affect per-

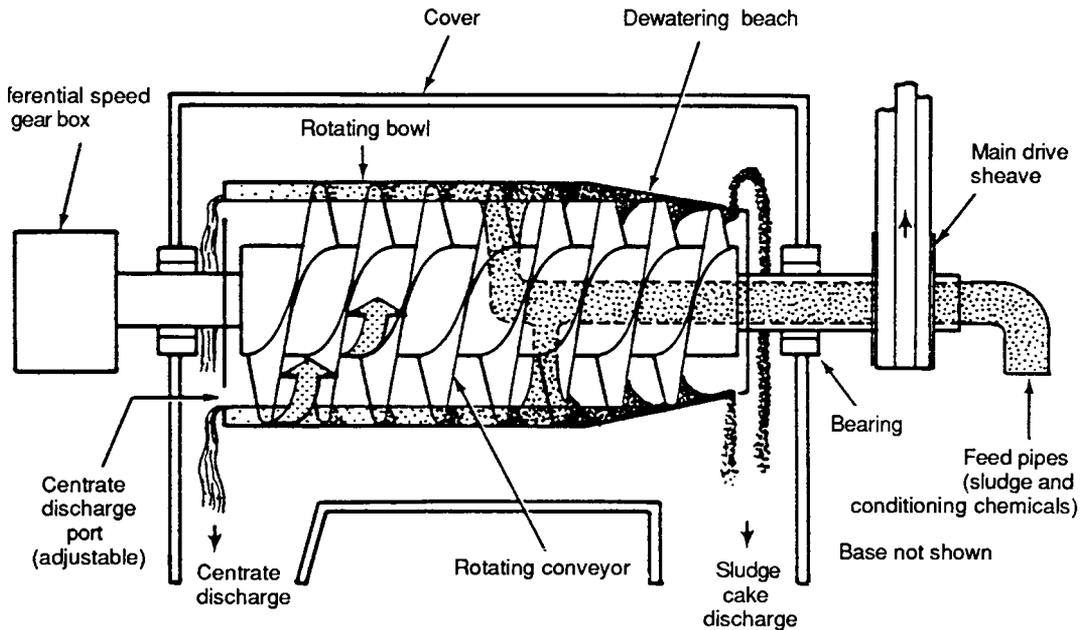


FIGURE 6.125 Countercurrent solid bowl centrifuge (88).

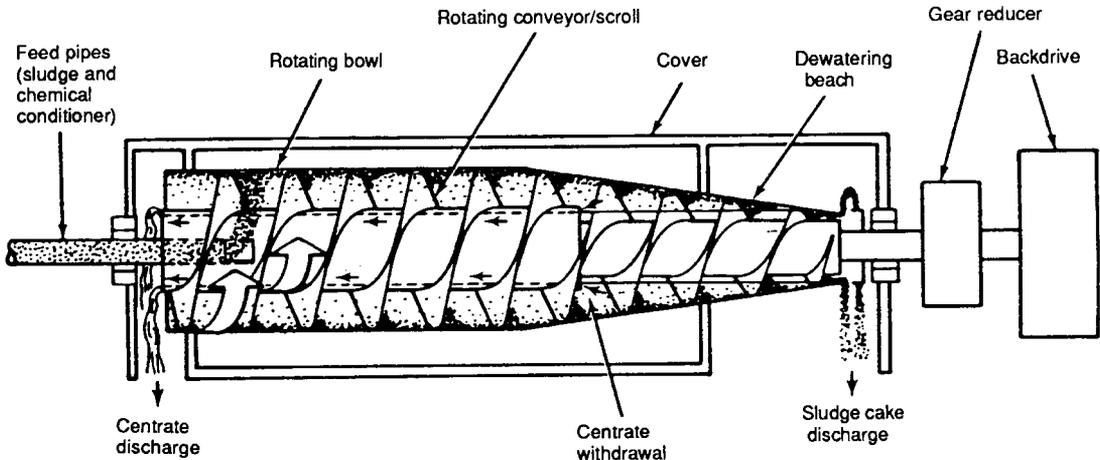


FIGURE 6.126 Concurrent solid bowl centrifuge (88).

formance are the bowl design, bowl speed, pool volume, conveyor speed, conveyor pitch, and sludge feed rate. Typical performance data for a variety of sludges are shown in Table 6.126.

Bowl design considers the ratio of bowl length to diameter. Desired ratios may range from 2.5:1 to 4:1; the higher ratios provide increased clarification capacity. Bowl speed is also a primary factor in clarification. The sludge solids settling rate is a function of the velocity squared. On the negative side, maintenance demands increase with higher speeds.

In general, machines with relatively large pool volumes (i.e., deep pools), are most suitable for dewatering. This occurs because of longer detention times and reduced forces on the sludge. Pool depth may be field-adjusted for improved clarification and sludge cake dryness.

The difference between bowl speed and conveyor speed determines the solids detention time. Also, the

TABLE 6.126 Centrifuge Dewatering Performance Data (16, 88)

Type of sludge	Solids concentration, %	
	Solid bowl centrifuge	Imperforated basket centrifuge
Undigested sludges		
Primary	25–36	25–30
Primary and trickling filter	20–30	14–20
Primary and activated sludge	18–25	11–17
Trickling filter	—	9–12
Activated sludge	8–14	8–14
Digested sludges		
Primary	22–32	18–25
Primary and activated sludge	15–21	10–14
Activated sludge	8–15	6–10
Heat-treated digested sludges		
Primary and activated sludge	30–40	—

conveyor pitch may be altered to increase the length of the clarifying zone with a steeper pitch. The conveyor speed and pitch design must also consider the need to minimize turbulence and provide adequate carrying capacity.

The sludge feed rate (volume versus time) affects clarity and sludge cake dryness; a balance is needed for desired results. For example, a larger feed rate means less detention and decreased solids recovery due to loss of fines; however, a drier sludge cake is produced. The sludge feed inlet design is also important to reduce turbulence; the concurrent flow machine lessens this impact.

Basket Centrifuge. Figure 6.127 illustrates a basket centrifuge, which may be of the perforated or imperforated type. Only the imperforated type is well suited for sludge dewatering. The machine rotates on a vertical axis and operates on semicontinuous feeding and sludge cake discharge. Performance data appear in Table 6.126.

The basket centrifuge has a lower capacity than the continuous-feed solid bowl centrifuge. However, the basket centrifuge is often more desirable because of its capability for higher solids recovery without chemical conditioning.

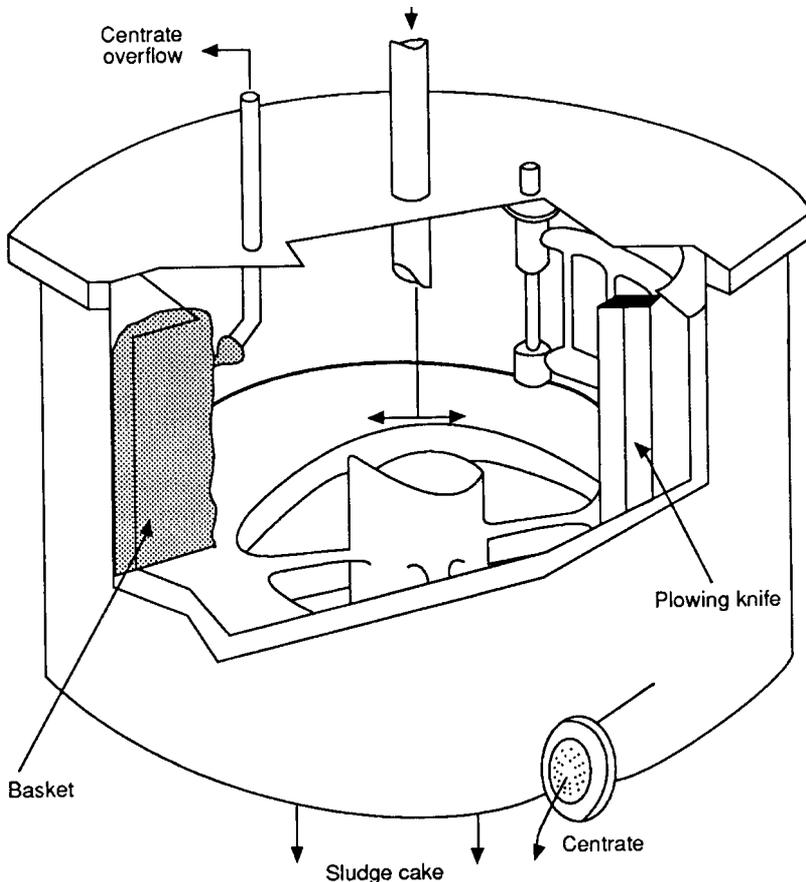


FIGURE 6.127 Basket centrifuge (86).

Drying Beds. The use of drying beds is most applicable to smaller facilities, since the process is weather-dependent as well as land- and labor-intensive. The operating principles are dewatering by evaporation and by percolation through the sludge and underlying media.

The sludge should be well stabilized to minimize odors and to improve dewaterability. In some cases, concentration is beneficial to improve the effectiveness of drying beds. Also, covers or enclosures (with good ventilation) should be considered in areas of high or frequent precipitation.

Although sand drying beds are most common, other types include paved, wedge wire, and vacuum-assisted drying beds.

Sand Drying Beds. As illustrated in Figure 6.128, sand drying beds are constructed with fine- to coarse-graded sand and gravel layers covering an open-joint pipe underdrain system. Advantages and disadvantages of sand drying beds are listed in Table 6.127.

Design criteria for sand drying beds are presented in Table 6.128. The sizing (area requirement) is generally at the high range when the beds serve as a primary dewatering process.

Following a batch application, the sludge is allowed to drain and dry until it cakes and cracks. At this stage, the sludge solids content reaches 20 to 40%. The beds may be cleaned either manually (30 to 40% solids cake) or mechanically (20 to 30% solids cake).

Appurtenances include a splash pad to prevent the incoming sludge from scouring sand and gravel and thus causing short-circuiting. Also, reinforced concrete running boards should be installed down the center of the beds so that trucks or other equipment may be used during sludge removal. A typical running board will be 5 ft (1.5 m) wide by 6 in (15 cm) deep and run in pairs on 3-ft (1-m) parallel centers. The distance between pairs will vary around 20 ft (6 m). Finally, provisions should be made to collect the underdrainage and return it to the treatment works.

Paved Drying Beds. With paved drying beds, mechanical sludge removal equipment can be used without damaging the underdrain piping. The beds are paved with asphalt or concrete and sloped toward the center where an underdrain is installed beneath the bed. The principal drawback of this system is the increased capital cost of pavement.

Wedge-Wire Drying Beds. In essence, wedge wire replaces sand as the medium in wedge-wire drying

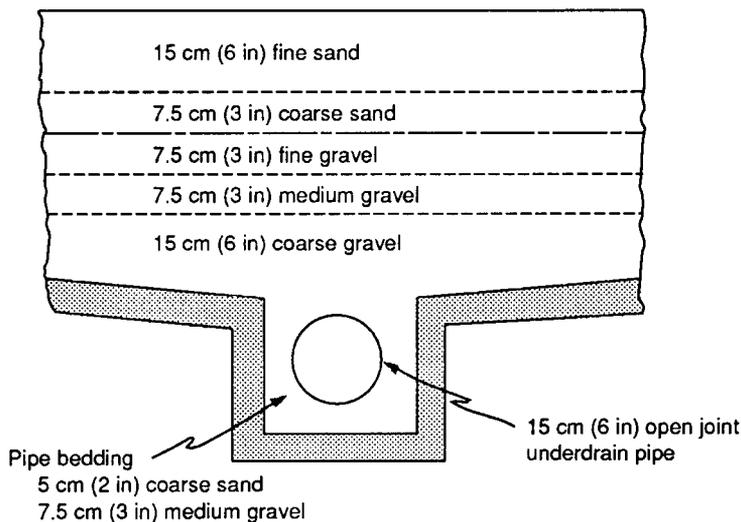


FIGURE 6.128 Sand drying bed.

TABLE 6.127 Advantages and Disadvantages of Sand Drying Beds (88)

Advantages	Disadvantages
Low capital cost—excluding land	Weather conditions such as rainfall and freezing weather have an impact on usefulness
Low operational labor and skill requirement	Requires large land areas
Low energy	High labor requirement for sludge removal
Low maintenance material cost	May be aesthetically unpleasing, depending on location
Little or no chemicals required	Potential odor problem with poorly stabilized sludge
High cake solids content possible	

beds. The wedge wire is placed in an open concrete basin, which may be a new structure or a retrofit of an existing sand drying bed (see Figure 6.129).

Since excess water cannot return to the sludge through capillary action, the sludge dewateres faster with this process. Also, tiltable units are available for easier sludge cake removal. The benefits of this relatively new process must be evaluated with respect to costs and operating requirements.

Vacuum-Assisted Drying Beds. A small vacuum can be used to speed dewatering of sludge applied to a porous medium plate set above an aggregate-filled support plenum which drains to a sump. This approach is relatively new and only a few vacuum-assisted drying beds are yet in service. Current performance data should be sought by the design engineer considering this process.

Drying Lagoons. Sludge lagoons can be utilized to store sludge or as a dewatering process. The main design difference is that drying lagoons are relatively shallow. Supernatant is decanted to hasten the drying.

TABLE 6.128 Sand Drying Bed Design Data (16, 46, 50, 86, 88)

Parameter	Typical value
Minimum number	Two
Shape	Rectangular
Length	20–100 ft (6–12 m)
Width	20 ft (6 m)
Sand layer	
Depth	9 in (23 cm)
Effective size	0.3–1.0 mm
Uniformity coefficient	Less than 4.0
Gravel layer	
Depth	12 in (30 cm)
Grading	0.12–1 in (3.2–25 mm)
Underdrain system	
Pipe size	4 in (10 cm) minimum
Spacing	Less than 20 ft (6.1 m)
Slope	1%
Freeboard above sand	12–18 in (30–45 cm)
Area requirements	
Open	1–2 ft ² /cap (0.09–0.18 m ² /cap)
Covered	0.7–1.5 ft ² /cap (0.06–0.13 m ² /cap)
Application depth	8–12 in (20–30 cm)

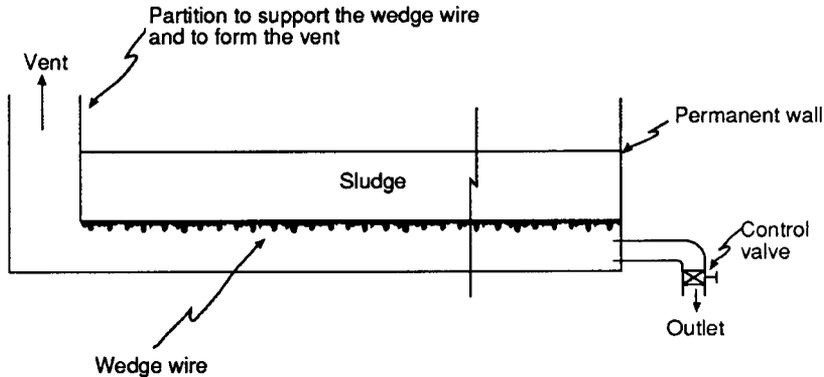


FIGURE 6.129 Wedge wire drying bed.

Typical design data are presented in Table 6.129. Other design considerations include diversion of surface water, provision of a ramp or other access for sludge removal equipment, and concern for aesthetics.

The use of drying lagoons is often restricted because of land requirements, odor and aesthetic controls, precipitation and climate patterns, and subsoil and groundwater conditions. Nonetheless, this low-cost, simple-to-operate system has its greatest application at very small facilities.

Vacuum Filters. Until the last decade, vacuum filters were the most common mechanical dewatering process, with special appeal to plants using trickling filter biological treatment units. A vacuum filter is simply a horizontal cylindrical drum that rotates partially submerged (20 to 30%) in a pool of conditioned sludge. A vacuum is used to extract filtrate and allow a sludge cake to form on the filter medium. A vacuum filter system is illustrated in Figure 6.130, and design and performance data are presented in Table 6.130.

The principal types of vacuum filters are the drum, coil, and belt. A cloth medium is used on the drum and belt types, and stainless steel springs are the medium for the coil type. The drum filter differs from the other two types in that the medium does not leave the drum and is washed in place. Operating data do not suggest substantial increases in yield when operated at vacuums greater than 15 in (38 cm) of mercury.

TABLE 6.129 Drying Lagoon Design Data (50, 86, 88, 89)

Parameter	Typical value
Minimum number Dikes	Two
Exterior slope	1:2
Interior slope	1:3
Depth	0.5–4 ft (0.15–1.2 m)
Depth to groundwater	Greater than 4 ft (1.2 m)
Loading rates	
Capacity	2.2–2.4 lb/ft ³ /year (35–38 kg/m ³ /year)
Surface (30–day)	1.7–3.3 lb/ft ² (8–16 kg/m ²)
Population	1–4 ft ² /capita (0.09–0.37 m ² /capita)
Decant	Single or multiple outlet
Sludge removal	1.5 to 3 years

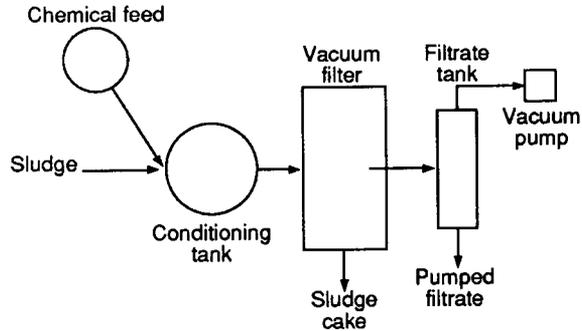


FIGURE 6.130 Vacuum filter system.

As the drum rotates, sections can be placed under vacuum or pressure. As the drum revolves through the sludge pool, a vacuum is applied to attach a mat of sludge of suitable thickness to the medium. The emerging mat is placed under a dewatering and drying vacuum, and the filtrate is drawn off for separate treatment. The resulting sludge cake is removed from the drum by a scraper; a slight pressure can be applied to facilitate removal of the cake. Figure 6.131 illustrates the three operating zones.

Pressure Filters. The recessed plate press is the more common pressure filter used for sludges. The two types are the fixed- and variable-volume recessed plate filter presses. The variable-volume unit is also called a diaphragm filter press and is a relatively new system. The recessed plate filter press is often confused with the plate-and-frame filter press, which has industrial and European experience but has decreasing acceptance for municipal sludges with the development of the recessed plate units. In general, chemical conditioning must precede the filter press, and applications are only cost effective for larger facilities.

Increased automation, improved filter medium, and larger unit capacities have led to renewed interest in pressure filtration. The ability to produce higher cake solids concentrations, improve filtrate clarity, improve solids capture, and reduce chemical consumption are considerations in favor of pressure filters. Disadvantages include manual assistance in cake removal, batch operation, as well as high capital and operating costs.

TABLE 6.130 Vacuum Filter Design and Performance Data (16, 88)

Type of Sludge	Loading rate, lb/ft ² /h (kg/m ² /h)	Sludge cake, % total solids
Undigested sludges		
Primary	5–10 (24–49)	25–32
Primary and trickling filter	3–6 (15–29)	20–28
Primary and activated sludge	2–5 (10–24)	16–26
Activated sludge	1–2 (5–10)	12–20
Digested sludges		
Primary and trickling filter	4–6 (20–29)	20–28
Primary and activated sludge	3–5 (15–24)	20–24
Lime (raw primary) sludge		
Low lime	3–6 (15–29)	25–30
High lime	5–10 (24–49)	30–40
Polymer (raw primary) sludge	8–10 (39–49)	25–38

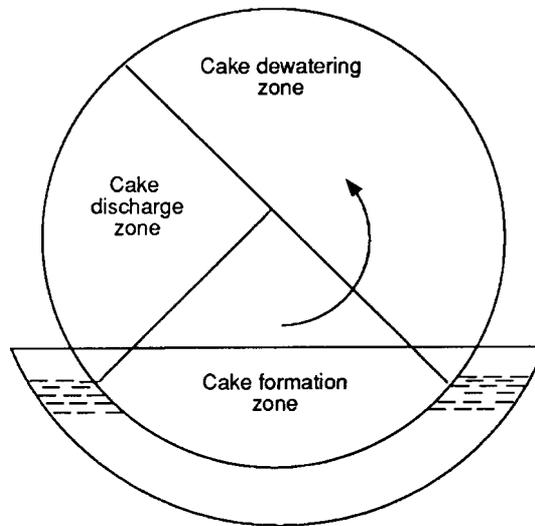


FIGURE 6.131 Vacuum filter operating zones.

The conventional filter press consists of vertical plates that are held in a frame and pressed together between a fixed and moving end (see Figure 6.132). Filter cloth is mounted on the face of each plate. With the diaphragm units, a rubber diaphragm supports the filter cloth on one side of the press cavity.

Sludge is fed into the press under pressure while water passes through the filter cloth, sludge solids are retained, and a cake is formed. When the cavities are filled, the sludge feed is stopped and the pressing cycle begins and continues until the flow of filtrate is nil. The total processing cycle includes filling, pressing, removing the cake, washing the medium, and reclosing the press.

The diaphragm press uses a rubber diaphragm to further compress the sludge after application of low pressure. The advantages of this press over the fixed-volume press are a dryer cake with a relatively uniform moisture content, an overall shorter operating cycle (higher output per work shift), no precoat required, and other advantages in ease of operation. On the other hand, the principal disadvantage is that the diaphragm press has an initial cost of two to three times that of the fixed-volume press.

The components of a common filter press system are shown in the flow diagram presented in Figure 6.133. Typical design data and performance data for filter press units appear in Tables 6.131 and 6.132, respectively. The design engineer should consult equipment manufacturers and their reference installations during the detailed design for a specific facility.

Belt Filter Presses. Several designs are available that use the principles of gravity and pressure to dewater sludges. Belt filter presses are a popular design that follow gravity dewatering with a compression and shear operating zone. Figure 6.134 illustrates these zones and the components of a belt filter. Also included are belt-washing sprays to keep the belts clean. Manufacturers should be consulted for current design and performance history. Typical performance data are presented in Table 6.133.

Like the belt filter, there are three other horizontal types of mechanical dewatering systems. The moving-screen concentrator uses a two-stage variable-speed moving screen (see Figure 6.135). Another unit, the capillary dewatering system (see Figure 6.136), uses capillary action for dewatering after a gravity drainage stage. Finally, the rotating gravity concentration system consists of two independent cells formed by a filter cloth; dewatering occurs in the first cell and cake formation in the second (see Figure 6.137).

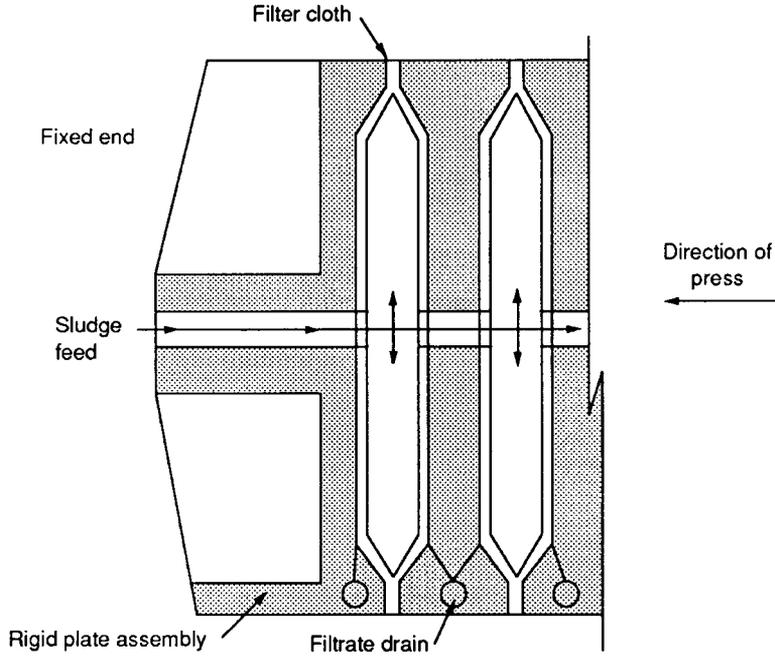


FIGURE 6.132 Filter press.

Drying

Sludge drying is generally a thermal process whereby both the volume and weight of sludge are reduced. Drying systems are characterized by the following:

- Heat treatment
- Incineration
- Pyrolysis
- Starved air combustion
- Wet-air oxidation

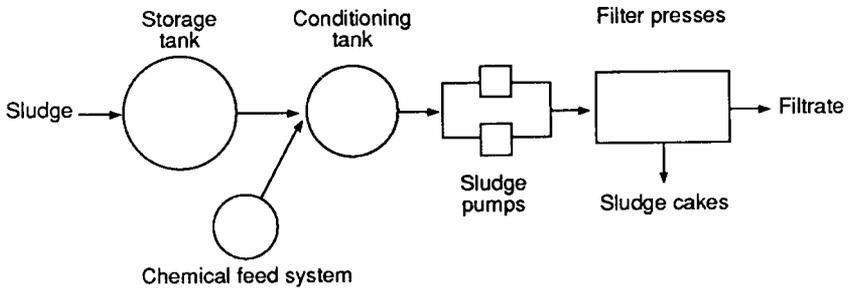


FIGURE 6.133 Filter press system.

TABLE 6.131 Pressure Filter Design Data (16, 50, 88)

Parameter	Typical value
Chamber volume	0.75–2.8 ft ³ (0.02–0.08 m ³)
Number of chambers	Up to 110
Filter area per chamber	14.5–45 ft ² (1.3–4.2 m ²)
Pressing cycle time	1.5–4 h
Total operating cycle time	2–6 h
Operating pressure	100–225 lb/in ² (700–1500 kN/m ²)
Sludge cake thickness	1–2 in (2.5–5 cm)
Solids content	25–50%

Only heat treatment provides a residue suitable for use as a soil conditioner. Pyrolysis yields a char that has some fuel value. Also, all these processes produce a residue suitable for landfilling.

If the sludge is nonhazardous and sufficient land is readily available, the benefits of sludge drying may not be cost effective. However, with decreasing land availability, increasing landfilling regulations, and increasing energy costs, drying processes become more feasible. In particular, the energy factor may make combined incineration of municipal refuse and sludge for energy recovery an attractive approach to satisfy local solid waste disposal needs.

Heat Treatment. In the heat treatment drying process, moisture in the sludge is evaporated by the introduction of a hot gas (air) stream; the dried sludge contains 8 to 10% solids. With temperatures around 700°F (370°C), there is no combustion of sludge solids in this process; however, most systems can be modified or expanded to include incineration or solids combustion.

Complete heat drying systems are available commercially, including wet scrubbers and dry solids collectors for air pollution (particulate and odor) control. The basic steps of a heat treatment system are

- Blending of wet and dried sludge
- Drying

TABLE 6.132 Pressure Filter Performance Data (16, 88)

Type of sludge	Solids concentration, %	
	Fixed-volume filter press	Diaphragm filter press
Undigested Sludges		
Primary	40–50	45–54
Primary and trickling filter	—	40–50
Primary and activated sludge	30–42	38–47
Activated sludge	24–35	31–42
Digested sludges		
Primary and trickling filter	—	38–48
Primary and activated sludge	35–45	—
Activated	24–40	—
Heat-treated digested sludges		
Primary and activated sludge	45–55	50–60

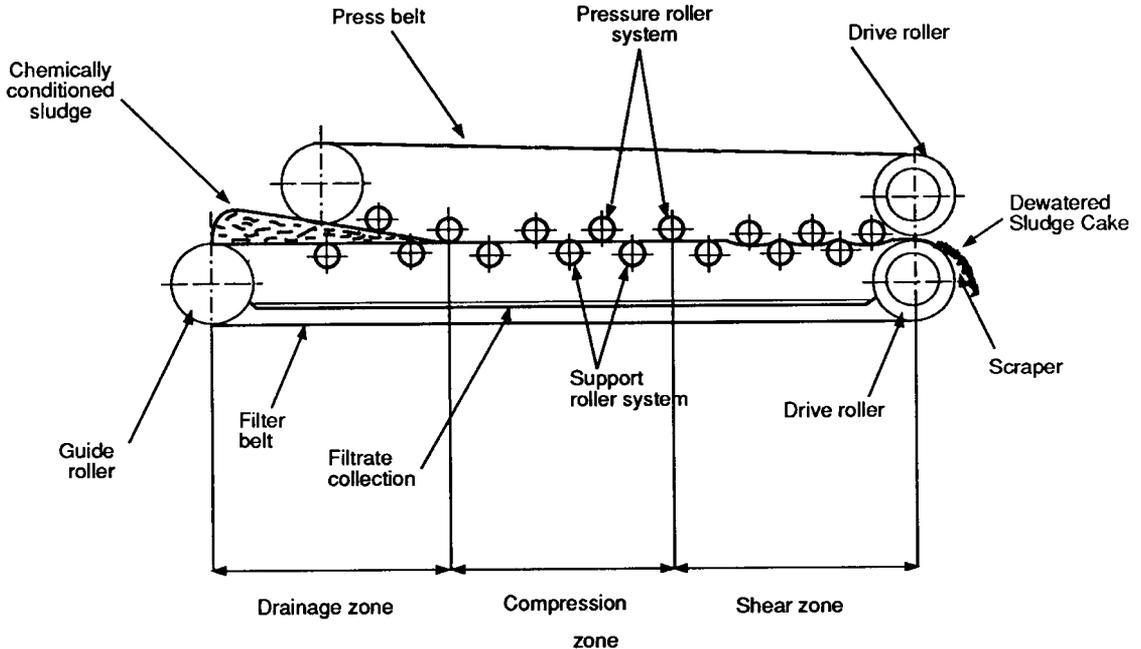


FIGURE 6.134 Belt filter press.

- Exhaust gas handling
- Dried solids handling

Because of the high capital and operating costs of heat treatment systems, the sludge moisture content should be reduced as much as possible by mechanical means before the drying process. The high costs also limit the application of heat treatment. An exception may be to dry sludge for use as a fertilizer supplement and soil conditioner, especially if the end product is marketed, and to dry sludge before an incineration process.

TABLE 6.133 Belt Filter Press Performance Data (88)

Type of sludge	Solids concentration, %
Undigested sludges	
Primary	28–40
Primary and trickling filter	24–32
Primary and activated sludge	20–30
Activated sludge	15–24
Digested sludges	
Primary and activated sludge	18–23
Heat-treated digested sludges	
Primary and activated sludge	38–50

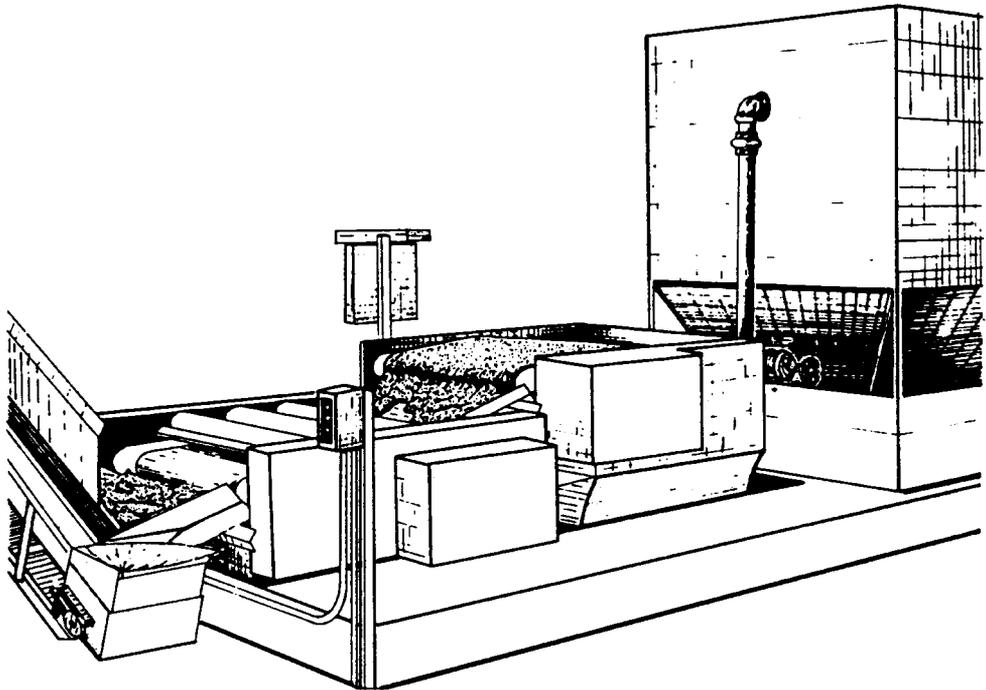


FIGURE 6.135 Moving screen concentrator (86).

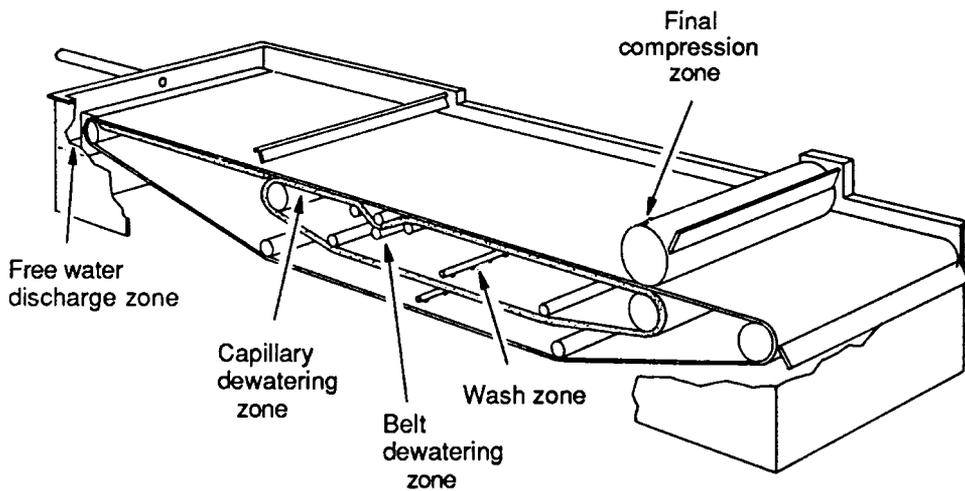


FIGURE 6.136 Capillary dewatering system (86).

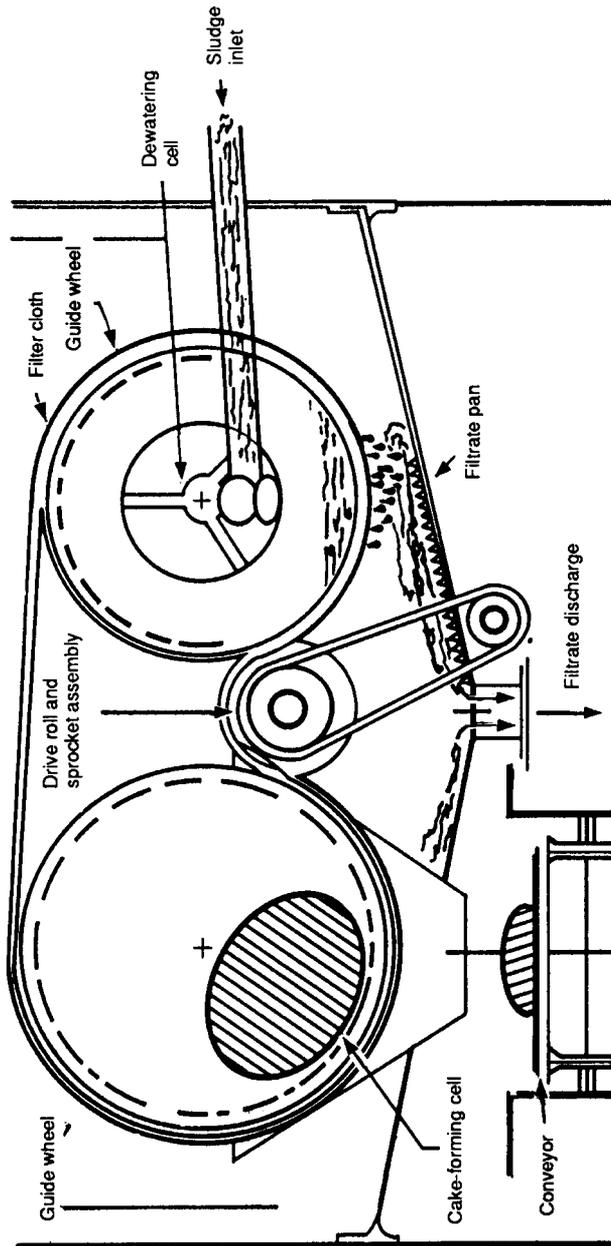


FIGURE 6.137 Rotating gravity concentrator (86).

The five types of dryers used for heat treatment are

1. Flash
2. Rotary
3. Toroidal
4. Multiple hearth
5. Spray

The most common system uses a flash dryer.

Flash Dryer: In the flash drying process, the sludge moisture is instantaneously vaporized by application of the sludge into a hot gas (air) stream. Initially, wet and dried sludges are blended to improve pneumatic conveyance. Hot gases at 1200 to 1400°F (650 to 760°C) and the blended sludge are mixed and conveyed into a cage mill, which mechanically agitates the sludge while the moisture vaporizes. Next, the dried sludge and spent gases are separated in a cyclone, with some sludge returned for mixing with the incoming wet sludge and the remainder sent to storage for further processing or final disposal.

Rotary Dryer: A rotary dryer consists of a slightly inclined cylinder that revolves at 5 to 8 r/min. Wet and dried sludges are blended before being fed to the dryer. Typically, flights or baffles along the length of the dryer are used to break up the blended sludge for more efficient drying.

Toroidal Dryer: Using the jet mill principle, a blend of wet and dried sludge is dried and classified simultaneously in a toroidal dryer. The blended sludge is fed into a doughnut-shaped dryer, containing heated gas (air) at 800 to 1100°F (425 to 600°C). As the sludge solids are dried, they are broken into fine particles that the gas stream carries out of the dryer.

Multiple-Hearth Dryers. By using burners at the top and bottom hearths and a downdraft of hot gases, a multiple-hearth furnace may be used to dry a blend of wet and dried sludge. The exit temperature from a multiple-hearth drier is about 100°F (38°C) for the solids and about 325°F (162°C) for the gases.

Spray Drying. In the spray drying system, wet sludge is sprayed in a tower through which there is a downdraft of hot gases. Another approach is to atomize (break into small particles) the sludge by use of a high-speed centrifugal bowl and then spray the sludge particles into a tower. Normally, the latter approach will be more efficient.

Incineration. Drying and reducing sludge volume and weight using thermal combustion is accomplished by incineration. Since incineration requires auxiliary fuel to obtain and maintain high temperatures and to evaporate the water contained in the incoming sludge, concentration techniques should be applied before incineration. The primary elements of sludge incineration are shown schematically in Figure 6.138.

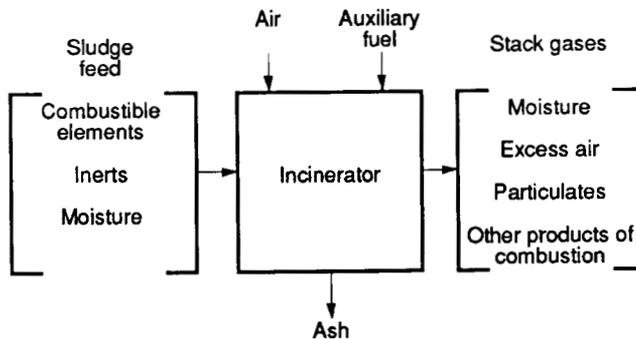


FIGURE 6.138 Elements of sludge incineration (86).

TABLE 6.133 Products of Complete Sludge Combustion (16)

Combustible	Stoichiometric oxygen required	Product	Heat value, Btu/lb (kJ/kg) of combustible, n
C	O ₂	CO ₂	14,600 (33,960)
2H ₂	O ₂	2H ₂ O	62,000 (144,000)
S	O ₂	2SO ₂	4050 (9420)
2CO	O ₂	2CO ₂	4400 (10,200)
CH ₄	2O ₂	CO ₂ + 2H ₂ O	23,900 (55,590)

The cost effectiveness of sludge incineration is significantly affected by the auxiliary fuel requirement, which is a function of the sludge composition (moisture and inert and combustible materials). The type of incinerator, operating requirements, air pollution control needs, site conditions, and residue disposal opportunities are other important considerations in the analysis of alternatives.

Sludge incineration is a two-step process involving drying and combustion after a preceding dewatering process, such as filters, drying beds, or centrifuges. The principal design considerations are fuel and air requirements. Also, residence time, temperature, and turbulence must be determined for a complete reaction.

Fuel for an incinerator will include the sludge and an auxiliary fuel, e.g., natural gas. The fuel's heating value is determined by its proportions of carbon, hydrogen, and sulfur. Table 6.134 shows the relative quantities of heat released by complete sludge combustion. Using these data and knowing the combustible sludge fractions, the heating value may be approximated as follows (16):

$$Q = 33,960C + 144,000(H - O/8) + 9420S$$

where Q = total heating value, kJ/kg

C = carbon, %

H = hydrogen, %

O = oxygen, %

S = sulphur, %

The best and most accurate method to determine sludge heating value is by use of a bomb calorimeter. Typical heating values of municipal wastewater sludges, as determined by bomb calorimetry, are shown in Table 6.135. Moreover, characteristic heating values of some auxiliary fuels is presented in Table 6.135.

Sensible and latent heat are required for incineration. The sensible heat is that heat gained or lost as reflected by a temperature change from a reference point. It equals the materials mean specific heat, Btu/lb/°F (kJ/kg/°C), times the mass of the material, lb (kg), times the change in temperature, °F (°C). Latent heat is

TABLE 6.135 Typical Heating Values of Municipal Sludges (16)

Type of sludge	Heating Value, Btu/lb (kJ/kg) of dry solids
Raw primary, chemically precipitated	7000 (16,300)
Raw primary	10,000–12,500 (23,300–29,100)
Anaerobically digested primary	5500 (12,800)
Activated	8500–10,000 (19,800–23,300)
Biological filter	8500–10,000 (19,800–23,300)

TABLE 6.136 Characteristic Heating Value of Auxiliary Fuels (16)

Fuel	Heating value, Btu/lb (kJ/kg) of fuel
Natural gas	22,800 (53,000)
Gasoline	20,700 (48,100)
Fuel oil	18,500 (43,000)
Crude oil	19,500 (45,400)
Anthracite coal	12,680 (29,500)

required for a change in state, which in incineration is the evaporation of moisture. In addition, operating heat losses and heat recovery must be considered. The desired result is to obtain equilibrium between the sludge heating value and the heat losses at a combustion temperature. This is called autogenous combustion.

The importance of providing a low and relatively constant sludge moisture content must be emphasized. The heat required for incinerator start-up is significantly impacted by moisture content. Also, since some 1800 to 2500 Btu (4 to 5 MJ) are required to evaporate 2 lb (1 kg) of water, autogenous combustion is sensitive to a fluctuating sludge moisture content.

Since air is commonly used to provide the oxygen required for combustion, it is noted that the theoretical quantity of air required will be 4.35 times greater than the quantity of oxygen required; air is composed of 23% oxygen by weight. Depending on the incinerator used and sludge characteristics, 20 to 100% excess air may be required to ensure effective operation.

Discussions and design information follow for multiple-hearth, fluidized-bed, and cyclonic incinerators. Manufacturers should be consulted as part of the preliminary engineering for application of any incinerator at a specific site.

Multiple-Hearth Incinerators. As illustrated in Figure 6.139, a multiple-hearth incinerator is a vertical cylindrical reactor containing a number of horizontal hearths. Supported by a single central rotating shaft, rabble arms rake the sludge radially across the hearths. As the sludge moves downward through the incinerator, there is an upward flow of combustion air. Typical design information for multiple-hearth incinerators appears in Table 6.137. In addition, Table 6.138 gives ranges of wet sludge loading rates for various types of sludges (assumed to have had no dewatering chemical additions); lower rates are applicable to smaller plants, and higher rates to larger plants.

The incineration process occurs in three zones of the multiple hearth incinerator. The name, location, and the associated sludge and air temperatures for each zone are presented in Table 6.139.

The most widely used sludge incinerator is the multiple-hearth type. Its advantages are simplicity, ease of control, durability, and flexibility of operation. The reactor will combust a wide variety of materials and can accommodate fluctuations in feed rate.

Fluidized-Bed Incinerators. Another popular incineration process uses a fluidized-bed incinerator, as illustrated in Figure 6.140. The vertical reactor is cylindrically shaped and contains a sand bed and fluidizing air distributor. The sludge feed is either introduced above or directly (most common) into the sand bed. Table 6.140 contains typical design information for a fluidized-bed incinerator. Typical wet sludge loading rates for a variety of sludges (assumed to have had no dewatering chemical additions) are shown in Table 6.141.

By providing good mixing of the sludge with the combustion air, the fluidized-bed incinerator provides for simultaneous drying and combustion. Also, the operation is relatively more reliable since the reactor has no moving parts.

The fluidized-bed incinerator requires less supplemental fuel than a multiple-hearth incinerator. Fuel costs may be reduced by providing an air preheater. Also, cooling tubes can be submerged in the bed for heat recovery. Some of the fuel cost savings are offset by the need for air pollution control devices.

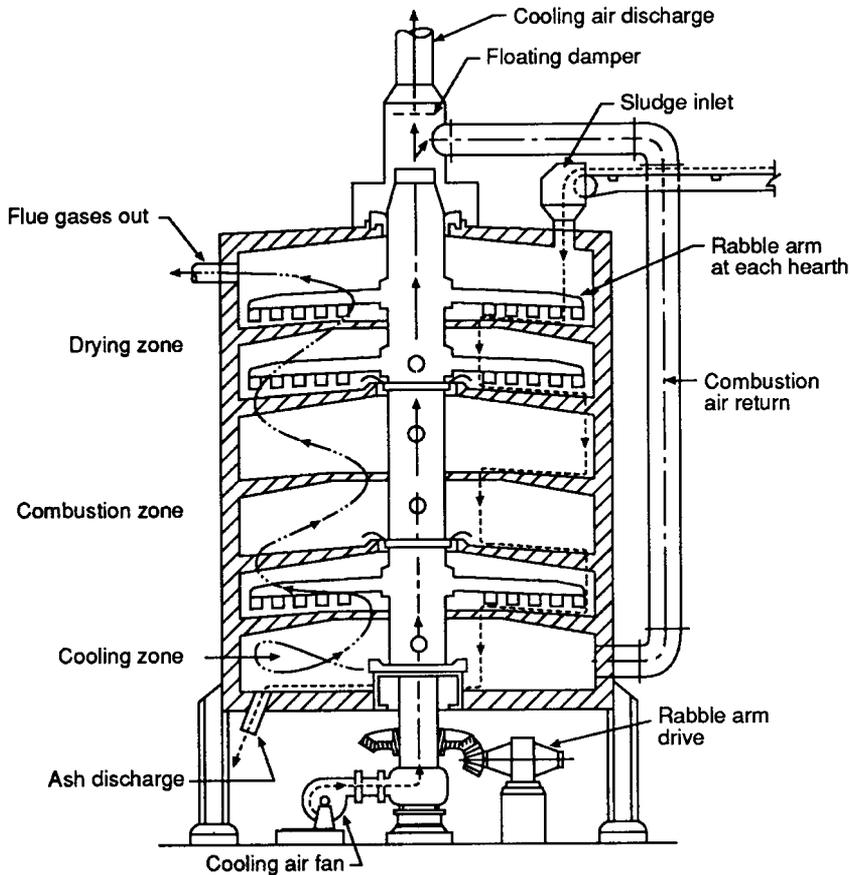


FIGURE 6.139 Multiple-hearth incinerator (86).

TABLE 6.137 Multiple-Hearth Incinerator Design Data (16, 50)

Parameter	Typical value
Diameter	9.25–25.75 ft (2.8–84.5 m)
Hearths	4–12 maximum
Burning zone temperature	
Normal	1500°F (815°C)
Maximum	1700°F (925°C)
Hearth loading rates	6–12 lb/ft ² /h (30–60 kg/m ² /h)
Combustion airflow	12–13 lb/lb (12–13 kg/kg) dry solids
Shaft cooling airflow	1/3–1/2 of combustion airflow
Excess air	75–100%
Ash density	
Dry	0.35 lb/ft ³ (5.6 kg/m ³)
Wet	55 lb/ft ³ (880 kg/m ³)
Sludge mass reduction	20–25%

TABLE 6.138 Multiple-Hearth Incinerator Loading Rates (90)

Type of sludge	Solids, %	Volatile solids, %	Loading rate, lb/ft ² /h (kg/m ² /h)
Primary	30	60	7–12(35–60)
Primary + 20 mg/L of ferric chloride	16	47	6–10(30–50)
Primary + 298 mg/L of low lime	35	45	8–12(40–60)
Primary + waste activated	16	69	6–10(30–50)
Primary + (waste activated + 20 mg/L of FeCl ₃)	20	54	6.5–11(32–55)
(Primary + 20 mg/L of FeCl ₃) + waste activated	16	53	6–10(30–50)
Waste activated	16	80	6–10(30–50)
Waste activated + 20 mg/L of FeCl ₃	16	50	6–10(30–50)
Digested primary	30	43	7–12(35–60)

Cyclonic Incinerators. For use in smaller wastewater treatment plants, cyclonic incinerators, which are available as package units, may be a feasible drying system. The cyclonic reactors introduce combustion air tangentially into a cylindrical combustion chamber, and the sludge is sprayed radially toward the heated walls of the chamber. Combustion occurs rapidly enough that the sludge does not adhere to the walls. The cyclonic incinerator may also be a viable incinerator alternative for large sludge-drying requirements.

Pyrolysis. A controlled combustion process alternative is pyrolysis, which is a process wherein organic solids are decomposed at high temperatures in an oxygen-deficient atmosphere. Pyrolysis systems are in the development stage but offer the advantages of eliminating air pollution problems and producing more useful by-products than conventional incineration processes. Some manufacturers have systems that are currently available.

The three products of pyrolysis are gas (mostly methane), liquid (oils), and solids (char). By controlling temperature, pressure, and residence time in the reactor, the composition and combinations of these products may be varied. In a subsequent process, each can be recovered as a by-product or decomposed by combustion in the presence of oxygen. The typical pyrolysis system will include a multiple-hearth furnace and an afterburner for combustion of off gases and odor control.

Starved Air Combustion. In order to achieve some of the benefits from pyrolysis, starved air combustion of sludge utilizes equipment and process flows similar to typical incineration systems except that less air (or pure oxygen) is supplied than is required for complete combustion. The char is more or less residual carbon, depending on the quantity of air supply. The use of less air also increases the process's thermal efficiency over that of incineration, and less costs are required for off-gas handling.

TABLE 6.139 Multiple-Hearth Incineration Zones (16)

Zone	Location	Typical temperature, °F (°C)	
		Sludge	Air
Drying	Upper hearths	160 (70)	800 (425)
Burning	Center hearths	1500 (815)	1500 (815)
Cooling	Lower hearths	400 (205)	350 (175)

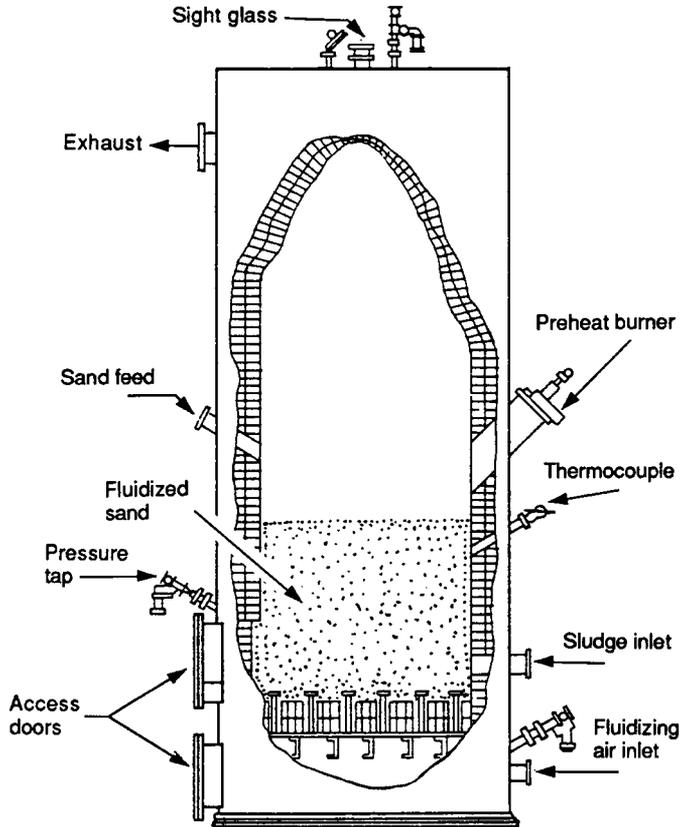


FIGURE 6.140 Fluidized-bed incinerator (86).

TABLE 6.140 Fluidized-Bed Incinerator Design Data (16, 50)

Parameter	Typical value
Diameter	9–50 ft (2.7–15 m)
Height	20–30 ft (6–10 m)
Temperature	
Optional preheat	1000°F (540°C)
Operating	Normal: 1500°F (815°C) Range: 1400–2200°F (760–1200°C)
Air pressure	3–5 lb/in ² (20,700–34,500 N/m ²)
Excess air	20–40%
Bed loading rate	50–60 lb/ft ² /h (245–290 kg/m ² /h)
Superficial bed velocity	2–3.5 ft/s (0.6–1.1 m/s)
Bed expansion	80–100%
Sand effective size	0.2–0.3 mm
Sand uniformity coefficient	1.8
Sand loss	5% bed volume per 300 h of operation
Sludge mass reduction	25–35%

TABLE 6.141 Fluidized-Bed Incinerator Loading Rates (90)

Type of sludge	Solids, %	Volatile solids, %	Loading Rate, lb/ft ² /h (kg/m ² /h)
Primary	30	60	14 (70)
Primary ± 20 mg/L of ferric chloride	16	47	6.8 (34)
Primary + 298 mg/L of low lime	35	45	18 (90)
Primary + waste activated	16	69	6.8 (34)
Primary + (waste activated + 20 mg/L of FeCl ₃)	20	54	8.4 (42)
(Primary + 20 mg/L of FeCl ₃) + waste activated	16	53	6.8 (34)
Waste activated	16	80	6.8 (34)
Waste activated + 20 mg/L of FeCl ₃	16	50	6.8 (34)
Digested primary	30	43	14 (70)

Starved air combustion reduces sludge volume and provides a sterilized residue. Also, it produces a heat-valued off-gas, which is suitable for combustion in an afterburner or boiler for fuel in another furnace.

Typically, a multiple-hearth furnace is used in one of three modes utilizing a range of temperatures and providing different residue characteristics. These modes are (50)

1. Low-temperature char, which produces a charcoallike residue with a high ash content
2. High-temperature char, which produces a charcoallike residue converted to fixed carbon and ash
3. Char burned, which reacts away all carbon and produces ash as a residue

For sludges containing 22% solids, the loading rate per hearth may range from 9 to 15 lb/ft²/h (45 to 75 kg/m²/h).

Wet-Air Oxidation. The wet-air oxidation process uses high water temperatures and pressures to oxidize volatile sludge solids. The pressure requirements may vary from 150 to 3000 lb/in² (0.1 to 20 N/m²) but typically range from 1000 to 1750 lb/in² (6.7 to 11.7 N/m²) depending on the degree of oxidation desired. Under the selected operating pressure, liquid water temperatures may vary between 250 to 700°F (120 to 370°C).

Also known as the Zimpro process, the wet-air oxidation process receives limited application for conventional sludge treatment due to the relatively high system costs. However, the system does have the advantage of being effective in treating concentrated wastewater streams or sludges with low (1%) solids content. Preliminary drying and dewatering are not required for this process but are necessary for conventional combustion processes.

Residual Disposal

The design of sludge-handling systems must include a method for final disposal of sludge residue. Due to restrictions on ocean dumping, final disposal generally involves some type of application to the land and often requires sludge to be transported away from the wastewater treatment plant site. The method of residual disposal is dependent on the sludge characteristics, local conditions, and government regulations.

The most common methods of residual disposal are burial at a sanitary landfill, application on agricultural land, and utilization for land reclamation. Other disposal methods include chemical fixation and deep-well injection.

Depending on the volume of residual sludge and the distance to the disposal site, the transportation considerations and costs can be quite significant. Liquid sludge may be conveyed by pipeline, truck, barge, or railroad. Trucks and railroad cars are used to transport dewatered sludges. In almost all cases, only pipelines or trucks can provide direct source to final disposal site transportation. Because of their destination flexibility, relative small capital cost, and ability to carry liquid and dewatered sludge, trucks are used most widely. Table 6.141 provides a checklist for the various modes of sludge transport.

Storage is also an important design consideration and cost factor. When mechanical systems are involved, adequate provisions must be made to store or provide alternate processing during maintenance or unplanned downtime. For the land application approaches, the affects of climate and site accessibility must also be part of the design considerations.

Use and Disposal of Sludge. Under the Clean Water Act of 1987, the U.S. Congress directed the Environmental Protection Agency (EPA) to promulgate technical standards for the use and disposal of sewage sludge and placed emphasis on identifying toxic pollutants in sludge that may adversely affect public health and the environment. The resulting regulation was 50CFR503, which defines specific limits for 12 contaminants (arsenic, beryllium, cadmium, chromium, copper, lead, mercury, molybdenum, nickel, selenium, zinc, and total hydrocarbons).

The municipal systems affected are those that are required to have an approved industrial waste pretreatment program or have been designated Class 1 by EPA; have a design flow greater than one mgd; or serve 10,000 or more people. Individuals who market and use processed wastewater sludge are covered by the regulation as well. Furthermore, states are required to adopt standards that are at least as stringent as the federal regulations.

Sludge use and disposal are consolidated into three methods: land application, surface disposal, and incineration. Specific use and disposal methods, such as agricultural use, site reclamation, and landfilling, are arranged to fit within these categories. Monitoring and recordkeeping requirements are contained within each part of the regulation.

The 40 CFR 503 regulation consists of five major subparts dealing with general provisions, land application, surface disposal, pathogen and vector attraction reduction, and incineration. The land application, surface disposal, and incineration subparts describe the pollutant limits, management practices, operational standards, and monitoring and record-keeping conventions specific to each practice. The subpart on pathogen and vector attraction reduction separately discusses definitions and practices considered capable of achieving pathogen reduction and decreasing the vector attraction potential of treated sludge. Some specific requirements imposed by each subpart are:

Subpart A—General Provisions. Treatment facilities will have one year to comply with the standard unless construction is required, in which case the deadline is two years.

Subpart B—Land Application. This subpart includes distribution and marketing, agricultural application, and nonagricultural application of sludge and sludge products.

- “Clean” sludge has regulated pollutant concentrations below regulation limits and may be used for unrecorded land application. Other sludges can be land applied as long as the application is recorded.
- Land used for sludge application must be certified not to have exceeded cumulative pollutant loading limits defined in the regulation.
- Application to agricultural and nonagricultural land must not exceed the agronomic rate at which the soil and vegetation can absorb sludge constituents.
- Sludge cannot be applied to flooded, frozen, or snow-covered land if runoff might contaminate surface waters.
- Distributed and marketed sludges must meet Class A pathogen and vector attraction reduction requirements before distribution.

TABLE 6.142 Sludge Transport Checklist (90)

-
1. Sludge characteristics
 - a. Quantity: maximum, average, minimum
 - b. Solids contents
 - c. Odor potential
 2. Pipeline
 - a. Alternate routes considered
 - b. Distance
 - c. Elevation changes (air relief, vacuum relief)
 - d. Operating program
 - e. Pipe selection (velocity, maximum and minimum size, material, friction losses, corrosion control)
 - f. Pumping facilities (number and location of pumping stations, number and type of pumps, pumping energy, station structure, station utilities, controls)
 - g. Pipeline cleaning provisions
 - h. Emergency operation (storage, standby power)
 - i. Excavation conditions verified (soil conditions, other underground utilities, highways, rail crossings)
 - j. Methods of right-of-way acquisition (existing utility easements, negotiation with landowners, condemnation)
 3. Trucks
 - a. Alternate routes considered
 - b. Haul distance, speed, and travel time
 - c. Compatibility of proposed route with existing road and traffic [weight and height limits, turns required, speed limits, traffic congestion, road width, elevation changes, traffic control (stops, etc.)]
 - d. Operating schedule (loading time, haul time, unloading time, return time fueling, and daily maintenance)
 - e. Fuel consumption
 - f. Labor requirements
 - g. Truck selection and acquisition
 - h. Contract haul considered
 - i. Facilities required (loading, unloading, vehicle maintenance, fueling, parking, washdown, sludge storage)
 4. Barge
 - a. Haul distance
 - b. Barge speed and travel time (traffic, drawbridges, locks, tides, currents, and height limitations)
 - c. Operating schedule (loading time, haul time, unloading time, return time)
 - d. Barge selection, acquisition, and/or tow boat acquisition (towed or self-propelled, type, size, quantity, useful life, purchase, lease)
 - e. Contract haul considered (towing or complete operation)
 - f. Manpower requirements
 - g. Fuel consumption
 - h. Facilities required (loading, unloading, barge and tow-boat maintenance, fueling, docking, washdown, and sludge storage)
 5. Railroad
 - a. Distance
 - b. Speed, load-limiting factors, and travel time (clearance limitations, track conditions, traffic schedule)
 - c. Operating schedule (loading, haul, unloading, and return times)
 - d. Car selection and acquisition (type, size, quantity, useful life, purchase, lease, or railroad-owned)
 - e. Fuel consumption
 - f. Labor requirements
 - g. Facilities required (loading, unloading, tank car maintenance, storage, and cleaning, sludge storage, siding extensions)
 6. Environmental impact (air, land, surface water, groundwater, social, health, economic, historical, and archaeological impacts, and environmentally sensitive areas)
 7. Regulations and standards
 - a. Surface water protection
 - b. Sludge loading and unloading
 - c. Construction within navigable waterways
 - d. Building codes
 - e. Reporting spills
 - f. Permits
-

- Packaged sludge must be labeled with the name of the preparer and contain application restriction information.
- Sludge-derived products may be blended with other materials to meet pollutant limits.

Subpart C—Surface Disposal. This subpart includes dedicated nonagricultural application and monofills as well as storage sites for holding sludge for more than two years.

- Sludge pollutant limits depend on whether the disposal area is lined and has a leachate collection system and the distance between the sludge application and site boundary.
- Sites must be managed to prevent groundwater and other environmental contamination, to restrict public access, and to prohibit grazing of food or feed crops unless public health is protected.
- Some pollutant limits may be exceeded on a case-by-case basis if approved by the permitting authority.
- Groundwater quality must be monitored.

Subpart D—Pathogen and Vector Attraction Reduction. Alternatives that EPA will accept for pathogen reduction and to avoid attraction of disease carrying vectors are listed. Class A and Class B pathogen reduction standards are defined.

- Class A Sludges have fecal coliform densities less than 1000 per gram of total solids or less than 3 *Salmonella*, 1 virus, or 1 helminth egg per 4 grams of total solids.
- Class B Sludges fecal coliform have densities less than 2 million per gram as determined by the geometric mean of at least seven samples.
- Both classes must either be shown to achieve the above standards by testing or they must utilize processes that have been proven capable of achieving the standard.
- Vector attraction reduction must employ one of eleven methods approved in the regulation for sludge and one method for septage.
- Processes to significantly reduce pathogens (PSRPs) and processes to further reduce pathogens (PFRPs) are included in the options.

Subpart E—Incineration. These requirements are based on performance testing at each facility. Feed rates are based on all incinerators at a facility.

- The total hydrocarbon (THC) limit for wastewater sludge to be incinerated is 100 ppm.
- Beryllium and mercury content must comply with National Emission Standards in 40 CFR 61.
- Arsenic, cadmium, chromium, lead, and nickel limits in feed sludge are to be determined by site-specific factors.
- Continuous monitoring of moisture, oxygen, temperature, and THC is required.

Monitoring, recordkeeping, and reporting are specific to each disposal method. Common requirements include:

- Monitoring must begin 120 days after the regulation is effective.
- Records must be kept for five years.
- Annual reports must be submitted to the permitting authority.

Sludge disposal at municipal solid waste landfills remains regulated under 40 CFR 60 and 258. Sludge that is hazardous will be regulated under RCRA regulations, 40 CFR 261 to 268, and sludge containing more than 50 ppm of PCBs must comply with TSCA regulations, 40 CFR 761.

Landfill. The application of sludge at a sanitary landfill receiving refuse or at a dedicated landfill is the most widely practiced technique for residual sludge disposal. Table 6.143 provides a listing of sludge landfill design considerations. Conventional sanitary landfills, trench fill, area fill, and diked containment are discussed in this section.

An assessment of landfilling suitability of sludges from different treatment processes is shown in Table 6.144. Similarly, the suitability of local soils must be considered; Table 6.145 presents pertinent soil characteristics. In addition, pH and cation exchange capacity (CEC) are important with respect to the disposition of heavy metals; this is addressed under the section on sludge application to agricultural lands.

Monitoring of groundwater is an integral part of any sludge landfilling operation. Monitoring wells are used to establish baseline groundwater quality, to provide surveillance during the landfilling operation, and to monitor the site after closure. A typical monitoring well is illustrated in Figure 6.141. If sampling at several depths is desired, a cluster of wells is required, as shown in Figure 6.142.

Sanitary Landfill. Sludge can be transported to a sanitary landfill either alone or in combination with other refuse for disposal. Local as well as state and federal regulations must be considered in design of such systems. Proper management requires precautions to minimize leachate and runoff from landfills in order to prevent contamination of ground and surface waters. When fill is completed on portions of the site, grassing or other means should be used to prevent erosion.

At the sanitary landfill, stabilized or unstabilized sludge may be mixed with refuse, and thus become part of the daily operation. In this case, close cooperation between the user and operator is very important. Local requirements will define the points of scheduling and cooperation necessary for an effective operations plan.

An alternate sludge disposal technique is to mix stabilized sludge with soil and apply the mixture as the interim or final cover over completed areas of the landfill. The advantages of this approach are that not only is the sludge disposed of but its fertilizer characteristics promote the development of a vegetation cover over the completed fill areas.

Typical design data for sludge–refuse and sludge–soil mixture operations at sanitary landfills is shown in Table 6.146. With each approach, care should be taken to minimize development of odor or other nuisance conditions.

Trench Fill. Sludge trenching is a landfill process where stabilized or unstabilized sludge is placed in an excavated trench and covered with soil. Leachate control methods may include membrane or impervious soil liners or a leachate collection and treatment system.

Trench fill operations are widely used and are categorized as narrow when the trench is less than 3 m (10 ft) in width and as wide when the width is greater than 3 m (10 ft). Typical design data and equipment requirements for both types of operations are presented in Table 6.147.

Sludge disposal in a narrow trench involves the application of a single layer of sludge followed by a layer of cover soil. With a trench width of 2 to 3 ft (0.6 to 1 m), sludge with a relatively low solids content (15 to 20%) may be applied; at this narrow width, the cover soil will bridge the sludge. Disadvantages of the narrow trench fill are that it is land-intensive, has lower application rates, and is rarely practical for installation of liners.

Sludges with a higher solids content are needed for wider trenches in order to support the cover soil. In a wide-trench sludge disposal system, the equipment typically operates inside the (excavated) trench. When a long, continuous trench is used, dikes are built to confine the sludge to specific areas. Sludge solids contents in excess of 20% are needed for use of this disposal method.

For both narrow and wide trench fill systems, utilization rates of the trench and land must be calculated. Also, the sludge application rate and site life are important design considerations. Table 6.148 illustrates the data required for these calculations.

Area Fill. The sludge disposal operation of placing sludge on the ground and covering it with soil is termed area fill. A bulking agent, usually soil, is used to absorb moisture and improve workability of the sludge and to achieve stability and bearing capacity of the fill layers. Provisions are required to divert surface drainage, and a liner may be necessary for groundwater protection.

TABLE 6.143 Sludge Landfilling Checklist (90)

-
1. Sludge characteristics
 - a. Type of processing at treatment plant
 - b. Quantity: maximum, average, minimum
 - c. Analysis (concentration, particle characteristics, volatile content, nitrogen, inorganic ions, pathogen content, toxic organic compounds, pH)
 2. Regulations and standards (sludge stabilization, sludge loading rates, frequency and depth of cover, distances to road, residences and surface water, monitoring, roads, building codes, permits)
 3. Site characteristics
 - a. Site plan
 - b. Soils (depth, texture, structure, bulk density, porosity, permeability, moisture, ease of excavation, stability, pH, cation exchange capacity)
 - c. Geohydrology (groundwater: location, extent, use, geologic formations, discontinuities, surface outcrops)
 - d. Climate (precipitation, evapotranspiration, temperatures, number of freezing days, wind direction)
 - e. Land use (present, final, adjacent property)
 4. Landfill type
 - a. Sludge-only trench fill
 - (1) Narrow trench
 - (2) Wide trench
 - b. Sludge-only area fill
 - (1) Mound
 - (2) Layer
 - (3) Diked contaminant
 - c. Codisposal with refuse
 - (1) Sludge-refuse mixture
 - (2) Sludge-soil mixture
 5. Landfill designs
 - a. Trench or area dimensions
 - (1) Length
 - (2) Width
 - (3) Depth
 - b. Berm dimensions
 - c. Trench or area spacing
 - d. Sludge depth
 - e. Intermediate cover depth
 - f. Final cover depth
 - g. Bulking agent
 - h. Bulking ratio
 - i. Soil importation
 6. Facilities
 - a. Leachate controls (adequate surface drainage, natural attenuation, soil liners, membrane liners, collection and treatment)
 - b. Gas control (permeable methods, impermeable methods, gas extraction)
 - c. Roads
 - d. Soil stockpiles
 - e. Inclement weather areas
 - f. Minor facilities (structures, utilities, fencing, lighting, washbacks, monitoring wells, landscaping, equipment fueling, storage and maintenance)
 7. Landfill equipment (excavation, sludge handling, backfilling, grading, road construction)
 8. Labor requirements
 9. Flexibility and reliability (expansion potential, modification for change in sludge volume or type, equipment failure locally, storage)
 10. Environmental impacts (air, land, surface water, groundwater, social, health, economic, historical, and archeological impacts, and environmentally sensitive areas)
-

TABLE 6.144 Suitability of Sludges for Landfilling (92)

Process	Feed	Sludge-only landfilling		Codisposal landfilling	
		Suitability	Reason	Suitability	Reason
Thickening					
Gravity	Primary	NS	OD, OP	NS	OD, OP
	WAS	NS	OD, OP	NS	OD, OP
	Primary and WAS	NS	OD, OP	NS	OD, OP
	Digested primary	NS	OD	MS	OD
	Digested primary and WAS	NS	OD	MS	OD
Flotation	Primary and WAS	NS	OD, OP	NS	OD, OP
	WAS with chemicals	NS	OD	NS	OD, OP
	WAS without chemicals	NS	OD, OP	NS	OD, OP
Treatment					
Aerobic digestion	Primary, thickened	NS	OD, OP	NS	OD, OP
	Primary and WAS, thickened	NS	OP	MS	OP
Anaerobic digestion	Primary, thickened	NS	OP	MS	OP
	Primary and WAS thickened	NS	OP	MS	OP
Incineration	Primary, dewatered	S	—	S	—
	Primary and WAS, dewatered	S	—	S	—
Wet oxidation	Primary or primary and WAS	NS	OD, OP	MS	OD, OP
Heat	Any, thickened	NS	OD, OP	MS	OD, OP
Lime stabilization	Primary, thickened	NS	OP	MS	OP
	Primary and WAS, thickened	NS	OP	MS	OP
Dewatering					
Drying beds	Any, digested	S	—	S	—
	Any, lime stabilization	S	—	S	—
Vacuum filter	Primary, lime conditioned	S	—	S	—
	Digested, lime conditioned	S	—	S	—
Pressure filtration	Digested, lime conditioned	S	—	S	—
Centrifugation	Digested	S	—	S	—
	Digested, lime conditioned	S	—	S	—
Heat drying	Digested	S	—	S	—

WAS = waste activated sludge; NS = not suitable; MS = marginally suitable; S = suitable; OD = odor problems; OP = operational problems.

The two types of area fill operations are called area fill mound and area fill layer. Typical design data and equipment needs for these relatively new methods of sludge landfilling are presented in Table 6.149.

In the area fill mound, stabilized sludge is mixed with a bulking agent and stacked in mounds about 6 ft (2 m) high; cover material is added between applications. After all the sludge applications, a final soil cover is made.

The area fill layer method involves spreading the stabilized or unstabilized sludge and bulking agent mixture in layers of 0.5 to 3 ft (0.15 to 1 m) thickness. An interim soil cover should be applied between layers, and a final cover is required after layering is completed at the site.

In both methods, the ratio of sludge to bulking agent and the thickness of cover soil is dependent on the solids content of the sludge and the general characteristics of the sludge and local soil.

Diked Containment. Stabilized and unstabilized sludges may also be applied to the land surface using the technique known as diked containment. Either landbased or sludge-based equipment is used. Typical design data for diked containment are presented in Table 6.150.

TABLE 6.145 Unified Soil Classification System and Characteristics Pertinent to Sludge Landfills (93).

Major Divisions	Symbol		Name	Potential Frost Action	Drainage Characteristics*	Value for Embankments	Permeability, cm/s	Compaction Characteristics†	SM AASHO Max Unit (Dry Weight, %)	Requirements for Seepage Control
	Letter	Hatching								
Gravel and Gravelly Soils	GW		Well-graded gravels or gravel-sand mixtures, little or no fines	None to very slight	Excellent	Very stable, pervious shells of dikes and dams	$k > 10^{-2}$	Good, tractor, rubber-tired steel-wheeled roller	125-135	Positive cutoff
	GP		Poorly graded gravels or gravel-sand mixtures, little or no fines	None to very slight	Excellent	Reasonably stable, pervious shells of dikes and dams	$k > 10^{-2}$	Good, tractor, rubber-tired steel-wheeled roller	115-125	Positive cutoff
	GM		Silty gravels, gravel-sand-silt mixtures	Slight to medium	Fair to poor, no practically impervious	Reasonably stable, not particularly suited to shells, but may be used for impervious cores or blankets	$k = 10^{-3}$ to 10^{-4}	Good, with close control, rubber-tired, sheepfoot roller	120-135	Toe trench to none
	GC		Clayey gravels, gravel-sand-clay mixtures	Slight to medium	Poor to practically impervious	Fairly stable, may be used for impervious cores	$k = 10^{-6}$ to 10^{-8}	Fair, rubber-tired, sheepfoot roller	115-130	None
Sand and Sandy Soils	SW		Well-graded sands or gravelly sands, little or no fines	None to very slight	Excellent	Very stable, pervious sections; slope protection required	$k > 10^{-3}$	Good, tractor	110-130	Upsream blanket and toe drainage or wells
	SP		Poorly graded sands or gravelly sands, little or no fines	None to very slight	Excellent	Reasonably stable, may be used in dike section with flat slopes	$k > 10^{-3}$	Good, tractor	100-120	Upsream blanket and toe drainage or wells
	SM		Silty sands, sand-silt mixtures	Slight to high	Fair to poor, no practically impervious	Fairly stable, not particularly suited to shells, but may be used for impervious cores or dikes	$k = 10^{-3}$ to 10^{-4}	Good, with close control, rubber-tired, sheepfoot roller	110-125	Upsream blanket and toe drainage or wells
	SC		Clayey sands, sand-clay mixtures	Slight to high	Poor to practically impervious	Fairly stable, use for impervious cores for flood control structures	$k = 10^{-6}$ to 10^{-8}	Fair, sheepfoot roller, rubber-tired	105-125	None
Silty and Clayey Soils	ML		Inorganic silts and very fine sands or rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Medium to very high	Fair to poor	Poor stability, may be used for embankments with proper control	$k = 10^{-3}$ to 10^{-4}	Good to poor, close control essential, rubber-tired roller, sheepfoot roller	95-120	Toe trench to none
	CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Medium to high	Practically impervious	Stable, impervious cores and blankets	$k = 10^{-6}$ to 10^{-8}	Fair to good, sheepfoot roller, rubber-tired	95-120	None
	OL		Organic silts and organic silts of low plasticity	Medium to high	Poor	Not suitable for embankments	$k = 10^{-4}$ to 10^{-6}	Fair to poor, sheepfoot roller	80-100	None
	MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Medium to very high	Fair to poor	Poor stability, core of hydraulic dam not desirable in rolled fill construction	$k = 10^{-4}$ to 10^{-6}	Poor to very poor, sheepfoot roller	70-95	None
Fine-Grained Soils	CH		Inorganic clays of high plasticity, fat clays	Medium	Practically impervious	Fair stability with flat slopes, thin cores, blankets and dike sections	$k = 10^{-4}$ to 10^{-6}	Fair to poor, sheepfoot roller	75-105	None
	OH		Organic clays of medium to high plasticity, organic silts	Medium	Practically impervious	Not suitable for embankments	$k = 10^{-4}$ to 10^{-6}	Poor to very poor, sheepfoot roller	65-100	None
	Pt		Peat and other highly organic soils							

NOT RECOMMENDED FOR SANITARY LANDFILL CONSTRUCTION

*Values are for guidance only; design should be based on test results.

†The equipment listed will usually produce the desired densities after a reasonable number of passes when moisture conditions and thickness of lift are properly controlled.

Compacted soil at optimum moisture content for Standard AASHO (Standard Proctor) compactive effort.

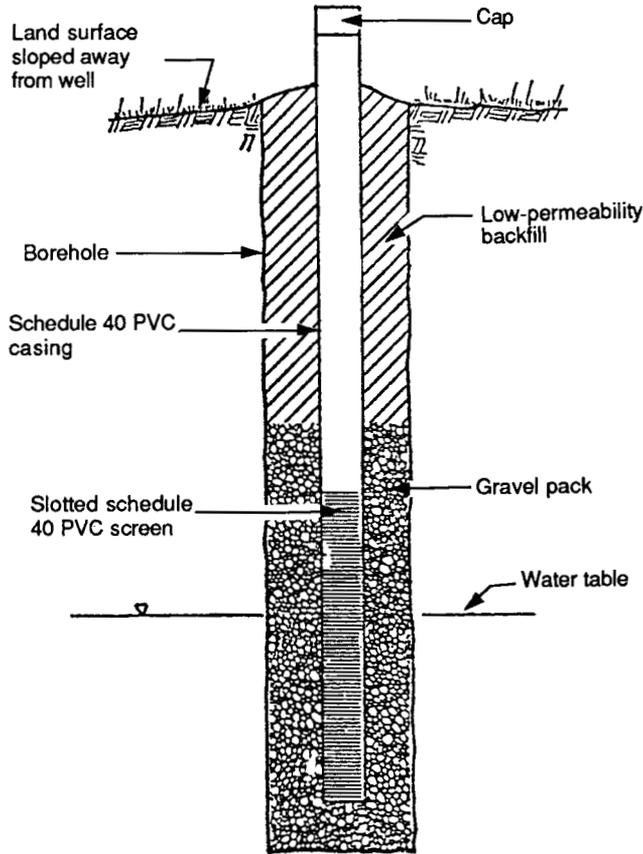


FIGURE 6.141 Typical monitoring well for a single vertical interval (92).

Containment dikes are provided during development of the landfill site. The dikes provide protection from surface drainage and access to the work area. In some cases, soil bulking may be used for dike construction. Typical dimensions are 50 to 100 ft (15 to 30 m) wide, 100 to 200 ft (30 to 60 m) long, and 10 to 30 ft (3 to 10 m) deep (50). However, site-specific conditions must be considered.

Land Application. Applying sludge to the land is a popular residual disposal method because of the relatively simple operating requirements and low operating costs if suitable land is nearby. Also, since it contains considerable quantities of organic matter and essential plant nutrients, sludge is an excellent soil conditioner for agricultural lands and for reclaiming disturbed lands.

Depending on the characteristics of the soil and the sludge, the soil will filter, buffer, sorb, and/or chemically and biologically react with the sludge. The preliminary engineering phase must consider these reactions as well as potential of developing nuisance conditions, vector problems, and/or contaminating groundwater. Liquid, dewatered, or dried sludge may be applied to the land using tank truck, subsurface injection, ridge and furrow spreading, spray irrigation, etc. methods. Most surface applications require that the sludge be mixed or covered with the soil.

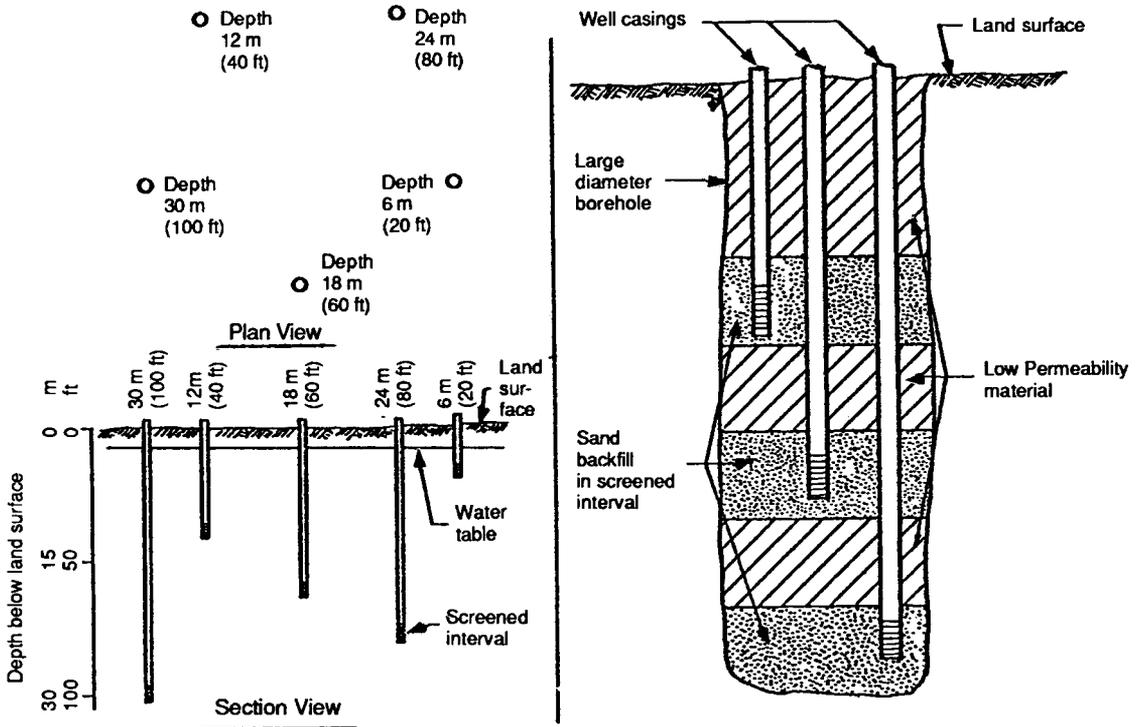


FIGURE 6.142 Typical monitoring well cluster configuration (94).

TABLE 6.146 Sanitary Landfill Design Data (92)

Parameter	Sludge-refuse	Sludge-soil
Sludge solids content	At least 3%	At least 20%
Ground slopes	Less than 30%	Less than 5%
Bulking ratio of agent to wet sludge		
3–10% solids	7:1	—
10–17% solids	6:1	—
17–20% solids	5:1	—
Over 20% solids	4:1	1:1
Cover soil thickness	0.5–1 ft (0.15–0.3 m) interim 2 ft (0.6 m) final	May not be required
Sludge application	500–4200 yd ³ /acre (900–7900 m ³ /ha)	1600 yd ³ /acre (3000 m ³ /ha)
Equipment	Tracked dozer and tracked loader	Tractor with disk

TABLE 6.147 Trench Fill Design Data (92)

Parameter	Narrow trench	Wide trench
Width	Less than 10 ft (3m)	Greater than 10 ft (3m)
Sludge solids content	15–20% for widths up to 3 ft (1 m) 20–28% for widths of 3 ft (1 m) or more	20–28% for land-based equipment Over 28% for sludge-based equipment
Ground slopes	Less than 20%	Less than 10%
Cover soil	2 to 3 ft (0.6–1 m) for widths up to 3 ft (1 m) 3–4 ft (1–1.2 m) for widths of 3 ft (1 m) or more	3–4 ft (1–1.2 m) for land-based equipment 4–5 ft (1.2–1.5 m) for sludge-based equipment
Sludge application rate	1200–5600 yd ³ /acre (2250–10,500 m ³ /ha)	3200–14,500 yd ³ /acre (6000–27,200 m ³ /ha)
Equipment	Backhoe with loader, excavator, trenching machine	Tracked loader, dragline, scraper, tracked dozer

TABLE 6.148 Trench Fill Utilization Calculations (90)

1. Trench utilization rate = $\frac{\text{sludge volume/day}}{\text{cross-sectional area of sludge in trench}}$
2. Sludge application rate = $\frac{\text{cross-sectional area of sludge in trench}}{\text{width of trench} + \text{spacing}}$
3. Land utilization rate = $\frac{\text{sludge volume/day}}{\text{land utilization rate}}$

Note: The term *sludge* includes any quantities of bulking agents required.

TABLE 6.149 Area Fill Design Data (92)

Parameter	Area fill mound	Area fill layer
Sludge solids content	At least 20%	At least 15%
Ground slopes	No limitation if suitably prepared	Level terrain preferred
Bulking ratio of soil to sludge:		
15–20% solids	—	1:1
20–28% solids	2:1	0.5:1
28–32% solids	1:1	0.25:1
Over 32% solids	0.5:1	Not required
Cover soil thickness	3 ft (0.9 m) interim 1 ft (0.3 m) final	0.5–1 ft (0.15–0.3 m) interim 2 to 4 ft (0.6–1.2 m) final
Sludge application rate	3000 to 14,000 yd ³ /acre (5630–26,300 m ³ /ha)	2000–9000 yd ³ /acre (3750–16,900 m ³ /ha)
Depth of lift		
15–20% solids	—	1 ft (0.3 m)
Over 20% solids	6 ft (1.8 m)	2–3 ft (0.6–0.9 m)
Number of lifts		
Over 15% solids	—	1 to 3 typical
20–28% solids	1 maximum	—
Over 28% solids	3 maximum	—
Equipment	Tracked loader, backhoe with loader, tracked dozer	Tracked loader, grader, tracked dozer

TABLE 6.150 Diked Containment Design Data (92)

Parameter	Land-based equipment	Sludge-based equipment
Sludge solids content	20 to 28%	Greater than 28%
Ground slopes	No limitation if suitably prepared	
Bulking ratio of soil to sludge	0.5:1	0.25:1 (not required with sludge greater than 32% solids)
Cover soil thickness	1–2 ft (0.3–0.6 m) interim 3–4 ft (0.9–1.2 m) final	2–3 ft (0.6–0.9 m) interim 4–5 ft (1.2–1.5 m) final
Sludge application rate	4800–15,000 yd ³ /acre (9100–28,400 m ³ /ha)	4800–15,000 yd ³ /acre (9100–28,400 m ³ /ha)
Depth of lift	4-ft (1.2–1.8 m)	6–10 ft (1.8–3 m)
Number of lifts	1–3 typical	1–3 typical
Equipment	Dragline, tracked loader, and scraper	

The design of sludge application systems includes consideration (see Table 6.151) of sludge origin and composition, soil characteristics, climate, vegetation, topography, and agricultural practices or reclamation objectives. (All of these factors should be monitored after the system is placed in operation to ensure appropriate application rates are maintained.) Annual sludge application rates may vary from as little as 0.5 to over 100 tons/acre (1 to over 225 Mg/ha).

Because of the potential of groundwater pollution with nitrates, nitrogen is typically the limiting factor in developing sludge application rates. Trace elements and pathogens are also important constituents; typical concentrations of metals in municipal sludges are presented in Table 6.152. The significance of heavy metals may be lessened by maintaining a pH of 6.5 or higher.

Agricultural Lands. When land is available and soil conditions are acceptable, sludge can be applied to agricultural lands. However, precautions must be taken in agricultural applications to prevent possible health hazards to those utilizing products from the farmlands.

Soil characteristics determine the feasibility of using a particular area to receive sludge applications. Favorable soils have high infiltration and percolation rates, provide high water and nutrient holding capacity, possess good drainability and aeration, and have a neutral or alkaline pH.

Since municipal sludges contain essential plant nutrients, they can be applied to provide all of the nitrogen and phosphorus and some of the potassium required by most crops. For example, the primary nutrient content of a liquid mixture (25% of dry weight) of digested primary and waste activated sludges is (86)

Nitrogen	3.5 to 6.4%
Phosphate (P ₂ O ₅)	1.8 to 8.7%
Potash (K ₂ O)	0.24 to 0.84%

Also, the value of feed and forage crops is increased by the addition of trace elements that may normally be deficient in the soil before sludge application. For example, zinc, copper, nickel, chromium, and selenium are trace elements often inadequate in animal diets.

Pathogen control is extremely important due to the possible direct exposure to sludge in the handling and application steps as well as in the food chain. Since some pathogens may survive a disinfection process, sludge disposal is not recommended for lands containing crops for direct human consumption. While the sludge generated by most municipal wastewater treatment plants is acceptable for use on agricultural lands, implementation of this technique often depends on the availability and interest of a recipient.

For agricultural sites, sludge application rates are lowest. Crop selection and sludge application tech-

TABLE 6.151 Sludge Land Application Checklist (90)

-
1. Sludge characteristics
 - a. Type of processing at treatment plant
 - b. Quantity: maximum, average, minimum
 - c. Analysis (concentration, particle characteristics, volatile, content nitrogen, phosphorus, potassium, heavy metals, pathogen content, pH)
 2. Type
 - a. Crop utilization (crop(s) chosen, tillage requirements, application/crop growth timing, application method)
 - b. Dedicated disposal (application method, tillage requirement)
 - c. Land reclamation (present condition of site, soil-sludge reactions, application methods, tillage, requirements)
 3. Site characteristics
 - a. Topography (limitations on application methods, erosion potential, crop compatibility)
 - b. Runoff control (from adjacent areas, on-site, storm flow added to liquid sludge quantity, cut-off trenches, embankments for runoff diversion)
 - c. Soil (Soil Conservation Service soil maps; soil profiles: location, physical properties, pH, CEC, heavy metals)
 - d. Geohydrology (map of geologic formations and discontinuities; groundwater: location, extent, use)
 4. Design criteria
 - a. Climatic factors (rainfall: quantity, duration, seasonal variation; wind velocity and direction; temperature variation)
 - b. Loading rates
 - (1) Metals (background level in soil, cation-exchange capacity limits, projected from industrial changes or pretreatment programs)
 - (2) Nitrogen (forms of nitrogen present, mineralization rate)
 - c. Crops
 - (1) Nutrient uptake (nitrogen, phosphorus, potassium, micronutrients)
 - (2) Compatibility with applications (tillage requirements, timing of planting and harvesting, can application be made on growing crop, forage, have leaves been washed by rain or irrigation)
 - (3) Harvesting requirement (single harvest each year, multiple harvest each year, continual or intermittent harvest (grazing))
 5. System components
 - a. Dewatering (percent solids achieved, chemicals used)
 - b. Field equipment (compatibility with crops, compatibility with sludge, moisture content, compatibility with climatic factors)
 - c. Storage (open, or covered; capacity to match nonoperating periods, nuisance condition prevention methods)
 - d. Buffer zones (size, vegetation provided, fencing)
 6. Monitoring
 - a. Land (pH, pathogens, available nutrients, heavy metals)
 - b. Crops (yield, tissue analysis, disease control)
 - c. Water quality (BOD, suspended solids, nutrients, coliforms, etc.)
 7. Reliability and flexibility (expandability, storage, alternate sludge management techniques)
-

niques influence the loading rate. For example, supplemental nitrogen requirements may be 5 to 10 tons/acre (10 to 20 Mg/ha) of liquid digested sludge. The application rates to supply phosphorus are less.

For agricultural lands, the design calculations typically include a four-step process (90):

1. Nitrogen balance
2. Metal accumulation
3. Nutrient balance
4. Application rate

TABLE 6.152 Typical Metal Content of Sludges (91)

Metal	Range	Value, mg/kg	
		Mean	Median
Arsenic	10–15	9	8
Barium	Up to 3000	1460	1300
Boron	200–1400	430	350
Cadmium	Up to 1100	87	20
Chromium	22–30,000	1800	600
Cobalt	Up to 800	350	100
Copper	45–16,030	1250	700
Lead	80–26,000	1940	600
Manganese	100–8800	1190	400
Mercury	0.1–89	7	4
Nickel	Up to 2800	410	100
Selenium	10–180	26	20
Silver	Up to 960	225	90
Strontium	Up to 2230	440	150
Vanadium	Up to 2100	510	400
Zinc	5 1–28,360	3483	1800

To determine the nitrogen balance, the sludge is first analyzed for its organic, ammonium, and nitrate nitrogen concentrations. Also, the nitrogen requirements of the crop must be determined for a desired crop yield (see Table 6.153). Finally, the amount of nitrogen available from mineralization of previously applied organic nitrogen and the amount of nitrogen available from present applications must be identified. The rate of conversion of organic nitrogen (mineralization rate) to plant-usable ammonium and nitrate forms must also be considered (see Table 6.154).

TABLE 6.153 Annual Nitrogen, Phosphorus, and Potassium Utilization by Crops (90)

Crop	Yield	Nitrogen, kg/ha	Phosphorus, lb/acre	Potassium
Corn	150 bu	207 (185)	39 (35)	200 (178)
	180 bu	269 (240)	49 (44)	223 (199)
Corn silage	29 Mg (32 tons)	224 (200)	39 (35)	228 (203)
Soybeans	50 bu	288 (257)	24(21)	112 (100)
	60 bu	377 (336)	33(29)	135 (120)
Grain sorghum	3.6 Mg (4 tons)	280 (250)	45 (40)	186 (166)
Wheat	60 bu	140 (125)	25 (22)	102 (91)
	80 bu	208 (186)	27 (24)	150 (134)
Oats	100 bu	168 (150)	27 (24)	140 (125)
Barley	100 bu	168 (150)	27 (24)	140 (125)
Alfalfa	7.2 Mg (8 tons)	504 (450)	39 (35)	446 (398)
Orchard grass	5.4 Mg (6 tons)	336 (300)	49(44)	349 (311)
Brome grass	4.5 Mg (5 tons)	186 (166)	33 (29)	236 (211)
Tall fescue	3.2 Mg (3.5 tons)	151 (135)	33 (29)	173 (154)
Bluegrass	2.7 Mg (3 tons)	224 (200)	27 (24)	167 (149)

*Values will vary with yield and regional crop practices.

TABLE 6.154 Release of Residual Nitrogen during Sludge Decomposition in Soil (90)

Years after sludge application	Organic N content of sludge, % of dry solids						
	2.0	2.5	3.0	3.5	4.0	4.5	5.0
	Kg N released per Mg sludge added						
1	2.0	2.4	2.8	3.4	3.8	4.4	4.8
2	1.8	2.4	2.8	3.2	3.6	4.2	4.6
3	—	2.2	1.3	3.0	3.4	4.0	4.4
	lb N released per ton sludge added						
1	1.0	1.2	1.4	1.7	1.9	2.2	2.4
2	0.9	1.2	1.4	1.6	1.8	2.1	2.3
3	0.0	1.1	1.3	1.5	1.7	2.0	2.2

The desired nitrogen balance is achieved by applying the amount of nitrogen required less than available and is expressed in pounds per acre (or kilograms or megagrams per hectare). The quantity of sludge required is the desired nitrogen loading divided by the percent nitrogen in the sludge; thus, it is the sludge dry weight application rate, measured in tons per acre (megagrams per hectare).

The soil cation-exchange capacity (CEC) is used to determine the maximum tolerable heavy metal accumulations for a given soil; values for three soil types appear in Table 6.155. If the soil pH is greater than 6.5, maximum values in Table 6.156 apply. Metal concentrations are cumulative.

Using the sludge application rate for nitrogen balance, the effects of metal accumulations are calculated. If metals would be applied in excess of the soil's CEC, the application is reduced, and the nitrogen balance is recalculated. With the reduced nitrogen loading from sludge, supplemental nitrogen must be added or a decrease in crop yield must be accepted.

In addition to nitrogen, the loadings of phosphorus and potassium should be determined to ensure that these nutrients are adequate for the prescribed crop yield. One or more supplements may be required.

Since sludges will have varying concentrations of water, the previous calculations were based on dry weight. Thus, actual sludge application rates are measured as volume per unit area. Similarly, the speed of application equipment, and the width of spreading must be considered.

Land Reclamation. Application of sludge to poor-quality lands frequently improves land quality; however, precautions must be taken to prevent leachate and runoff contamination from reclamation sites. Lands receiving sludge for reclamation purposes are subject to similar soil restriction as those outlined for agricultural applications. On a small scale, dewatered sludges may be used by local residents for soil conditioning.

Very high sludge application rates are used for land reclamation; the terrain may control the design due to

TABLE 6.155 Typical Ranges of Cation Exchange of Various Types of Soils (90)

Soil type	Range of CEC, meq/100 g
Sandy soils	1–10
Silt loams	12–20
Clay and organic soils	Over 20

TABLE 6.156 Total Amount of Sludge Metals Allowed on Agriculture Land (90)

Metal	Soil cation-exchange capacity, meq/100 g		
	0–5	5–15	> 15
	Amount of metal, lb/acre (kg/ha)		
Pb	500 (560)	1000 (1120)	2000 (2240)
Zn	250 (280)	500 (560)	1000 (1120)
Cu	125 (140)	250 (280)	500 (560)
Ni	50 (56)	100 (112)	200 (224)
Cd	5 (6)	10 (11)	20 (22)

the availability of practical application and site-management techniques. For example, steep slopes (over 5%) and low-lying fields require special attention. Also, the higher application may permit excessive accumulations of nitrogen, trace elements, or other sludge constituents. Monitoring and good soil management practice are required.

Specific physical process considerations and/or monitoring requirements during land application of sludges are (86)

- The trace element composition of sludge, soil, and crops
- The nitrogen content of sludge, soil, and crops, and potential nitrate pollution of aquifers
- The use of only disinfected sludge on low-growing fruits or vegetables to be consumed raw
- The hydraulic overloading of soil
- The ultimate use of land
- The practices to control runoff and erosion

Chemical Fixation. Particularly for sludges containing hazardous wastes, chemical fixation is an ultimate disposal alternative. Due to the high costs, this is not a feasible alternative for disposal of most wastewater sludges.

The process involves the mixing of the sludge with a chemical mixture that will harden to form a solid mass capturing the sludge and preventing leaching of pollutants. Portland cement and other commercial and proprietary cementing agents may be used.

Deep-Well Injection. The use of deep-well injection is an alternative for sludge disposal but must be thoroughly evaluated to minimize potential groundwater contamination from system failures. This is especially important in the disposal of sludges containing hazardous wastes.

The principle of deep-well injection is to locate a previous stratum below potable groundwater sources and an underlying impervious stratum. If the area is, and remains, free of geological faults, no chemical reactions occur to deteriorate the impervious stratum, and the well equipment functions properly, sludge disposal will be safely achieved.

TREATMENT FACILITIES

A prudent combination of theory and practical experience is necessary to evaluate and control wastewater pollutants. The engineering documents and the process, physical, and operational design considerations required for development of a wastewater treatment facility are discussed in this section.

Engineering Documents

The essential engineering documents for development of a wastewater collection, treatment, and disposal system are an engineering report, construction drawings, and specifications. The principles and general content of these documents, as well as operation and maintenance manuals, are presented in Chapter 1.

Engineering Report. The engineering report may be in letter form to address a specific improvement or may be a comprehensive bound document to address total system needs. Especially for the latter, the principal topics of a project report are

- Introduction
- Water quality objectives
- Regulatory requirements
- Existing environmental, water supply, wastewater disposal, land use, demographic, and economic development considerations
- Forecasts of flow and waste loads and of changes in the characteristics of the service area
- Evaluation of existing collection, treatment, and disposal facilities and wastewater sources and characteristics
- Development of project in terms of service area boundary, collection system routes, process design parameters, and treatment plant site requirements
- Analysis of process and site alternatives
- Detailed basis of design data for proposed project
- Implementation plan

Each aspect of the report must be tailored to the needs and objectives of the specific project.

Plans and Specifications. Construction drawings (plans) and specifications are the instruments necessary for a construction contractor to implement the project described by the engineering report. The principal engineering activities during the design phase are listed in Table 6.157.

After award of the construction contract, the plans and specifications not only control construction activities but also the program for keeping existing facilities in service and practical start up of new facilities during the construction period. Typically, record (as-built) drawings are prepared after construction to service as a reference for operating personnel and for future construction on the site.

TABLE 6.157 Principal Design Phase Activities

Notice to proceed	Equipment selection
Coordination with engineering report	Detailed design drawing
Pre-design conference	Specification outline
Site investigation	Utility easement preparation
Field survey	Specifications complete
Preliminary site plan	Contract documents complete
Soils investigation	Quality control review
Process/layout evaluation	Regulatory coordination
Final system layout	Final documents available
Preliminary cost estimate	Design complete
Basis of process design completed	

Process Design

The quantity of flow and individual pollutants to be treated and numerous other factors make process design considerations very site-specific. Key decision factors are listed in Table 6.158.

Using the five general categories of wastewater treatment processes for municipal-type sewage and the six categories of sludge-handling processes, Figure 6.143 illustrates a comprehensive process flow sheet. Being more specific, Figure 6.144 presents a flow sheet for an activated sludge system; the combination of processes is typical of a modern day secondary treatment plant.

Physical Design

Practical experience is desired in development of the physical features of the wastewater treatment plant. To compliment the treatment processes, the engineer considers site conditions, plant layout, plant hydraulics, aesthetics, and plant utilities. Each is important in minimizing capital and operating costs and in providing a successfully designed, operable facility.

Site Conditions. Soil conditions and flood and wetlands protection are the primary site conditions affecting design. Since site selections may be made without benefit of in-depth engineering investigations, sub-surface and flood control design problems may be encountered.

Soil conditions are defined by the soil type and erosion potential, including supportive strength. The sub-surface investigation should also identify and locate rock, compressible strata, and groundwater. If piles are required, additional studies are needed.

Through the use of high ground or dikes, the treatment plant should be protected as much as possible from flooding and flood damage. The generally accepted design practice is to protect structures and equipment from damage by the 100-year flood and to maintain treatment operations and access during the 25-year flood. Wetlands on the site will require special attention based on prevailing regulations.

Plant Layout. The layout of plant features must consider site conditions, such as elevations, soils, and aesthetics, as well as existing and future space requirements. The plant layout is also an important direct and indirect factor in overall constructability.

The components of a plant layout analysis include equipment and process layout, structures, roads, piping, earthwork, retaining walls, and hydraulic profile. Singularly and in combination, each component should be analyzed with respect to cost and benefits and to future operability of the treatment work.

TABLE 6.158 Process Design Considerations

Decision parameter	Remarks
Flow	Average and peak
Pollutant	Individual and combinations
Regulatory agencies	Acceptable processes and permit requirements
Existing facilities	Maximize use
Costs	Capital and operating limitations
Proven performance	Technology and application basis
Operability	Ease and flexibility of operation and maintenance
Expandability	Capability for future
Site restraints	Size and topographic
Recycle and reuse	By-product recovery
Environmental effects	Primary and secondary impacts

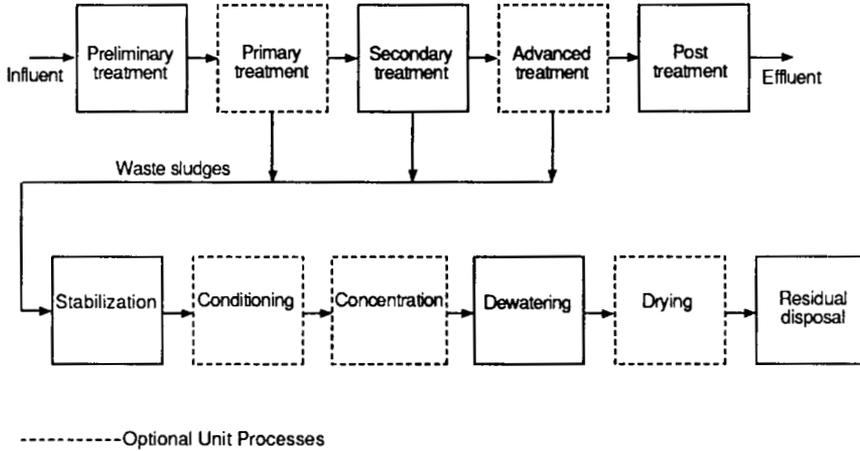


FIGURE 6.143 Conventional unit wastewater treatment processes.

Plant Hydraulics. A fundamental of wastewater treatment design is to provide as much flow by gravity as possible through the treatment facilities. To this end, hydraulic profiles should be developed for initial as well as design average and peak flow rates. The unit elevations are altered so that facilities will not be flooded or backed up during peak flows or as a result of effluent discharge restrictions due to stream flooding. During this process, head requirements are established for any pumps that may become necessary.

The plant hydraulic analysis should also consider the design of connecting pipes and channels. Sufficient velocity should be maintained to keep solids in suspension. Minimum velocities of 2 ft/s (0.6 m/s) for untreated (settled) wastewaters and 1 to 1.5 ft/s (0.3 to 0.5 m/s) for transport of organic solids are typically used.

Aesthetics. The wastewater treatment plant should have a good appearance and be acceptable to the local surroundings. The principal aesthetic features are landscaping, architecture, odor, and noise. Landscaping and architecture have received increased interest and support in recent years.

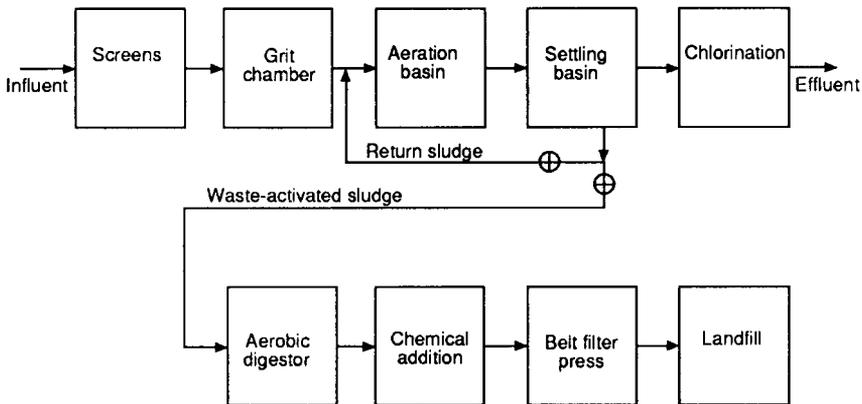


FIGURE 6.144 Typical secondary wastewater treatment system.

Most often, odor is the primary aesthetic problem. Obviously, the best odor control is to avoid the formation of odor-producing conditions. If necessary, chlorine or other oxidizing agents are used to reduce odor production. In some cases, odor control is through the collection of odorous gases and treatment with conventional processes, such as wet scrubbers. The least effective odor control is by use of a masking agent.

Noise management at a wastewater treatment plant not only impacts the neighborhood as a nuisance, but can be an occupational health problem. Federal standards are in effect to protect plant personnel.

Plant Utilities. For all wastewater treatment plants, there is a need for a potable water supply and electrical power. In some cases, natural gas is used as a power source; this may be in a dual system using digester gas.

The basic requirements for a water supply are

- Sanitary uses
- Cleanup operations
- Equipment cooling
- Equipment operation
- Process uses
- Fire protection

Cross-connection control and backflow prevention are extremely important for the system. With appropriate health safeguards, plant effluent may be used for many treatment plant operations.

Electrical power is common to all treatment plants. Its uses vary from influent pumping and area lighting to heavy demands for treatment process equipment such as aerators. The reliability of the power source is extremely important to the operation of the treatment works; for example, alternate sources or standby generators are normally required. The power system design significantly affects the operating costs for energy.

Operational Design

The technical operational aspects of the engineer's design is also affected by considerations for reliability, safety, and laboratory facilities.

Reliability. To the extent possible, the treatment works' design should provide for the capability of continuous operation in accordance with design objectives. Common factors impacting reliability are

- Significant variations in the quantity and/or quality of wastewater
- Power outage
- Equipment and/or instrumentation failure
- Maintenance disruptions
- Extreme floods
- Human error

It is good design practice to include some of the following features to improve reliability:

- Multiple units and equipment
- Unit bypasses
- Manual operation backup
- Emergency holding and storage capacities
- Standby or alternate power systems
- Identifying all operating devices
- Operator's manual and training

Safety. The physical and health dangers associated with wastewater treatment emphasize the need to consider safety during design. The basic safety objectives are the prevention of physical injuries, body infections, oxygen deficiency, and exposure to hazardous gases and substances. The design engineer should provide for not only safety features but also safety equipment at the treatment facility.

Typical safety features at a wastewater treatment plant are to

1. Install fencing or otherwise enclose the plant.
2. Use guardrails, chains, and machine guards around open tanks, equipment rooms, etc.
3. Provide a potable water supply protected by a backflow preventer and identify nonpotable water sources.
4. Install adequate lighting for nighttime operation and security.
5. Avoid crowding of equipment and operating devices, e.g., valves.
6. Use mechanical ventilation to exhaust areas with potentially noxious or hazardous gases.
7. Provide signage and special safety features in areas of chemical usage and storage.
8. Segregate anaerobic sludge-digestion facilities.
9. Secure and ground all electrical units.

Laboratory. Except in special cases (usually smaller facilities), a laboratory should be provided for analytical determinations of permitted parameters and for operational control tests.

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CHAPTER 7

STORMWATER MANAGEMENT

F. X. Browne, Ph.D., P.E.*

Stormwater management encompasses all elements of the hydrologic cycle but focuses on how humans affect the production, movement, and control of surface runoff. In a natural system, the rate of surface runoff is controlled by the rainfall rate, soil conditions, vegetation, and subsurface geology.

Urbanization creates large impervious areas that increase the quantity and peak rate of runoff. Rainfall then washes deposited materials directly into surface waters, causing stream pollution. Organics create oxygen demands, nutrients accelerate lake eutrophication, and heavy metals accumulate in bottom sediments.

Environmental engineers use modern stormwater management practices to employ natural and man-made systems to minimize environmental damage. A complete stormwater management program contains many elements, including on-site infiltration and detention, collection and transport systems, regional flood control, and major stream channel improvements (Figure 7.1).

In years past, sanitary wastes and stormwater were collected in the same (combined) sewer. During heavy rainfall, the sewers would overflow, creating water pollution problems. Many of these combined sewers have been separated, and the remaining overflows are treated as point sources. Stormwater management practices are used to lessen the requirements and costs for combined sewer overflow treatment facilities.

RAINFALL

Rainfall is the basic controlling factor in stormwater runoff processes. Because rainfall is the factor controlling stormwater runoff, it is important to understand some of the processes controlling rain formation as well as data sources and methods of describing rainfall. This section describes the storm types of primary interest along with common sources of rainfall data. Techniques for describing the temporal and spatial distribution of rainfall are also presented.

Occurrence and Distribution of Rainfall

Rainfall occurs through the rising and the subsequent expansion and cooling of moist air. This cooling causes the formation of droplets on particles of dust suspended in the air. These droplets grow to sufficient size to overcome air resistance and fall to earth by two processes: (1) coalescence or the collision of numerous small droplets; or (2) ice crystal growth by diffusion of water vapor away from liquid droplets to ice crystals because of the lower vapor pressure at the ice surface.

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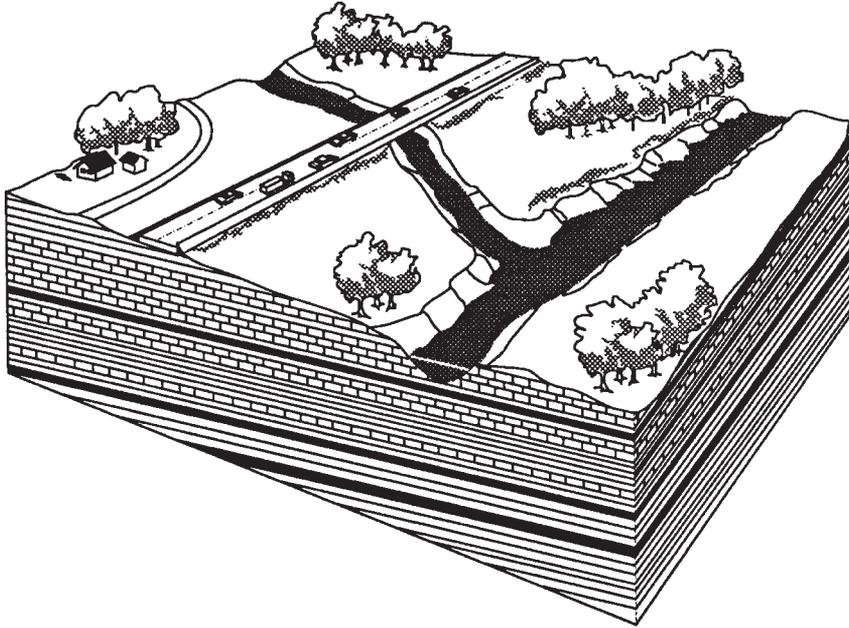


FIGURE 7.1 Elements of the total stormwater system.

Rainfall is commonly classified according to the factors primarily responsible for the rise of the air mass. These factors can be separated into three main categories: convective, orographic, and cyclonic.

Convective Precipitation. This form of precipitation is typical of warm areas, primarily the tropics, and is caused by heating of air at the ground surface. The heated air expands and rises while picking up moisture along the way. At some point, the air mass rises to the point, where cooling and condensation begin to occur.

Thunderstorms are extreme examples of convective precipitation. In these storms, strong rapid updrafts occur during the initial phases. In the upper portion of the storm cloud, the air mass expands to the point at which condensation and droplet formation occur. The strong updrafts prevent these droplets from falling to earth while a large quantity of water accumulates in the top of the cloud. Eventually the weight of the water being supported retards the updraft enough to permit rain to fall. The falling water destroys the strong updraft, allowing the accumulated water to fall in large downpours.

When a thunderstorm first develops, it usually covers only a small area of 3 to 4 mi² (8 to 10 km²) but the area may spread out to 50 to 100 mi (80 to 160 km) in length. These storms most commonly travel from west to east at approximately 30 mi/h (50 km/h). The average life of a thunderstorm is approximately 6 h (1).

Orographic Precipitation. This form of precipitation is caused by the mechanical lifting of a moist air mass over mountain barriers. The high average annual rainfall along the coast of the Pacific Northwest of the United States is caused by orographic lifting.

Cyclonic Precipitation. This type of precipitation results from the movement of masses of air from areas of high pressure to low pressure. Cyclonic precipitation can be further classified as frontal or nonfrontal. Frontal precipitation occurs when warm air is forced over a colder air mass. A front is termed a warm front

if the warm air is advancing and a cold front if the converse is true. Frontal precipitation commonly occurs throughout the United States during the cooler months, with storms that last up to several days at a time.

Rainfall Data Sources

There are two general types of rainfall data: raw rainfall data and rainfall frequency data.

Raw Rainfall Data. The primary source of rainfall information in the United States is the National Weather Service, which maintains a system of approximately 13,000 rain gauge stations throughout the United States (2). Rainfall data are published in a number of different forms. Daily rainfall amounts are published for each state in “Climatological Data” (3). Total storm rainfall and hourly amounts can be obtained from “Hourly Precipitation Data” (4). Data on specific storms can be obtained from the National Climatic Center, NOAA Environmental Data Service in Asheville, North Carolina.

The National Weather Service rain gauge network consists of both recording and nonrecording stations. The majority of these stations are simple bucket gauges, the design of which is standardized to ensure uniformity of results. The three types of recording gauges in common use are the tipping bucket, weighing gauge, and float gauge. Tipping bucket gauges record the rainfall rate, whereas weighing and float gauges record accumulated rainfall. Most of the gauges in current use record the variation in precipitation by a pen trace, but these are gradually being replaced by machine-readable paper tape recorders.

In addition to the National Weather Service, many local individuals and agencies collect their own rainfall information. Of particular interest are flood control and irrigation districts, water and wastewater treatment plants, universities, and private individuals. Informal bucket surveys of water collected in troughs, buckets, or similar containers can be very useful in analyzing storms when no other data are available.

Rainfall Frequency Data. Depth–duration–frequency analyses of historic rainfall have been performed for most areas of the United States and are published by a variety of sources. The most common source of this information are isopluvial maps published by the National Weather Service in “NOAA Atlas 2” (5) for the 11 conterminous states of the United States west of the 103rd parallel and “NWS HYDRO-35” (6) and “USWB TP-40” (7) for the remaining 37 conterminous states. More detailed information is often available from state agencies (8).

Isopluvial maps consist of lines of equal rainfall for a specific duration and return period. The first collection of these maps was prepared by Yarnell (9).

Maps from HYDRO-35 and TP-40 presenting 2-year and 100-year frequency rainfall depths for 5-min, 15-min, 1-h, and 24-h durations are shown in Figures 7.2 to 7.9. Equations (7.1) to (7.6) were developed as part of these studies and can be used to calculate intermediate durations.

$$10 \text{ min} = 0.59 (15 \text{ min}) + 0.41 (5 \text{ min}) \quad (7.1)$$

$$30 \text{ min} = 0.49 (60 \text{ min}) + 0.51 (15 \text{ min}) \quad (7.2)$$

$$2 \text{ h} = 0.11 (24 \text{ h}) + 0.89 (1 \text{ h}) \quad (7.3)$$

$$3 \text{ h} = 0.22 (24 \text{ h}) + 0.78 (1 \text{ h}) \quad (7.4)$$

$$6 \text{ h} = 0.50 (24 \text{ h}) + 0.50 (1 \text{ h}) \quad (7.5)$$

$$12 \text{ h} = 0.75 (24 \text{ h}) + 0.25 (1 \text{ h}) \quad (7.6)$$

For example, the rainfall depth for a 10-min duration rainfall can be calculated by substituting the rainfall depths for the 15- and 5-min rainfalls in Eq. (7.1).

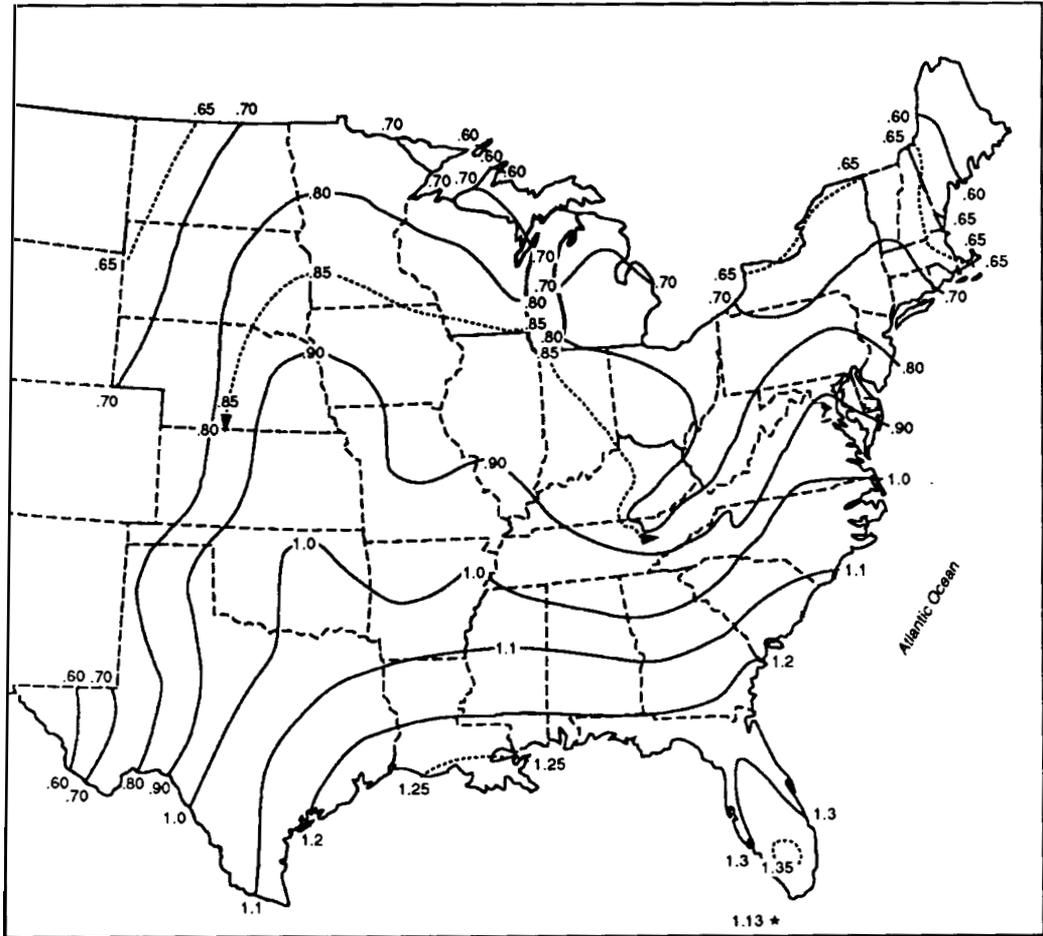


FIGURE 7.2 2-year, 15-min precipitation (inches; 1 in = 2.54 cm) (6).

Equations (7.7) to (7.10) were derived directly from the Fisher–Tippett Type I distribution and can be used to calculate rainfall depths for frequencies between 2 and 100 years.

$$5 \text{ year} = 0.278 (100 \text{ year}) + 0.674 (2 \text{ year}) \quad (7.7)$$

$$10 \text{ year} = 0.449 (100 \text{ year}) + 0.496 (2 \text{ year}) \quad (7.8)$$

$$25 \text{ year} = 0.669 (100 \text{ year}) + 0.293 (2 \text{ year}) \quad (7.9)$$

$$50 \text{ year} = 0.835 (100 \text{ year}) + 0.146 (2 \text{ year}) \quad (7.10)$$

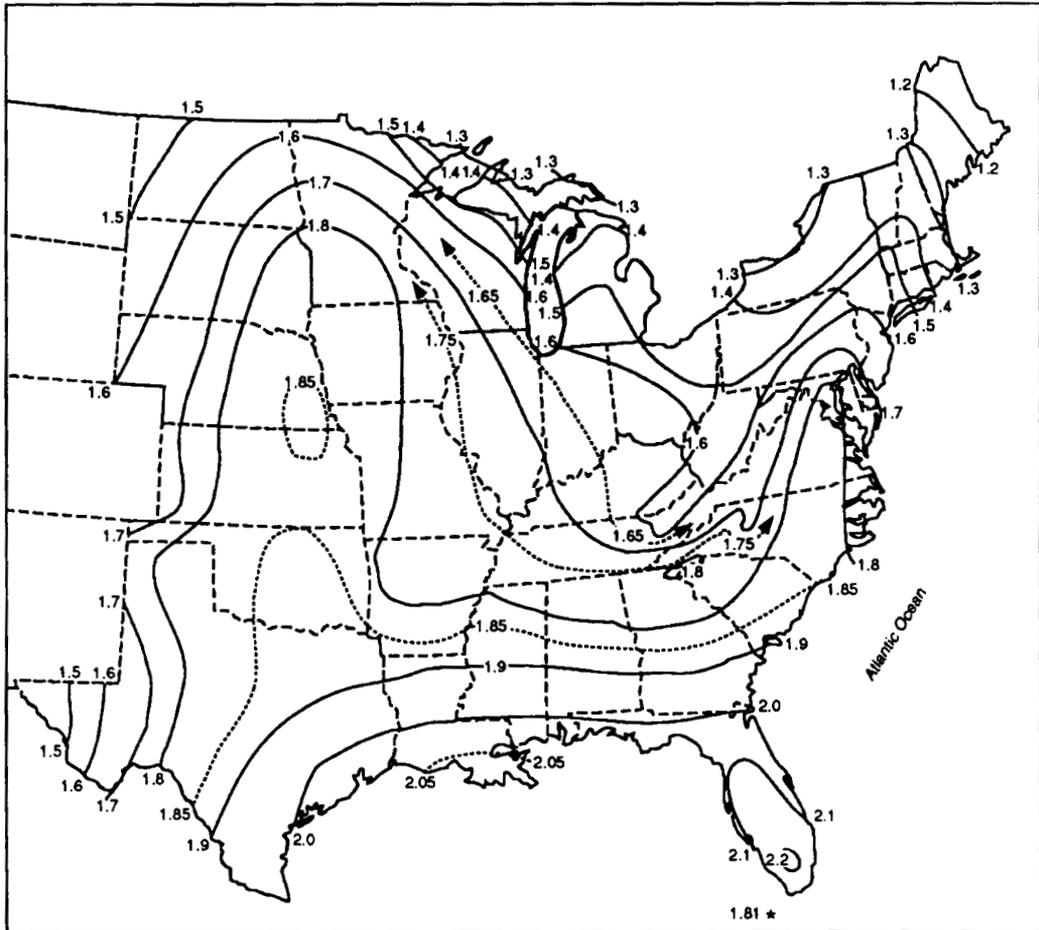


FIGURE 7.3 100-year, 15-min precipitation (inches; 1 in = 2.54 cm) (6).

Design Storms

In designing most stormwater management structures, it is usually necessary to select one or at best two or three rainfall depths and time distributions to be used in design. The optimum procedure would be to perform a continuous simulation of rainfall, temperature, and soil moisture to determine a full record of runoff. The frequency of a given event could then be defined and the optimum design could be determined based on expected costs and benefits. This type of analysis is obviously not warranted for most small stormwater management structures and a single or group of “design storms” is usually selected.

The design storm used may be a specific historical event or a hypothetical event based on frequency analyses. Selection of the type of event depends on the analyses required. Common practice in stormwater

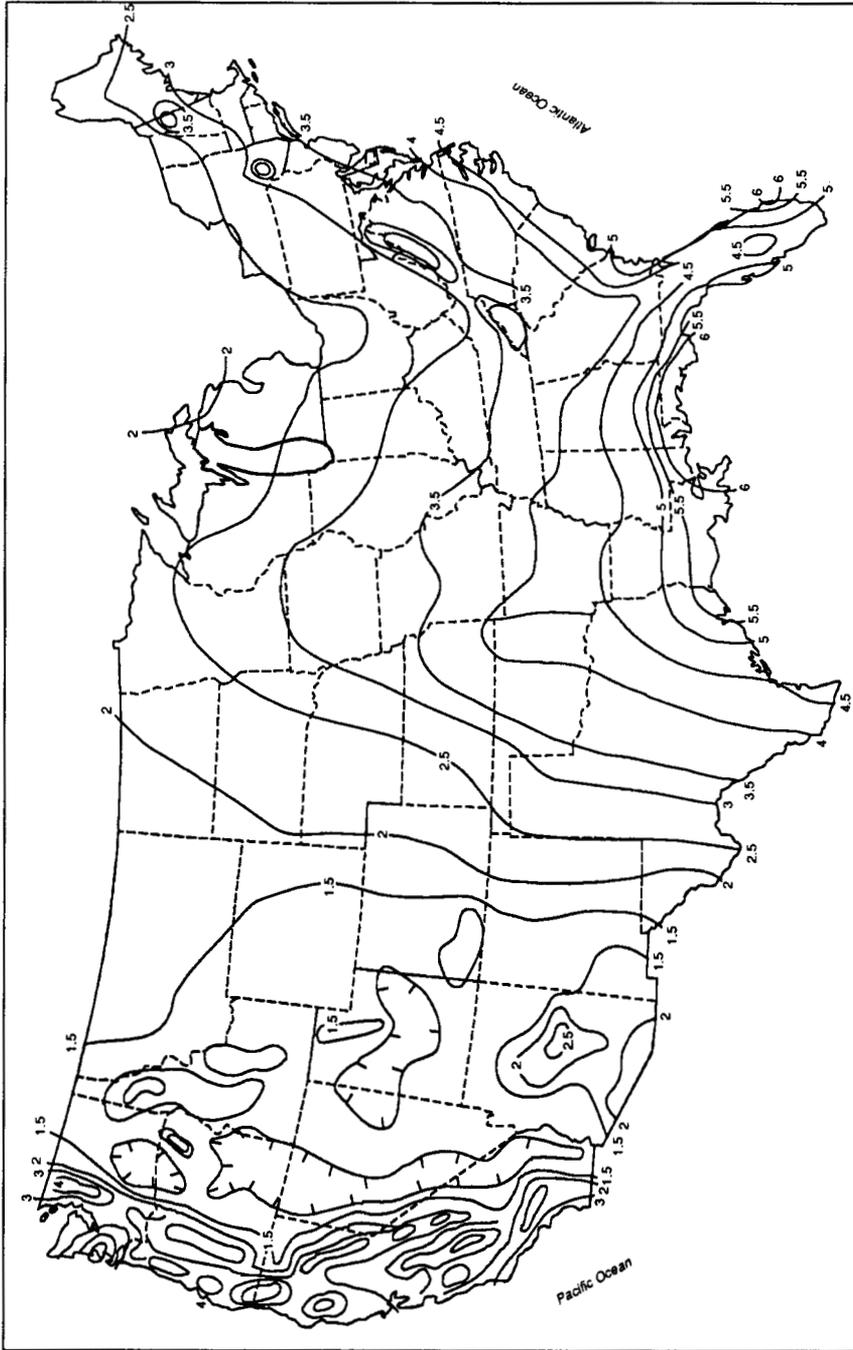


FIGURE 7.4 2-year, 24-h precipitation (inches; 1 in = 2.54 cm) (7).

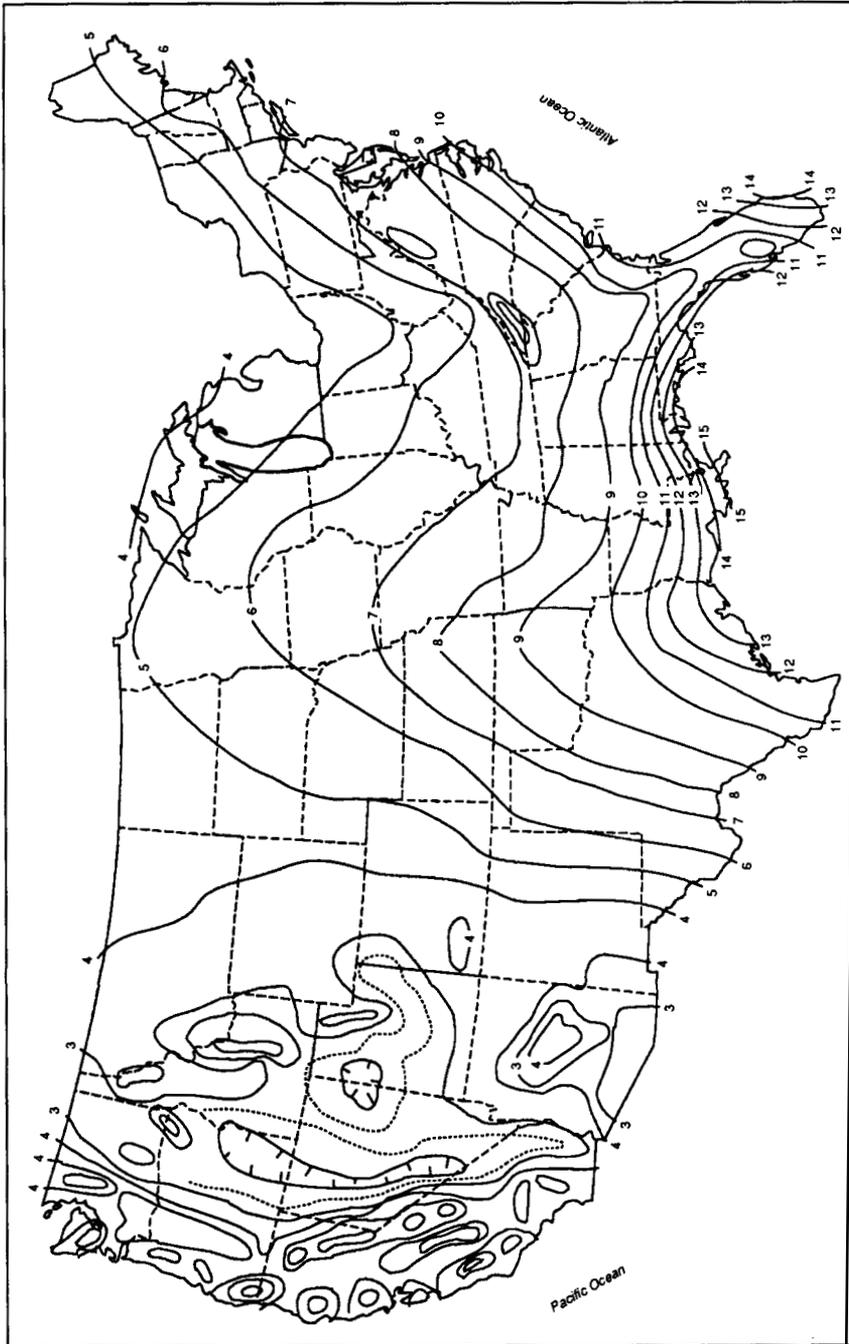


FIGURE 7.5 100-year, 24-h precipitation (inches; 1 in = 2.54 cm) (7).

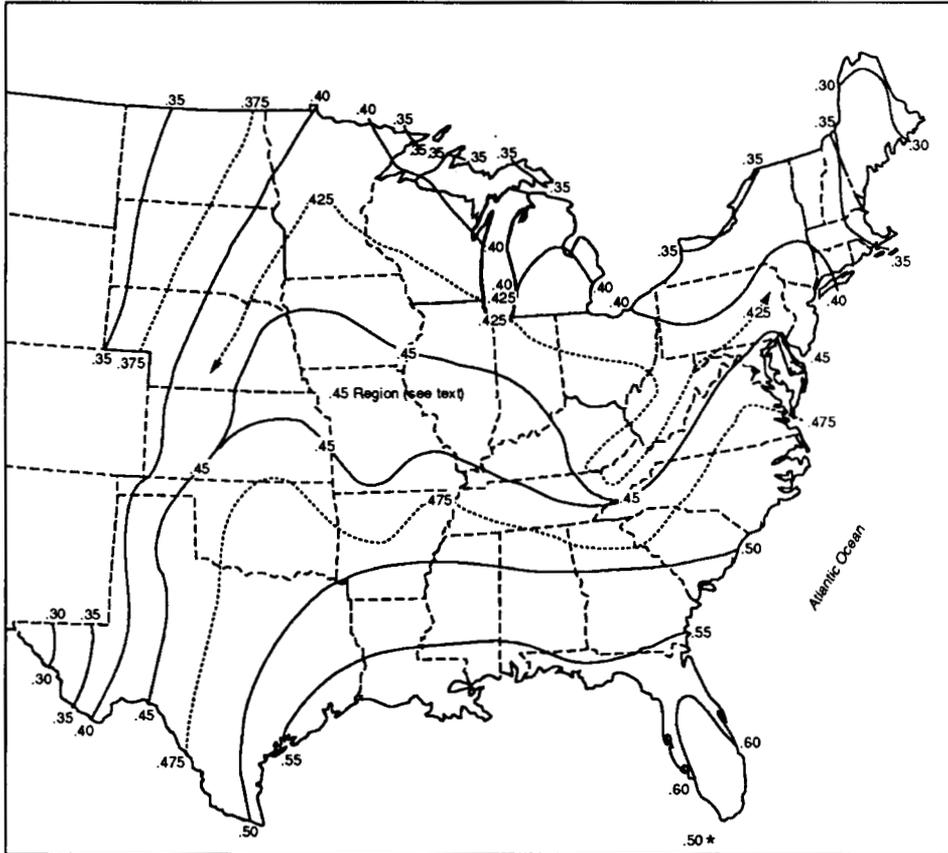


FIGURE 7.6 2-year, 5-min precipitation (inches; 1 in = 2.54 cm) (6).

management is to design controls based on a peak runoff rate or volume for a given recurrence interval (T_w). A design rainfall of the same recurrence interval is applied to the watershed under average conditions, and the predicted runoff rate or volume is assumed to have the same recurrence interval. This assumption is extremely common but also incorrect.

The probability that a given runoff rate or volume will occur is a function of the watershed characteristics and the joint probability of rainfall and other meteorological conditions. In much of the United States, stormwater management design is aimed at high-intensity thunderstorms. These thunderstorms most commonly occur in the summer, when soil moisture is low and vegetation is most dense. The conditions that occur during the summer are definitely not the year-round average and will not produce the same quantity of runoff that average conditions will. The design storms used for other areas may be different, but the result is similar: a storm of recurrence interval T_w , will not necessarily produce runoff of the same recurrence interval under average conditions.

Design based on a given frequency rainfall is nonetheless very useful. A 10-year rainfall under average conditions may not precisely predict the 10-year runoff, but it will give some measure of the approximate quantity of runoff to be expected. Also, many government agencies specify frequencies of rainfall to be used

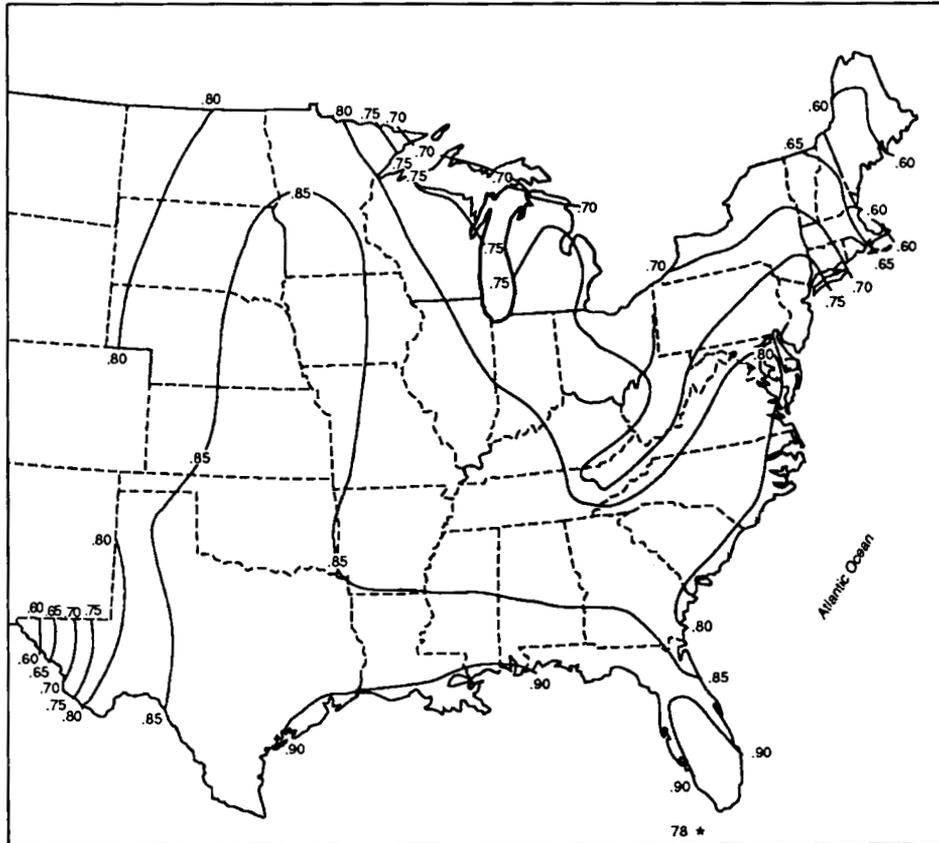


FIGURE 7.7 100-year, 5-min precipitation (inches; 1 in = 2.54 cm) (6).

in design. These design rainfalls, while not exactly fulfilling their original intention, are very helpful, and years of use have shown them to be adequate for most purposes.

In this chapter, rainfalls of a given recurrence interval are discussed as if they would predict runoffs of the same recurrence interval. Although this is not precisely correct, it is done to conform with common practice.

Design Rainfall Frequency. The frequency to be used in the design of stormwater management facilities is generally specified by state or local governments. Common design frequencies are shown below (11):

- 2-year storm—curb and sump inlets
- 10-year storm—storm sewers, culverts under minor roads
- 25-year storm—major roads, primary highways
- 50-year storm—interstate highways

Intensity–Duration–Frequency Relationships. Intensity–duration–frequency curves are generally more convenient to use when designing small structures than the depth frequency maps available from the Nation-

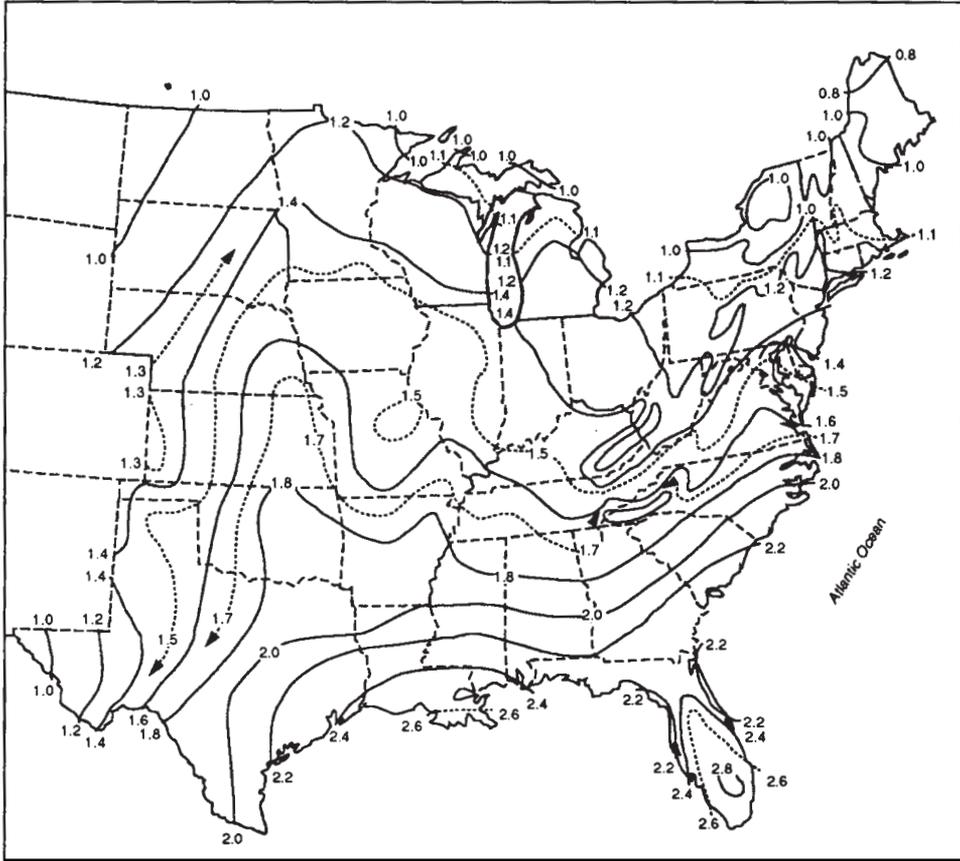


FIGURE 7.8 2-year, 60-min precipitation (inches; 1 in = 2.54 cm) (6).

al Weather Service. These curves (Figure 7.10) are available for most major cities from local government agencies (14).

When using computers in the design of stormwater management facilities, it is convenient to transform intensity–duration–frequency curves into mathematical equations which can be easily programmed. Experience has shown that Eqs. (7.11) and (7.12) fit these curves well (15).

$$i = \frac{a}{(t_d + c)^b} \tag{7.11}$$

and

$$i = \frac{a}{(t_d^b + c)} \tag{7.12}$$

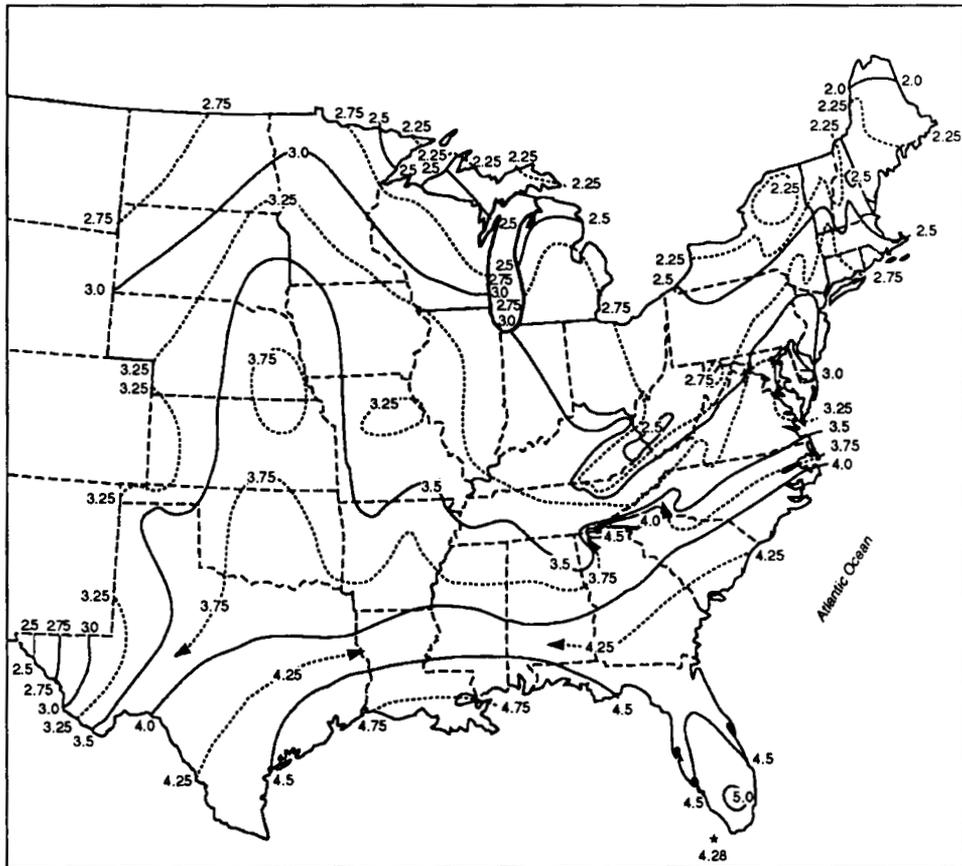


FIGURE 7.9 100-year, 60-min precipitation (inches; 1 in = 2.54 cm) (6).

where i = rainfall intensity

t_d = duration

a = constant dependent on return period

b, c = constants independent of return period

The constants in these equations can be calculated by using three simultaneous equations with data taken from existing graphs or by using curve-fitting procedures. These equations should only be used to calculate rainfall intensities within the range for which the equations were calibrated.

Coefficients for these equations vary considerably with location. Constants for various cities and a 10-year recurrence interval, calculated and summarized by Chen (16), are shown in Table 7.1.

Design Hyetographs. When designing stormwater management facilities, graphs of rainfall intensity versus time (hyetographs) are frequently required as input data. Intensity–duration–frequency curves describe the probability of a given rainfall intensity and duration but tell nothing about the order in time that the rain-

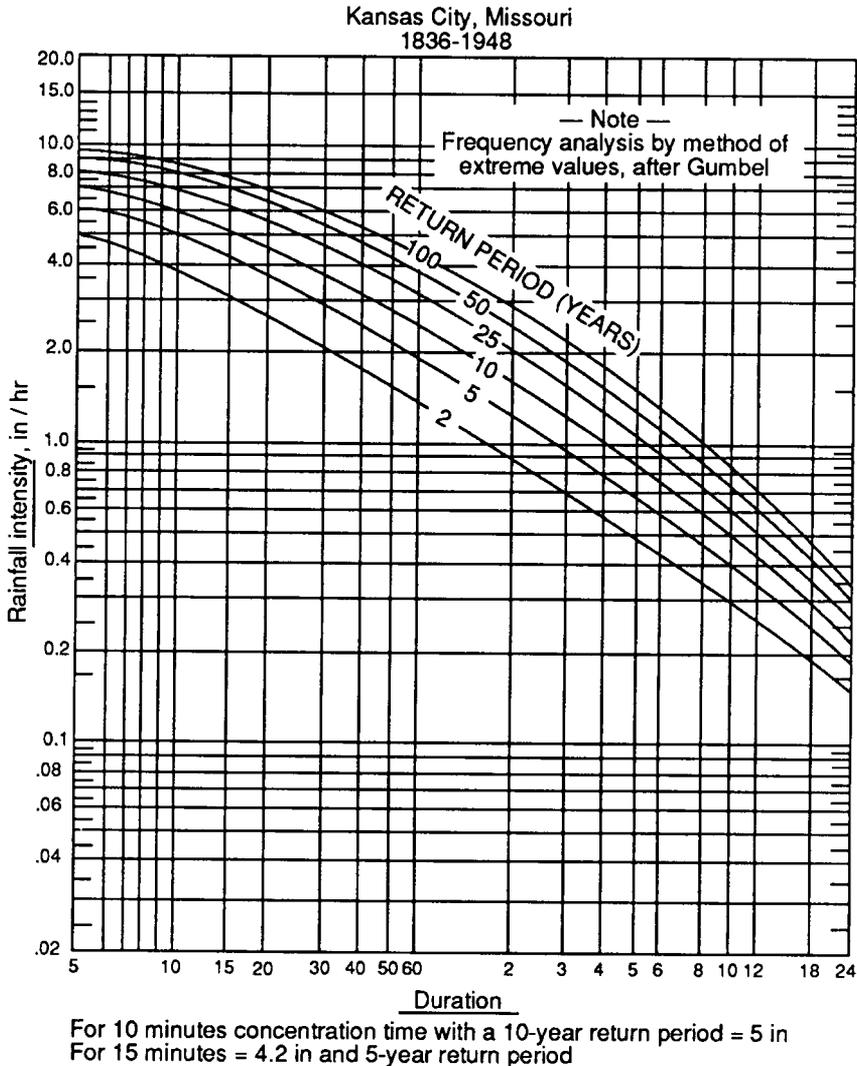


FIGURE 7.10 Rainfall intensity-duration curves (14).

fall may occur. Various researchers have proposed methods for developing design hyetographs (17-21), several of which are shown in Figure 7.11.

The most commonly used design hyetographs are the SCS Type I and II distributions. Mass curves for these distributions are shown in Figure 7.12. These distributions were developed by taking the maximum 5-min rainfall intensity and nesting it within the maximum 10-min intensity and so on up to 24-h. The shape of each distribution was found to be similar for areas in which the same type of storm controls peak rainfall. In areas that experience intense short-duration summer thunderstorms, a Type II distribution fits best. In areas where intense short-duration storms are uncommon, a Type I distribution fits best.

TABLE 7.1 Constants for 10-Year Return Period Calculations (16)*

Location	$i = \frac{a}{(t_d + c)^b}$			$i = \frac{a}{(t_d^b + c)}$		
	<i>a</i>	<i>b</i>	<i>c</i>	<i>a</i>	<i>b</i>	<i>c</i>
Chicago	60.9	0.81	9.56	94.9	0.88	9.04
Denver	50.8	0.84	10.50	96.6	0.97	13.90
Houston	98.3	0.80	9.30	97.4	0.77	4.80
Los Angeles	10.9	0.51	1.15	20.3	0.63	2.06
Miami	79.9	0.73	7.24	124.2	0.81	6.19
New York	51.4	0.75	7.85	78.1	0.82	6.57
Olympia	6.3	0.40	0.60	13.2	0.64	2.22
Atlanta	64.1	0.76	8.16	97.5	0.83	6.88
Helena	30.8	0.81	9.56	36.8	0.83	6.46
St. Louis	61.0	0.78	8.96	104.7	0.89	9.44
Cleveland	47.6	0.79	8.86	73.7	0.86	8.25
Santa Fe	32.2	0.76	8.54	62.5	0.89	9.10

*Constants correspond to *i* in inches per hour and *t_d* in minutes; 1 in = 2.54 cm.

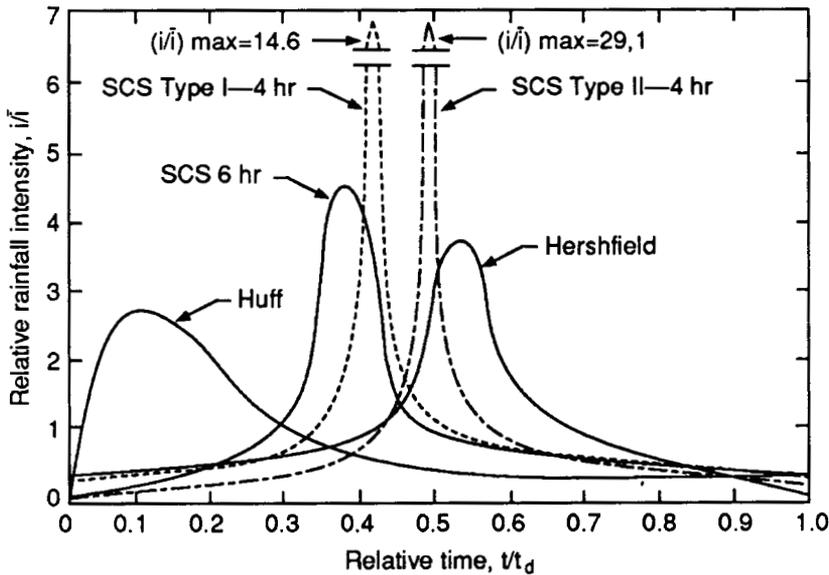


FIGURE 7.11 Dimensionless design rainfall hyetographs (17–21).

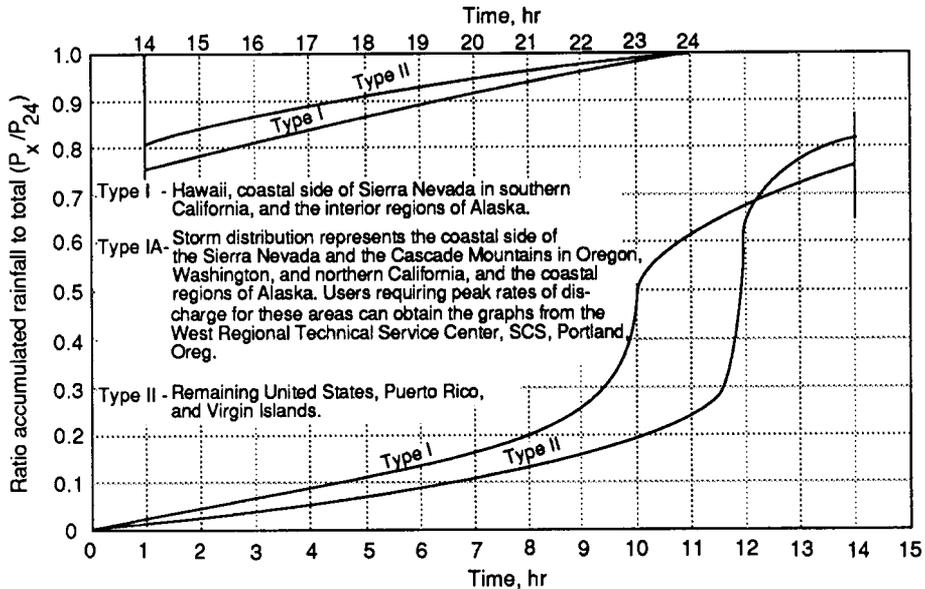


FIGURE 7.12 Standard SCS 24-h rainfall distribution curves (21).

SURFACE RUNOFF

Surface runoff is the primary concern in most stormwater management problems. Culverts, drainage channels, and detention ponds are all typically designed for surface runoff with groundwater flow assumed to be negligible. An exception to this is the analysis of infiltration to sanitary sewer systems. In these systems, groundwater is the primary control and must be analyzed in detail to properly characterize flows. The analysis of groundwater patterns is beyond the scope of this text and the reader is referred to standard hydrology texts for treatment of this subject (1, 22–24).

Knowledge of the peak rate of runoff is sufficient for the analysis and design of simple stormwater management systems. In these cases, use of empirical methods such as the rational formula or site-specific regression equations can be used to calculate the peak runoff rate. Where more detailed analysis of the hydrology and hydraulics is required, calculation of runoff volume and hydrographs are necessary.

Runoff Processes

The processes that control the generation and transport of surface runoff must be understood in order to accurately predict the flow rate at any point. When rainfall falls from the sky, a portion wets the surfaces of vegetation and other objects (interception). Water reaching the soil surface may seep into the ground (infiltration) or collect in small dips or holes (depression storage). At the beginning of a storm, the available infiltration rate is usually high and drops to a lower steady-state rate with continued rainfall. If the rainfall rate exceeds the available infiltration rate, surface runoff will occur. This runoff will move downslope as an irregular sheet of overland flow; the velocity is controlled by the land slope and friction losses. This water eventually collects into small rivulets and gulleys that flow into larger creeks and rivers. The flow rate at any given point is controlled by the surface runoff contributed by the drainage areas and the flow path of the water.

Stormwater runoff may be generated by several processes, including Hortonian overland flow, saturation overland flow, and subsurface storm flow.

Hortonian Overland Flow. This type of flow occurs when the rainfall rate exceeds the soil's capacity to absorb water and is most common in arid or semiarid areas where the hydraulic conductivity of the soil is low. An extreme example of this type of flow is runoff from impervious surfaces. In this case, the infiltration rate is zero.

In humid areas, vegetation breaks up the soil surface and Hortonian overland flow only occurs in small areas where vegetation has been removed and the soil compacted. In areas where there is significant vegetation, the limiting soil infiltration capacity is frequently very high; the Hortonian overland flow is restricted to only the most extreme storms. Stormwater runoff is commonly observed for small storms in these areas so some other process must be controlling.

Saturation Overland Flow. This type of flow occurs when the water table rises to the ground surface and prevents rainfall infiltration. In areas where this occurs, any rainfall will run off as overland flow. Saturated areas are common in valley bottoms and typically expand upslope as the storm progresses. Saturation overland flow can also occur when the downward movement of water is impeded by a soil layer of lower hydraulic conductivity. As infiltration occurs, a perched water table builds upon this layer and may eventually reach the soil surface, causing overland flow to occur.

Subsurface Storm Flow. This type of flow occurs everywhere, but it is only significant in soils with very high hydraulic conductivities. Storm flow volumes are typically less than 30% of rainfall, and peak flows may be delayed 20 to 30 h for even small watersheds (24). Subsurface storm flow, therefore, generally responds too slowly to be of significance in stormwater management.

Drainage Basin Parameters

The most basic descriptor is drainage basin area. Most basins are idealized as approximately pear-shaped as shown in Figure 7.13. When the basin outline varies radically from this shape, such as shown in Figure 7.14, it is necessary to divide the basin into subareas. If the times of concentration T_c (see next section) are approximately equal, the peak flows may be added directly. If the times of concentration are significantly different, hydrographs for each basin must be developed and added together.

Time of Concentration and Basin Lag. Time of concentration T_c is defined as the time of travel from the most remote point in the watershed to the basin outlet. Basin lag T_L is defined as the time between the centroid of the rainfall excess and the hydrograph peak. Virtually all methods for predicting peak runoff of hy-

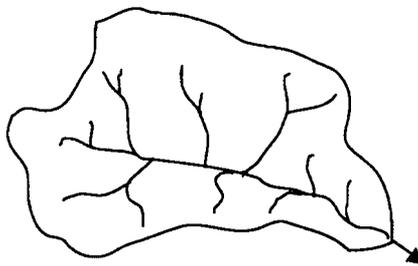


FIGURE 7.13 Idealized pear-shaped watershed.

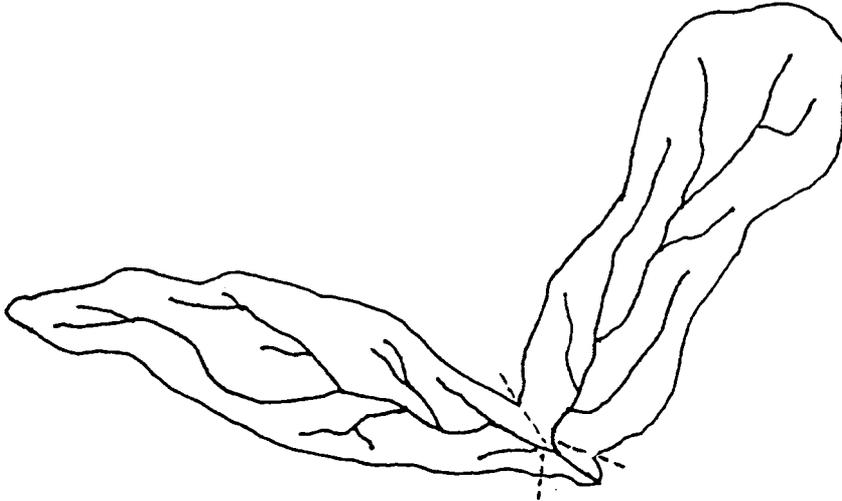


FIGURE 7.14 Subdivided irregularly shaped watershed.

drograph shape require one or both parameters. Basin lag is used in many methods, but it is frequently defined in terms of T_c . The Soil Conservation Service (SCS) defines $T_L = 0.6T_c$ (10). T_c is usually computed by adding the overland flow travel time and the channel travel time.

Overland Flow Travel Time. The overland flow travel time is usually calculated using one of the equations shown below. Detailed reviews of methods for calculating overland travel time may also be found in Refs. 37 and 38. Overland travel time is always given in minutes.

The Federal Aviation Agency calculates overland time travel as

$$\text{Overland travel time} = \frac{X(1.1 - C)L^{0.50}}{S^{0.333}} \quad (7.13)$$

where C = rational method runoff coefficient

L = length of overland flow, ft or m

S = surface slope, %

X = 1.8 (3.26 for SI units)

This method, called the kinematic wave formula, was developed from airfield drainage data assembled by the Corps of Engineers. It is intended for use on airfield drainage problems but has been used frequently for overland flow in urban basins (28).

$$\text{Overland travel time} = \frac{XL^{0.6}n^{0.6}}{i^{0.4}S^{0.3}} \quad (7.14)$$

where X = 0.94 (2.78 for SI units)

L = length of overland flow, ft or m

n = Manning roughness coefficient (Table 7.2)

i = excess precipitation intensity, in/h or cm/h

S = average overland slope, ft/ft or in/in

TABLE 7.2 Recommended Manning Roughness Factors for Overland Flow (25)

Type of surface	Manning's <i>n</i>
Smooth impervious surface	0.035
Smooth, bare, packed soil, free of stones	0.05
Poor grass, moderately bare surface	0.10
Pasture or average grass cover	0.20
Dense grass or forest	0.40

This equation was developed from kinematic wave analysis of surface runoff from developed areas (29). When solved mathematically, this method requires iteration since both *i* (excess rainfall intensity) and the overland travel time are unknown. Values of Manning's *n* can be obtained from Table 7.2.

The SCS lag equation is another formula.

$$\text{Overland travel time} = \frac{XL^{0.8} \left(\frac{1000}{\text{CN}} - 9 \right)^{0.7}}{1900S^{0.5}} \tag{7.15}$$

- where *L* = hydraulic length of longest flow path, ft or m
- CN = SCS runoff curve number (Table 7.9)
- S* = average watershed slope, %
- X* = 60 (155 for SI units)

This equation, developed by the SCS from agricultural watershed data (41), is generally used for rural areas. It tends to overestimate the overland travel time for mixed areas. The SCS recommends adjustment factors be applied to correct for channel improvement (Figure 7.15) and impervious areas (Figure 7.16) (31). McCuen et al. (32) indicate that the present SCS lag factors found in TR-55 are inadequate and propose a set of modified lag factors (Figure 7.17). Studies by Kibler et al. (25) indicate that McCuen's revised lag factors yield more accurate estimates than the original SCS factors.

The time of concentration can be calculated by dividing the flow length by the overland flow velocity obtained from Figure 7.18 (31).

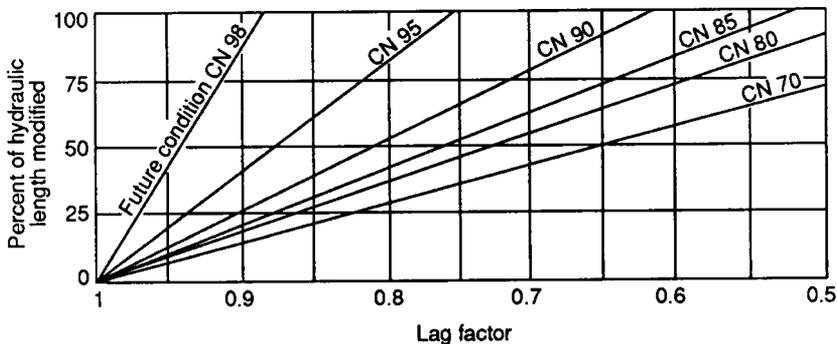


FIGURE 7.15 Factors for adjusting lag from Eq. (7.15) when the main channel has been hydraulically improved (31).

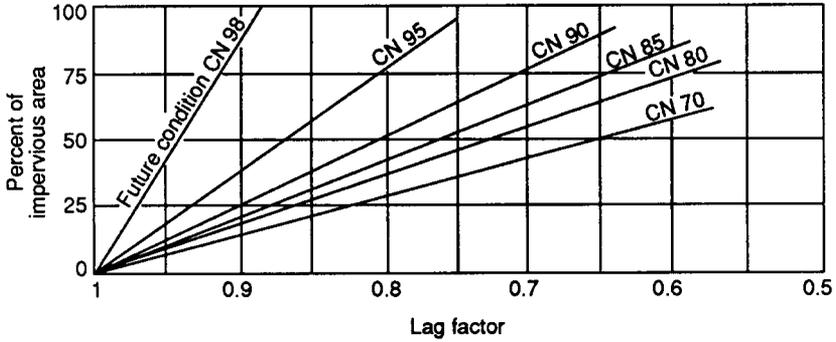


FIGURE 7.16 Factors for adjusting lag from Eq. (7.15) when impervious areas occur in the watershed (31).

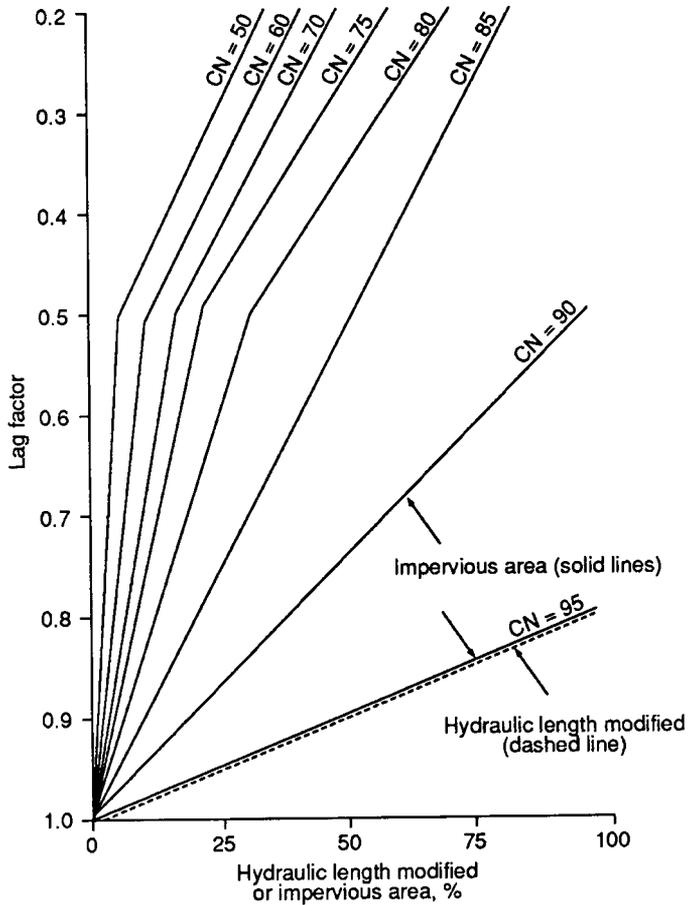


FIGURE 7.17 Proposed lag factor versus hydraulic length modified or impervious area (32).

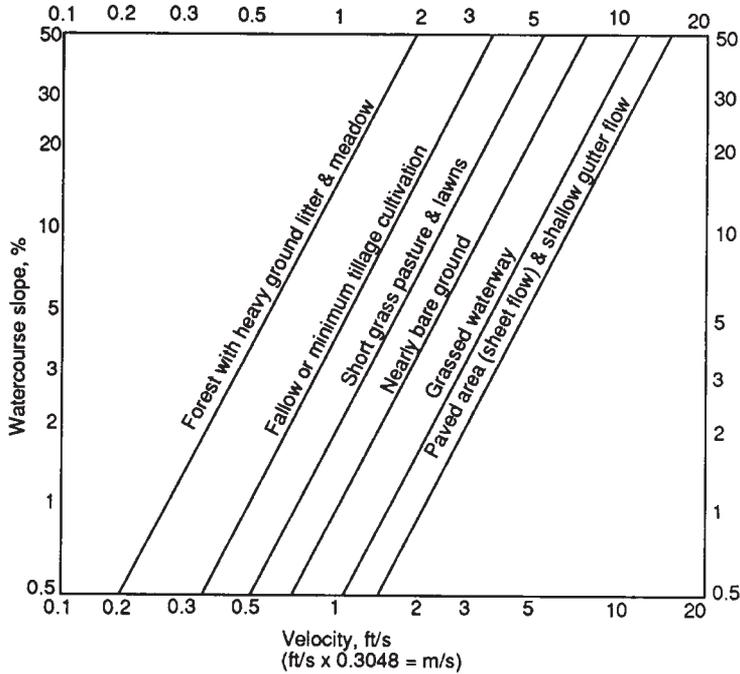


FIGURE 7.18 Average velocities for estimating travel time for overland flow.

Channel Travel Time. Typical channel velocities are presented in Table 7.3. Where cross sections of the channel are available, Manning’s equation should be used to calculate velocity.

$$V = \frac{X}{n} R_H^{2/3} S^{1/2} \tag{7.16}$$

- where V = velocity, ft/s or m/s
- R_H = hydraulic radius, ft or m
- S = channel slope, ft/ft or m/m
- n = Manning’s n
- $X = 1.486$ (3.28 for SI units)

When using this equation to calculate travel time, the channel is normally assumed to be flowing full.

TABLE 7.3 Typical Channel Velocities (13)

Slope, %	Woodlands (upper portion watershed)		Pastures (upper portion watershed)		Natural channel (not well defined)	
	m/s	ft/s	m/s	ft/s	m/s	ft/s
0–3	0.3	1.0	0.5	1.5	0.3	1.0
4–7	0.6	2.0	1.0	3.0	1.0	3.0
8–11	1.0	3.0	1.2	4.0	1.5	5.0
12–15	1.1	3.5	1.4	4.5	2.4	8.0

Rational Method

The rational method is the most commonly used procedure for designing small drainage structures. The equation used is

$$Q = XCiA \quad (7.17)$$

where Q = peak rate of runoff, ft³/s or m³/s
 C = runoff coefficient
 A = area of drainage basin, acres or hectares
 i = rainfall intensity, in/h or cm/h
 X = 1.01 (0.0278 for SI units)

The rainfall intensity i is obtained from the intensity–duration–frequency curves described earlier with a rainfall duration equal to the time of concentration T_c of the basin.

The rational method assumes that the maximum flow will occur at the time when all of the runoff flows from the contributing watersheds reach the outlet. Rainfall durations longer than the time of concentration T_c will result in a lower peak flow since the average intensity will be less. Other assumptions of the rational method are

1. The rainfall intensity is constant over the watershed and throughout the storm.
2. The runoff coefficient does not vary throughout the storm.

The runoff coefficient C is the most difficult to estimate. Typical values of C are shown in Table 7.4. Wide ranges are given because of the effects of soils, rainfall intensity, duration, slope, and impervious area. The effects of recurrence interval on C are illustrated in Figure 7.19. Estimates of impervious area can be obtained from Table 7.5 or from aerial photographs.

Rawls et al. (37) developed a simplified procedure for determining C based on the earlier work of Ross-miller (36) and for including the effects of land use, slope, and soil type. Values of C developed by Rawls are provided in Table 7.6.

Hydrograph Procedures

In the design of many stormwater management facilities such as detention basins, the flow variation throughout a storm is as important as the instantaneous peak flow. This variation in runoff with time is defined as a hydrograph.

Storm Flow Volume. When generating the runoff hydrograph, the quantity of runoff must first be estimated. This volume is predicted by calculating the rainfall abstractions and subtracting these from the accumulated rainfall. The most important abstractions in stormwater management are interception, depression storage, and infiltration.

Interception. Interception is the segment of the total rainfall that wets and adheres to aboveground objects until it is returned to the atmosphere through evaporation. While seemingly small, interception can be very important in densely forested areas, where it can hold as much as 25% of the total annual precipitation (23).

Depression Storage. Depression storage is the portion of rainfall captured in small surface depressions. This quantity is normally small for developing areas but can be substantial for rural areas. The volume stored at any time V_s can be estimated using Eq. (7.18), presented by Linsley et al. (2).

STORMWATER MANAGEMENT

TABLE 7.4 Rational Equation Runoff Factors (33)

Description of area of surface	C factors
Business	
Downtown	0.70–0.95
Neighborhood	0.50–0.70
Residential	
Single-family	0.30–0.50
Multiunits, detached	0.40–0.60
Multiunits, attached	0.60–0.75
Residential (suburban)	0.50–0.70
Apartment	0.50–0.70
Industrial	
Light	0.50–0.80
Heavy	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard	0.20–41.35
Unimproved	0.10–0.30
Pavement	
Asphaltic and concrete	0.70–0.95
Brick	0.70–0.85
Roofs	0.75–0.95
Lawns, sandy soil	
Flat, 2%	0.05–0.10
Average, 2–7%	0.10–0.15
Steep, 7%	0.15–0.20
Lawns, heavy soil	
Flat, 2%	0.13–0.17
Average, 2–7%	0.18–0.22
Steep, 7%	0.25–0.35

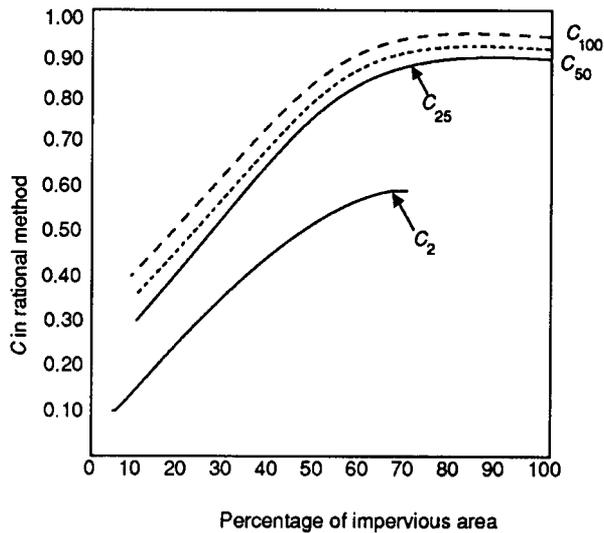


FIGURE 7.19 Effect of recurrence interval on runoff coefficient [Relation of C in the rational formula to percentage of impervious area. (Subscripts refer to storm recurrence interval)] (34).

7.21

TABLE 7.5 Impervious Area by Land Use (35)

Land use description	% impervious area
Residential	
Average single-family unit lot size	
$\frac{1}{8}$ acre (0.05 ha) or less	65
$\frac{1}{4}$ acre (0.10 ha)	35
$\frac{1}{3}$ acre (0.13 ha)	30
$\frac{1}{2}$ acre (0.2 ha)	25
1 acre (0.4 ha)	20
more than 1 acre	15
Apartments and townhouses	75
Schools and churches	50
Business and commercial	85
Industrial	70
Parks and cemeteries	15
Open or undeveloped	1

$$V = S_d(1 - e^{-Kp})$$

where S_d = the depression storage capacity of the basin

$$K = 1/S_d$$

These authors suggest a range of 0.25 in (0.64 cm) for S_d in pervious areas to 0.0625 in (0.16 cm) in impervious surfaces. Typical depression storage values for the Denver area are presented in Table 7.7.

Infiltration. Infiltration is the passage of water through the soil surface. The maximum rate that water can enter the soil is the infiltration capacity f_p which decreases rapidly from a maximum f_o , at the beginning of the storm to a constant rate f_c , as the soil becomes saturated. Horton described this decay by Eq. (7.19).

$$f_p = f_c + (f_o - f_c)e^{-Kt} \quad (7.19)$$

where t is time from beginning of rainfall, and K , f_c , and f_o depend on the soil (39). Numerous other equations have been developed for predicting infiltration capacity, all of which follow the same general decay curve. Some of these equations include other variables, such as accumulated rainfall; however, all suffer from the same weakness: the parameters of each equation require field measurement of infiltration rate and cannot be estimated from commonly available information, such as soil type and cover. An exception to this is the SCS curve number procedure.

SCS Curve Number Method. Details of the SCS curve number method procedures can be found in the following SCS publications:

1. NEH-4: "Hydrology," Sec. 4, *National Engineering Handbook* (10)
2. TR-55: "Urban Hydrology for Small Watersheds" (31)

A brief summary of this method is presented here.

The curve number method is the result of a long-term effort by the SCS to develop an easy tool for prediction of runoff rates and volumes. Extensive field studies were conducted in the 1930s and 1940s for differing land covers and soil types (40). Equation (7.20) was proposed by Mockus (10) and has come into general use as the accepted rainfall-runoff function.

TABLE 7.6 Runoff Coefficients for the Rational Formula by Hydrologic Soil Group and Slope Range (37)*

Land use	A			B			C			D		
	0–2%	2–6%	6%+	0–2%	2–6%	6%+	0–2%	2–6%	6%+	0–2%	2–6%	6%+
Cultivated land	0.08	0.13	0.16	0.11	0.15	0.21	0.14	0.19	0.26	0.18	0.23	0.31
	0.14	0.18	0.22	0.16	0.21	0.28	0.20	0.25	0.34	0.24	0.29	0.41
Pasture	0.12	0.20	0.30	0.18	0.28	0.37	0.24	0.34	0.44	0.30	0.40	0.50
	0.15	0.25	0.37	0.23	0.34	0.45	0.30	0.42	0.52	0.37	0.50	0.62
Meadow	0.10	0.16	0.25	0.14	0.22	0.30	0.20	0.28	0.36	0.24	0.30	0.40
	0.14	0.22	0.30	0.20	0.28	0.37	0.26	0.35	0.44	0.30	0.40	0.50
Forest	0.05	0.08	0.11	0.08	0.11	0.14	0.10	0.13	0.16	0.12	0.16	0.20
	0.08	0.11	0.14	0.10	0.14	0.18	0.12	0.16	0.20	0.15	0.20	0.25
Residential												
Lot size 1/8 acre (0.05 ha)	0.25	0.28	0.31	0.27	0.30	0.35	0.30	0.33	0.38	0.33	0.36	0.42
	0.33	0.37	0.40	0.35	0.39	0.44	0.38	0.42	0.49	0.41	0.45	0.54
Lot size 1/4 acre (0.10 ha)	0.22	0.26	0.29	0.24	0.29	0.33	0.27	0.31	0.36	0.30	0.34	0.40
	0.30	0.34	0.37	0.33	0.37	0.42	0.36	0.40	0.47	0.38	0.42	0.52
Lot size 1/3 acre (0.13 ha)	0.19	0.23	0.26	0.22	0.26	0.30	0.25	0.29	0.34	0.28	0.32	0.39
	0.28	0.32	0.35	0.30	0.35	0.39	0.33	0.38	0.45	0.36	0.40	0.50
Lot size 1/2 acre (0.2 ha)	0.16	0.20	0.24	0.19	0.23	0.28	0.22	0.27	0.32	0.26	0.30	0.37
	0.25	0.29	0.32	0.28	0.32	0.36	0.31	0.35	0.42	0.34	0.38	0.48
Lot size 1 acre (0.4 ha)	0.14	0.19	0.22	0.17	0.21	0.26	0.20	0.25	0.31	0.24	0.29	0.35
	0.22	0.26	0.29	0.24	0.28	0.34	0.28	0.32	0.40	0.31	0.35	0.46
Industrial	0.67	0.68	0.68	0.68	0.68	0.69	0.68	0.69	0.69	0.69	0.69	0.70
	0.85	0.85	0.86	0.85	0.86	0.86	0.86	0.86	0.87	0.86	0.86	0.88
Commercial	0.71	0.71	0.72	0.71	0.72	0.72	0.72	0.72	0.72	0.72	0.72	0.72
	0.88	0.88	0.89	0.89	0.89	0.89	0.89	0.89	0.90	0.89	0.89	0.90
Streets	0.70	0.71	0.72	0.71	0.72	0.74	0.72	0.73	0.76	0.73	0.75	0.78
	0.76	0.77	0.79	0.80	0.82	0.84	0.84	0.85	0.89	0.89	0.91	0.95
Open space	0.05	0.10	0.14	0.08	0.13	0.19	0.12	0.17	0.24	0.16	0.21	0.28
	0.11	0.16	0.20	0.14	0.19	0.26	0.18	0.23	0.32	0.22	0.27	0.39
Parking	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87	0.85	0.86	0.87
	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97	0.95	0.96	0.97

*First row of each entry gives runoff coefficients for storm recurrence intervals less than 25 years; second row gives runoff coefficients for storm recurrence intervals of 25 years or more.

TABLE 7.7 Typical Depression and Detention Values for Various Land Covers in the Denver Area (38)

Land cover	Centimeters	Inches
Large paved areas	0.13–0.30	0.05–0.15
Roofs, flat	0.25–0.75	0.1–0.3
Roofs, sloped	0.13–0.25	0.05–0.1
Grassed lawns	0.5–1.3	0.2–0.5
Wooded areas and open fields	0.5–1.5	0.2–0.6

$$Q = \frac{(P - la)}{(P - la) + S} \quad (7.20)$$

where Q = direct runoff
 P = total precipitation
 S = potential maximum retention
 la = initial abstraction

An empirical relationship between la and S was developed in order to eliminate the need to estimate both la and S (10).

$$la = 0.2S \quad (7.21)$$

When this relationship is combined with Eq. (7.20), the standard SCS rainfall function results.

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S} \quad (7.22)$$

The ratio of initial abstraction la to the potential maximum retention S has been questioned by Aron et al. (41), who suggest that la may frequently be no more than 10% of S . For the remainder of this discussion, the standard SCS relationship will be used, but the reader is cautioned that this relationship may not be applicable in all cases.

To allow easier interpolation between differing values of S , the SCS uses the transformation

$$CN = \frac{1000}{S} - 10 \quad (7.23)$$

CN , the curve number, is an index of the infiltration capacity of a specific soil-cover complex. Estimates of CN are based on antecedent moisture condition (AMC), hydrologic soil group, and land use. The hydrologic soil groups, as defined by SCS soil scientists, are

- A. *Low runoff potential*: Soils having a high infiltration rate even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels.
- B. Soils having a moderate infiltration rate when thoroughly wetted and consisting chiefly of moderately deep to deep, and moderately well- to well-drained soils with moderately fine to moderately coarse texture.
- C. Soil having a slow infiltration rate when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water or soils with moderately fine to fine texture.
- D. *High runoff potential*: Soils having a very slow infiltration rate when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over nearly impervious material.

Descriptions of soils and their hydrologic soil group can be found in most soil surveys. When the soil type is known, the hydrologic soil group may be obtained from Chapter 7 of NEH-4.

Estimates of CN for agricultural and developing areas with an average antecedent moisture condition can be found in Tables 7.8 and 7.9. Procedures for the determination of CN for forests can be found in Chapter 9, NEH-4.

In some cases, the analysis of dry (AMC-I) or wet (AMC-III) conditions preceding the storms may be of

TABLE 7.8 Runoff Curve Numbers for Selected Land Uses (3I)

Land use description	Hydrologic soil group			
	A	B	C	D
Cultivated land				
Without conservation treatment	72	81	88	91
With conservation treatment	62	71	78	81
Pasture or range land				
Poor condition	68	79	86	89
Good condition	39	61	74	80
Meadow, good condition	30	58	71	78
Wood or forest land				
Thin stand, poor cover, no mulch	45	66	77	83
Good cover	25	55	70	77
Open spaces, lawns, parks, golf courses, cemeteries, etc.				
Good condition: grass cover on 75% or more of the area	39	61	74	80
Fair condition: grass cover on 50–75% of the area	49	69	79	84
Commercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential				
Average lot size				
Average % impervious				
1/8 acre (0.05 ha) or less	65	77	85	90
1/4 acre (0.10ha)	38	61	75	83
1/3 acre (0.13 ha)	30	57	72	81
1/2 acre (0.2 ha)	25	54	70	80
1 acre (0.4 ha)	20	51	68	79
Paved parking lots, roofs, driveways, etc.	98	98	98	98
Streets and roads				
Paved with curbs and storm sewers	98	98	98	98
Gravel	76	85	89	91
Dirt	72	82	87	89

interest. In these cases, Table 7.10 should be used to adjust the *CN* to reflect the differing moisture conditions.

The definition of each moisture condition is as follows:

Condition I Soils are dry but not to wilting point; satisfactory cultivation has taken place.

Condition II Average conditions.

Condition III Heavy rainfall, or light rainfall and low temperatures, have occurred within the last 5 days; saturated soil.

Table 7.11 gives seasonal rainfall limits for the three antecedent moisture conditions.

Once *CN* is determined, the total runoff for a given rainfall is determined using Eq. (7.22) or Figures 7.20 and 7.21. When there are several differing land uses or soil types within a drainage basin, the runoff can be determined in one of two ways

1. For small differences in *CN* between land uses, a weighted-average *CN* may be used for the area.
2. For large differences in *CN*, runoff should be calculated separately for each area and then weighted to find the average for the total drainage basin.

TABLE 7.9 Runoff Curve Numbers for Hydrologic Soil-Cover Complexes (10)

Land use	Cover		Hydrologic soil group			
	Treatment or practice	Hydrologic condition	A	B	C	D
			Fallow	Straight row	—	77
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured	Poor	66	74	80	82
	Contoured and terraced	Good	62	71	78	81
	Contoured and terraced					
Small grain	Straight row	Poor	65	76	84	88
	Contoured	Good	63	75	83	87
	Contoured and terraced	Poor	63	74	82	85
		Good	61	73	81	84
		Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes* or rotation meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured and terraced	Poor	63	73	80	83
	Contoured and terraced	Good	51	67	76	80
Pasture or range	Contoured	Poor	68	79	86	89
	Contoured	Fair	49	69	79	84
	Contoured	Good	39	61	74	80
		Poor	47	67	81	88
		Fair	25	59	75	83
		Good	6	35	70	79
		Good	30	58	71	78
		Poor	45	66	77	83
Meadow						
Woods		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		—	59	74	82	86
Roads						
	dirt [†]	—	72	82	87	89
	hard surface [†]	—	74	84	90	92

*Close drilled or broadcast.

[†]Including right-of-way.

This distinction is necessary since the rainfall-runoff equation is nonlinear and a linear averaging of CN will yield erroneous results. The CNs listed in Table 7.8 are linear averages of pervious and impervious areas and should be used with caution because of this. Where precise delineation of flows is necessary, the runoff from pervious and impervious areas should be calculated separately then added.

When developing runoff hydrographs, accumulated rainfall at specific times T_i in the storm are tabulated. Total runoff R_i at these times is then calculated using the rainfall-runoff equation. The runoff rate is determined by calculating the difference between accumulated runoff at a point in time and the following time, and dividing this by the time interval.

TABLE 7.10 Curve Numbers and constants for the case $la = 0.2S (10)$

CN for condition II	CN for condition I	CN for condition III	S values,* in	Curve starts where $P = ,$ in	CN for condition II	CN for condition I	CN for condition III	S values,* in	Curve starts where $P = ,$ in
100	100	100	0	0	60	40	78	6.67	1.33
99	97	100	0.101	0.02	59	39	77	6.95	1.39
98	94	99	0.204	0.04	58	38	76	7.24	1.45
97	91	99	0.309	0.06	57	37	75	7.54	1.51
96	89	99	0.417	0.08	56	36	75	7.86	1.57
95	87	98	0.526	0.11	55	35	74	8.18	1.64
94	85	98	0.638	0.13	54	34	73	8.52	1.70
93	83	98	0.753	0.15	53	33	72	8.87	1.77
92	81	97	0.870	0.17	52	32	71	9.23	1.85
91	80	97	0.989	0.20	51	31	70	9.61	1.92
90	78	96	1.11	0.22	50	31	70	10.0	2.00
89	76	96	1.24	0.25	49	30	69	10.4	2.08
88	75	95	1.36	0.27	48	29	68	10.8	2.16
87	73	95	1.49	0.30	47	28	67	11.3	2.26
86	72	94	1.63	0.33	46	27	66	11.7	2.34
85	70	94	1.76	0.35	45	26	65	12.2	2.44
84	68	93	1.90	0.38	44	25	64	12.7	2.54
83	67	93	2.05	0.41	43	25	63	13.2	2.64
82	66	92	2.20	0.44	42	24	62	13.8	2.76
81	64	92	2.34	0.47	41	23	61	14.4	2.88
80	63	91	2.50	0.50	40	22	60	15.0	3.00
79	62	91	2.66	0.53	39	21	59	15.6	3.12
78	60	90	2.82	0.56	38	21	58	16.3	3.26
77	59	89	2.99	0.60	37	20	57	17.0	3.40
76	58	89	3.16	0.63	36	19	56	17.8	3.56
75	57	88	3.33	0.67	35	18	55	18.6	3.72
74	55	88	3.51	0.70	34	18	54	19.4	3.88
73	54	87	3.70	0.74	33	17	53	20.3	4.06
72	53	86	3.89	0.78	32	16	52	21.2	4.24
71	52	86	4.08	0.82	31	16	51	22.2	4.44
70	51	85	4.28	0.86	30	15	50	23.3	4.66
69	50	84	4.49	0.90					
68	48	84	4.70	0.94	25	12	43	30.0	6.00
67	47	83	4.92	0.98	20	9	37	40.0	8.00
66	46	82	5.15	1.03	15	6	30	56.7	11.34
65	45	82	5.38	1.08	10	4	22	90.0	18.00
64	44	81	5.62	1.12	5	2	13	190.0	38.00
63	43	80	5.87	1.17	0	0	0	∞	∞
62	42	79	6.13	1.23					
61	41	78	6.39	1.28					

*For CN in column 1.

Note: 1 in = 2.54 cm.

TABLE 7.11 Total Five-Day Antecedent Rainfall (2I)

AMC	Dormant season, in	Growing season, in
I	Less than 0.5	Less than 1.4
II	0.5–1.1	1.4–2.1
III	Over 1.	Over 2.1

Note: 1 in = 2.54 cm

$$\text{Runoff rate} = \frac{R_{i+i} - R_i}{t} \quad (7.24)$$

A typical calculation is shown in Table 7.12. The graph of the incremental runoff versus time (Figure 7.22) is the effective precipitation hyetograph—a required input for most watershed routing methods. A step-by-step procedure for calculating the effective precipitation hyetograph can be found in Chapter 16 of NEH-4.

Watershed Routing. In an earlier section, procedures for calculating the excess precipitation hyetograph were described. Watershed routing procedures are now necessary to describe the lagging and attenuation that will occur as this excess is routed through the watershed.

There are various procedures for routing hydrographs, ranging in sophistication from computer solution of the theoretical equations of flow to simpler linear attenuation of the hyetograph. Complete derivation of each is beyond the scope of this work, but brief summaries of the more important methods are given below.

Kinematic Wave. A kinematic wave is a type of unsteady flow where momentum is negligible. The slope of the bed is assumed equal to the friction slope. Attenuation occurs when excess precipitation builds up on the land surface essentially in the same manner as it would in a detention pond. The velocity and rate of runoff is controlled by the depth of flow, bed slope, and bed roughness, much as it would be in any open channel. Descriptions of the mathematics of the kinematic wave can be found in Ref. 42.

Unit Hydrograph. The unit hydrograph is essentially a typical hydrograph for a specific basin at a specific rainfall duration. To aid calculations, this hydrograph is adjusted until the volume under the hydrograph is equivalent to 1 in or cm of runoff. The shape of the hydrograph is assumed to depend solely upon watershed characteristics. The ordinate of the hydrograph at any time is assumed to be a linear function of the runoff volume. Because of this, unit hydrographs may be superimposed to reflect variations in runoff during a storm.

In order to calculate the outflow hydrograph for complex storms, the excess precipitation hydrograph is broken into increments of uniform rainfall (and, therefore, uniform runoff). Individual hydrographs for each rainfall excess are calculated by multiplying the hydrograph ordinate by the respective increment of rainfall excess. Individual hydrographs are lagged according to the time from the beginning of the storm to the rainfall increment. The total storm runoff hydrograph is obtained by adding the ordinates of each of the individual hydrographs. A typical hydrograph is shown in Figure 7.23.

The assumption that flow rate varies linearly with runoff volume is obviously invalid. The basic equations of flow, such as Manning's equation or the Dancy–Weisbach equation, show that flow and velocity vary nonlinearly with depth. Nevertheless, the assumption of linearity has been found to be adequate when the unit hydrograph has been derived from storms of the same general magnitude as the intended analysis. When studying extreme events, this assumption should be made with caution.

In stormwater management, gauging station records are rarely available to generate unit hydrographs for the specific basin to be studied. Synthetic unit hydrographs are usually generated to overcome this. These

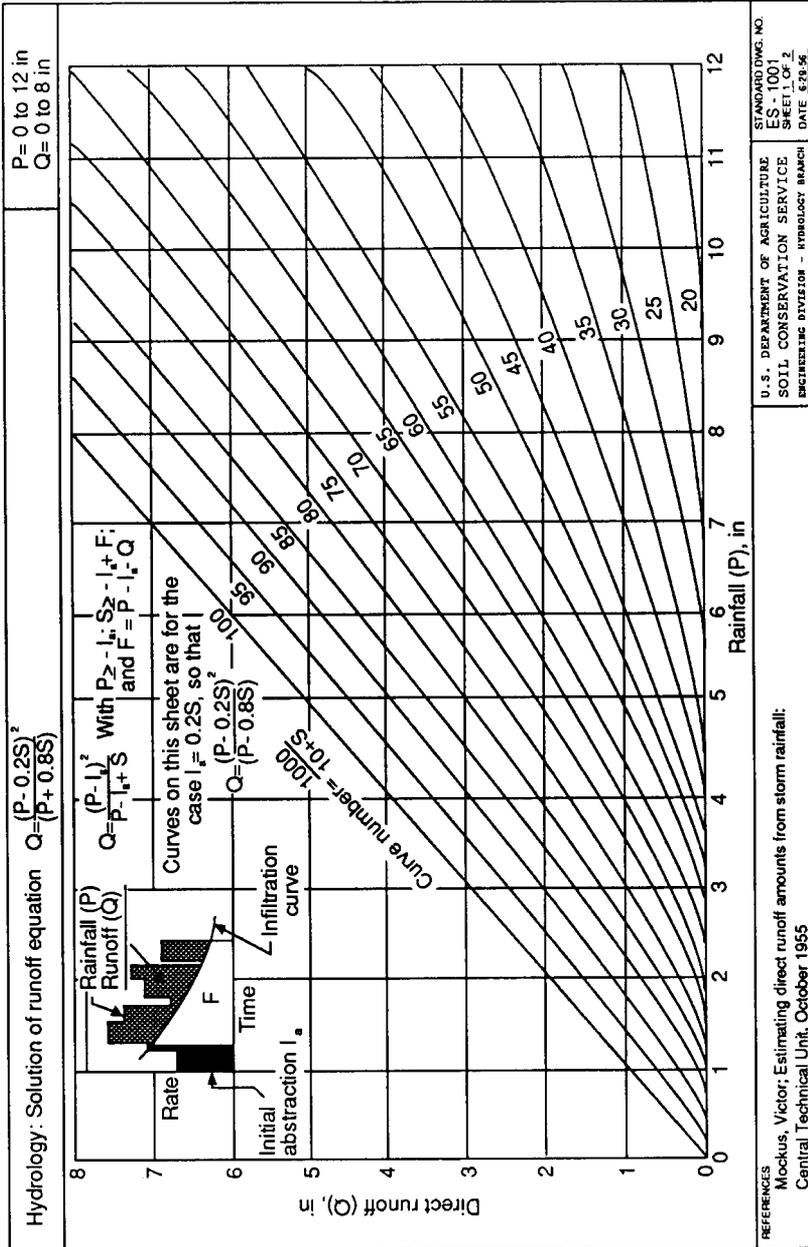


FIGURE 7.20 Hydrology: Solution of runoff equation (10).

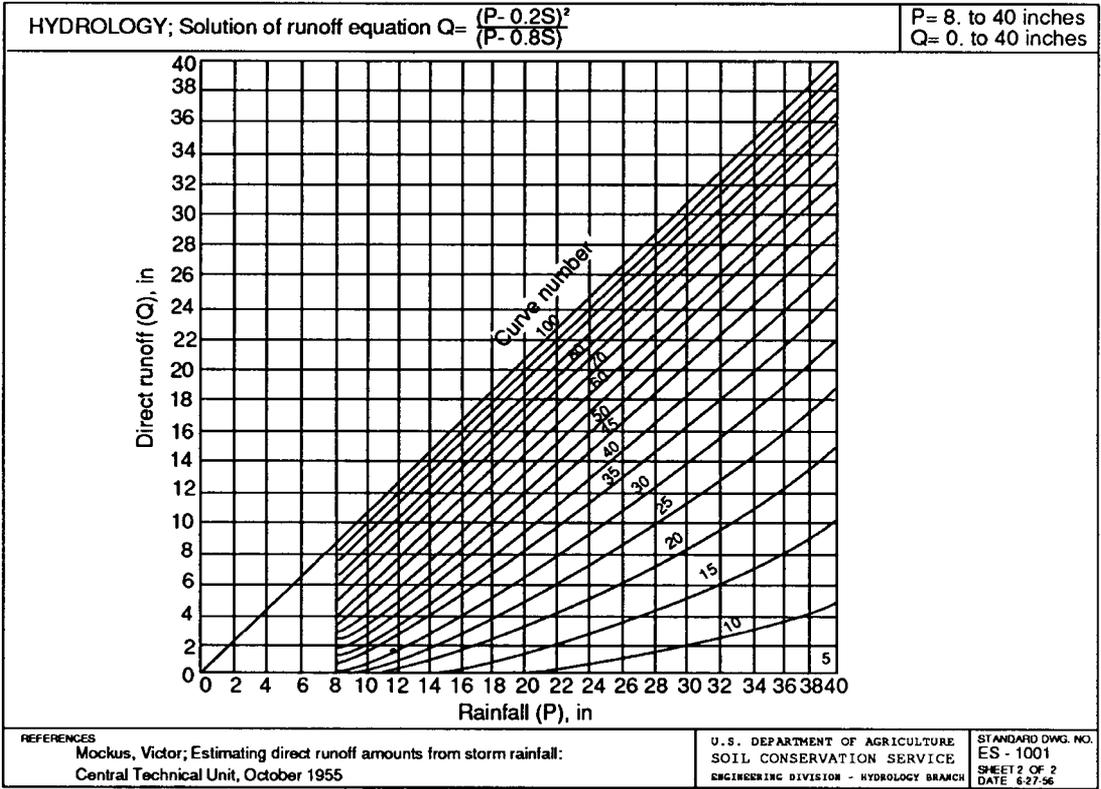


FIGURE 7.21 Hydrology: Solution of runoff equation (10).

TABLE 7.12 Calculation of Effective Precipitation Hydrograph, $CN = 80$

Time, h	Accumulated rainfall, in	Accumulated runoff, in	Incremental runoff, in	Runoff rate, in/h
0	0	0	0	0
1	0.2	0	0	0
2	0.3	0	0	0
2.2	0.5	0	0.39	3.90
2.3	1.7	0.39	0.57	5.70
2.4	2.6	0.96	0.44	4.40
2.5	3.2	1.40	0.24	0.48
3.0	3.5	1.64	0.32	0.32
4.0	3.9	1.96	0.16	0.16
5.0	4.1	2.12		

Note: 1 in 2.54 cm.

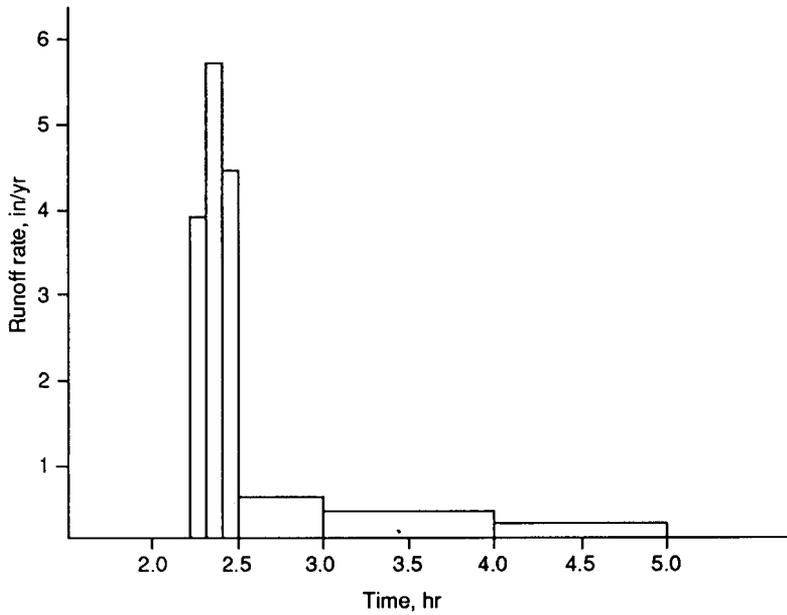


FIGURE 7.22 Effective precipitation hyetograph.

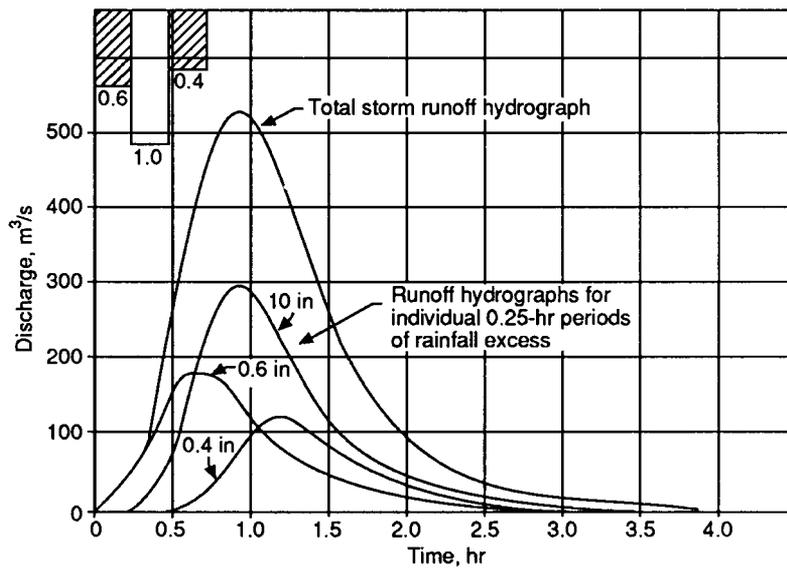


FIGURE 7.23 Combining hydrographs.

synthetic graphs are based on monitoring in other basins and time parameters that can be measured or calculated for the basin to be studied. The most commonly used graphs are the SCS and Espey hydrographs described in the following sections.

SCS TR-55. In TR-55, "Urban Hydrology for Small Watersheds," the SCS has generated outflow hydrographs for various combinations of watershed time of concentration T_c and channel travel time T_l using their computer program TR-20 (43). These hydrographs are based on the Type II storm distribution (Figure 7.12) and the SCS dimensionless unit hydrograph (Figure 7.24). Channel routing is performed using the convex method (43). The hydrographs have been tabulated for times of concentration from 0.1 to 2.0 h and channel travel times of 0 to 4.0 h (Table 7.13). Values are listed as cubic feet per second per square mile per inch total storm runoff (ft³/s/mi²/in); SI conversion is 0.0043 m³/s/km²/cm. The time ordinate is listed from 11 to 20 h because the most intense period of the Type II rainfall distribution is in the middle of the 24-h cycle.

When only the peak outflow from a site is needed, Figure 7.25 can be used.

The SCS dimensionless hydrograph (Figure 7.24) can be used to generate hydrographs for specific storms using the standard procedure described under unit hydrographs. Some of the characteristics of their hydrographs are listed below.

$$L = 0.6T_c \tag{7.25}$$

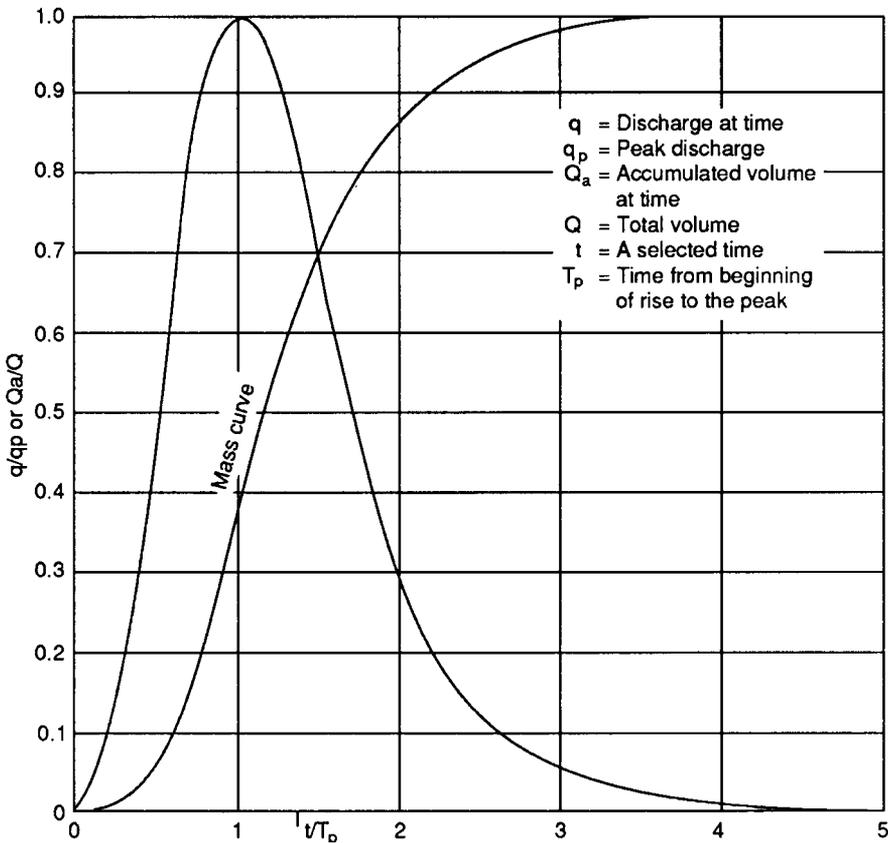


FIGURE 7.24 Dimensionless unit hydrograph and mass curve (10).

TABLE 7.13 Discharges for Type II Storm Distribution (3I)*

		Time of concentration = 0.1 h																							
		Hydrograph time, h																							
T_r		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	24	51	299	991	746	477	233	152	132	121	111	85	74	70	68	65	52	48	39	33	29	24	24	18	14
0.25	20	38	66	140	327	626	686	546	364	236	169	137	117	97	83	75	66	52	41	35	30	24	24	18	14
0.50	15	27	36	43	67	133	288	482	580	543	429	310	222	168	134	110	81	63	47	38	32	26	26	19	15
0.75	12	20	25	29	34	42	65	125	245	392	4%	515	452	360	273	206	127	80	53	42	35	27	19	15	
1.00	9	15	19	21	24	28	32	41	63	115	209	328	427	470	451	389	245	121	64	47	38	29	20	16	
1.50	6	10	12	13	14	16	17	19	22	25	29	38	56	92	154	236	410	360	133	66	47	33	21	16	
2.00	3	6	7	8	9	10	11	12	13	14	16	18	20	23	27	34	74	244	371	142	68	38	23	17	
2.50	2	4	4	5	5	6	7	7	8	9	10	11	12	13	15	16	21	41	243	343	150	48	26	19	
3.00	1	2	2	3	3	4	4	4	5	5	6	7	7	8	9	10	12	17	50	239	321	74	29	20	
3.50	0	1	1	1	1	2	2	2	3	3	4	4	4	4	5	6	6	7	10	17	59	304	159	33	21
4.00	0	0	0	0	0	1	1	1	1	1	2	2	2	2	3	3	4	5	6	10	18	67	290	39	23

		Time of concentration = 0.2 h																							
		Hydrograph time, h																							
T_r		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	23	47	208	509	7%	641	424	245	170	138	121	104	85	75	71	68	56	49	40	34	29	24	24	18	14
0.25	18	34	49	91	1%	419	603	627	486	341	235	173	138	114	96	83	70	55	43	36	31	25	18	15	
0.50	14	24	32	37	50	87	181	341	490	545	497	397	296	219	167	133	92	67	49	39	33	26	19	15	
0.75	11	18	23	26	30	36	49	84	161	284	409	491	481	422	340	263	157	89	56	43	36	27	19	15	
1.00	9	14	18	20	22	25	29	35	48	79	143	240	347	426	452	427	299	147	69	49	39	29	20	16	
1.50	5	9	11	12	13	14	16	18	20	23	26	32	43	67	110	176	330	399	159	72	50	33	22	17	
2.00	3	6	7	7	8	9	10	11	12	13	15	16	18	21	24	29	56	192	363	168	75	40	24	18	
2.50	1	3	4	5	5	6	6	7	7	7	8	9	10	11	12	13	15	19	33	200	337	174	51	26	19
3.00	0	2	2	2	3	3	4	4	5	5	6	6	7	8	8	9	11	15	40	203	316	32	29	20	
3.50	0	0	1	1	1	1	2	2	2	2	3	3	4	4	5	5	6	7	9	16	46	300	180	34	22
4.00	0	0	0	0	0	1	1	1	1	1	1	2	2	2	3	3	4	4	6	9	16	53	286	41	24

(continues)

TABLE 7.13 Discharges for Type II Storm Distribution (3I)* (continued)

		Time of concentration = 0.3 h																							
		Hydrograph time, h																							
T_r		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	21	43	141	324	586	658	535	372	251	184	148	124	102	86	77	71	61	51	41	34	30	24	18	14	
0.25	17	31	43	67	134	279	461	559	378	428	318	234	179	143	116	97	76	59	45	37	32	25	18	15	
0.50	13	22	29	34	42	65	124	238	378	479	499	447	363	281	216	168	110	74	51	41	34	26	19	15	
0.75	10	17	21	24	27	32	41	63	114	203	316	413	457	443	389	319	198	105	60	45	37	28	20	15	
1.00	8	13	16	18	20	23	26	31	40	60	103	176	269	358	415	426	344	182	77	51	41	30	20	16	
1.50	5	8	10	11	12	13	15	16	18	21	24	28	36	52	82	132	272	382	192	81	52	34	22	17	
2.00	3	5	6	7	8	8	9	10	11	12	14	15	17	19	21	25	44	151	351	198	85	41	24	18	
2.50	1	3	4	4	5	5	6	6	7	8	8	9	10	11	12	14	17	28	162	328	200	54	27	19	
3.00	0	1	2	2	3	3	3	4	4	4	5	6	6	7	8	9	10	14	33	169	309	94	30	20	
3.50	0	0	1	1	1	1	2	2	2	3	3	3	4	4	5	5	6	9	14	38	172	294	35	22	
4.00	0	0	0	0	0	0	1	1	1	1	1	2	2	2	3	3	4	5	9	15	43	281	42	24	

		Time of concentration = 0.4 h																							
		Hydrograph time, h																							
T_r		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	20	39	103	224	419	558	575	451	331	247	190	155	127	105	90	80	66	53	42	35	30	24	18	14	
0.25	15	28	38	54	98	196	343	467	508	464	380	295	228	180	145	119	87	64	47	38	32	26	19	15	
0.50	12	20	26	30	37	53	92	172	286	395	462	453	402	332	266	211	137	84	54	42	35	27	19	15	
0.75	10	16	19	22	25	29	36	51	85	150	242	338	407	429	406	356	241	128	65	47	38	29	20	16	
1.00	8	12	15	17	19	21	24	28	34	49	78	132	208	292	362	403	368	220	88	55	42	30	21	16	
1.50	5	8	9	10	11	12	14	15	17	19	22	25	31	43	65	102	220	365	224	93	56	35	22	17	
2.00	3	5	6	6	7	8	9	9	10	10	11	13	14	16	17	20	23	37	119	338	225	99	43	24	18
2.50	1	3	3	4	4	5	5	6	6	6	7	8	9	10	11	12	13	16	25	132	317	225	58	27	19
3.00	0	1	2	2	2	3	3	3	4	4	4	5	5	6	7	8	10	13	28	140	300	107	31	21	
3.50	0	0	1	1	1	1	2	2	2	2	3	3	3	4	4	5	6	8	13	32	146	286	36	22	
4.00	0	0	0	0	0	0	1	1	1	1	1	2	2	2	3	3	3	5	8	14	36	275	44	24	

		Time of concentration = 0.5 h																							
		Hydrograph time, h																							
T_r		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	18	36	80	166	301	433	496	474	395	309	242	194	158	130	109	94	75	57	43	36	31	25	18	15	
0.25	15	26	37	52	94	172	277	372	425	424	383	326	270	221	182	150	107	73	49	39	33	26	19	15	
0.50	12	20	25	30	38	58	101	169	252	327	374	385	366	329	285	241	169	103	59	44	36	27	19	15	
0.75	9	15	19	22	25	30	41	63	103	162	229	292	335	354	348	325	255	157	77	50	39	29	20	16	
1.00	7	12	15	17	19	21	25	31	43	66	103	153	210	264	304	327	317	231	109	61	44	31	21	16	
1.50	5	8	9	10	11	12	14	15	17	20	24	31	43	63	92	129	214	295	224	55	65	36	23	17	
2.00	3	5	6	6	7	8	9	10	11	12	13	14	16	19	23	30	38	143	271	216	120	46	25	18	
2.50	1	3	3	4	4	5	5	6	7	7	8	9	10	11	12	14	18	39	150	253	209	71	28	19	
3.00	0	1	2	2	2	3	3	4	4	4	5	5	6	7	7	8	10	15	48	154	239	126	32	21	
3.50	0	0	1	1	1	1	1	2	2	2	2	3	3	4	4	5	6	8	16	56	155	227	38	23	
4.00	0	0	0	0	0	0	1	1	1	1	1	1	2	2	2	3	4	5	9	19	63	217	52	25	

		Time of concentration = 0.75 h																							
		Hydrograph time, h																							
T_r		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0	15	29	~7	98	163	248	329	375	388	369	325	276	232	195	165	142	107	76	51	39	33	26	19	15	
0.25	12	21	29	39	61	100	158	227	291	336	355	348	321	285	247	212	156	103	62	44	36	27	19	15	
0.50	10	16	21	24	29	41	63	100	150	208	263	305	327	329	314	288	226	147	79	52	40	29	20	16	
0.75	8	13	16	18	20	24	30	43	65	98	142	192	239	278	303	311	286	208	107	63	45	31	21	16	
1.00	6	10	13	14	15	17	20	24	31	44	65	95	134	177	220	256	294	264	149	81	53	33	21	16	
1.50	4	6	8	9	10	11	12	13	14	16	19	23	31	42	60	83	147	269	248	152	85	40	23	17	
2.00	2	4	5	5	6	7	7	8	9	10	11	12	14	16	18	23	39	97	251	235	153	56	26	19	
2.50	1	2	3	3	4	4	4	5	5	6	7	7	8	9	10	11	15	28	107	218	236	91	29	20	
3.00	0	1	1	2	2	2	2	3	3	4	4	5	5	6	6	7	8	12	33	113	225	153	34	22	
3.50	0	0	1	1	1	1	1	1	2	2	2	2	3	3	3	4	4	5	7	13	39	117	215	44	24
4.00	0	0	0	0	0	0	0	0	1	1	1	1	1	1	2	2	2	3	4	7	15	45	207	63	26

(continues)

TABLE 7.13 Discharges for Type II Storm Distribution (3I)* (continued)

		Time of concentration = 1.0 h																							
		Hydrograph time, h																							
T_t		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0		13	24	45	66	107	155	211	258	301	313	316	301	277	247	217	188	146	102	64	46	36	27	19	15
0.25		10	18	24	32	45	68	102	146	193	238	272	293	299	293	275	252	200	139	81	54	41	29	20	16
0.50		8	14	17	20	24	32	46	68	99	136	178	219	251	274	284	283	254	187	105	65	47	31	21	16
0.75		7	11	13	15	17	20	25	33	46	67	94	128	165	202	233	256	273	236	140	82	55	33	21	16
1.00		5	9	11	12	13	15	17	20	25	33	46	65	90	121	154	187	240	262	183	107	66	37	22	17
1.50		3	5	7	7	8	9	10	11	12	14	16	19	24	31	43	58	103	185	244	181	110	48	24	18
2.00		2	3	4	4	5	6	6	7	8	8	9	10	11	13	15	18	29	69	182	230	178	70	27	19
2.50		1	2	2	3	3	3	4	4	5	5	6	6	7	8	9	10	12	21	77	178	219	114	31	21
3.00		0	1	1	1	1	2	2	2	3	3	3	4	4	5	5	6	7	10	25	83	210	172	39	22
3.50		0	0	0	0	1	1	1	1	1	2	2	2	2	3	3	3	4	6	11	29	88	202	52	25
4.00		0	0	0	0	0	0	0	0	1	1	1	1	1	1	2	2	2	4	6	12	33	195	77	28
		Time of concentration = 1.25 h																							
		Hydrograph time, h																							
T_t		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0		11	21	37	51	79	107	147	187	219	249	264	271	267	256	241	219	177	128	81	56	42	29	20	16
0.25		9	15	21	27	36	53	74	103	137	172	205	231	249	259	259	253	223	167	102	67	48	31	21	16
0.50		7	12	18	24	32	46	68	99	136	178	219	251	274	284	283	247	251	209	130	82	56	34	21	16
0.75		6	9	12	13	15	17	21	27	36	50	69	93	120	149	177	202	235	242	165	103	67	38	22	17
1.00		4	7	9	10	11	13	14	17	21	27	36	49	66	88	113	139	190	236	200	130	83	43	23	17
1.50		3	5	6	6	7	8	8	9	10	12	14	16	20	25	33	44	76	142	223	195	131	58	26	18
2.00		1	3	3	4	4	5	5	6	6	7	8	9	10	11	13	15	24	52	143	212	189	86	29	20
2.50		1	1	2	2	2	3	3	3	4	4	5	5	6	7	7	8	10	17	58	143	201	132	35	21
3.00		0	1	1	1	1	1	2	2	2	2	3	3	3	4	4	5	6	9	20	64	143	196	45	23
3.50		0	0	0	0	0	1	1	1	1	1	1	2	2	2	2	3	4	5	9	23	68	190	62	26
4.00		0	0	0	0	0	0	0	0	0	0	1	1	1	1	1	1	2	3	5	10	26	184	91	30

		Time of concentration = 1.5 h																							
		Hydrograph time, h																							
T_t		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0		10	18	31	42	57	81	105	133	164	192	209	227	235	236	236	225	201	153	99	68	50	32	20	16
0.25		8	13	17	22	30	41	57	76	99	125	153	178	199	215	227	230	224	188	122	82	58	36	21	16
0.50		6	10	13	15	18	22	30	40	54	72	94	118	143	167	188	204	224	214	152	99	6	39	22	17
0.75		5	8	10	11	13	15	18	22	29	39	52	69	89	111	134	157	194	219	182	122	82	44	23	17
1.00		4	6	8	9	10	11	12	14	17	22	29	38	50	66	84	105	148	198	214	150	100	50	24	18
1.50		2	4	5	5	6	7	7	8	9	10	12	14	17	21	26	34	58	109	191	204	149	70	28	19
2.00		1	2	3	3	4	4	4	5	5	6	7	8	8	10	11	13	19	40	112	184	197	102	33	20
2.50		0	1	1	2	2	2	3	3	3	4	4	5	5	6	6	7	9	14	45	114	190	147	40	22
3.00		0	0	1	1	1	1	1	1	2	2	2	3	3	3	4	4	5	7	16	49	115	184	53	25
3.50		0	0	0	0	0	0	1	1	1	1	1	1	2	2	2	2	3	4	8	18	53	178	74	28
4.00		0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	1	2	2	4	8	21	174	105	34

		Time of concentration = 2.0 h																							
		Hydrograph time, h																							
T_t		11.0	11.5	11.7	11.8	11.9	12.0	12.1	12.2	12.3	12.4	12.5	12.6	12.7	12.8	12.9	13.0	13.2	13.5	14.0	14.5	15.0	16.0	18.0	20.0
0		7	14	22	30	38	49	64	80	95	114	133	152	165	175	184	192	190	176	129	93	68	41	23	17
0.25		6	10	13	17	22	28	37	47	61	75	91	108	126	143	157	168	185	189	153	109	79	46	24	17
0.50		5	8	10	13	17	21	27	35	45	57	71	86	103	119	135	162	186	186	172	129	92	52	26	18
0.75		4	6	8	8	10	11	13	16	21	26	34	43	55	67	82	97	129	166	183	149	109	59	27	18
1.00		3	5	6	7	7	8	9	11	13	16	20	26	33	42	52	64	92	136	180	167	127	68	29	19
1.50		1	3	3	4	4	5	5	6	7	8	9	10	12	15	18	23	37	68	135	175	163	93	34	21
2.00		1	1	2	2	3	3	3	4	4	5	5	6	6	7	8	10	14	26	71	133	170	127	42	23
2.50		0	1	1	1	1	1	2	2	2	3	3	3	4	4	5	5	7	11	29	74	132	166	53	26
3.00		0	0	0	0	1	1	1	1	1	1	2	2	2	2	3	3	4	5	12	32	76	162	71	30
3.50		0	0	0	0	0	0	0	0	1	1	1	1	1	1	1	2	2	3	6	13	35	158	95	35
4.00		0	0	0	0	0	0	0	0	0	0	0	0	0	0	1	1	1	2	3	6	14	80	155	43

*Discharge given in ft³/s/mi²/in, which is equivalent to 0.0043 m³/s/km²/in.

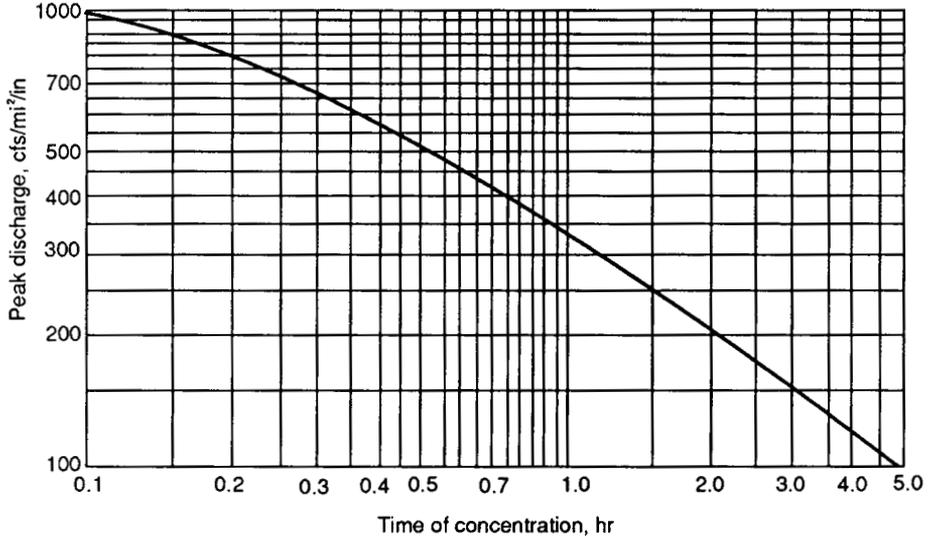


FIGURE 7.25 Peak discharge versus time of concentration T_c for 24-h, Type II storm distribution (1 ft³/s/mi²/in = 0.0043 m³/sec/km²/cm) (31).

$$T_p = L + \frac{D}{2} \tag{7.26}$$

and

$$T = 5T_p \tag{7.27}$$

- where L = lag time of the time from the centroid of the excess precipitation to the hydrograph peak
- D = duration of the increment of excess precipitation
- T_p = time from the beginning of rainfall excess to the hydrograph peak
- T_c = watershed time of concentration
- T = total time of direct runoff

In order to use this procedure, the dimensionless hydrograph must first be adjusted for the specific watershed. The SCS recommends that the increments of rainfall excess D be equal to approximately $0.133T_c$. Once D is defined, T_p is calculated by Eq. (7.26). The peak rate of runoff (q_p) for 1 in of rainfall excess is then defined by

$$q_p = \frac{484A}{t_p} \tag{7.28}$$

where A = watershed area in square miles

Derivations of these equations and step-by-step procedures can be found in NEH-4, Chapter 16.

Espey Unit Hydrograph. In studies of the urbanizing area around Houston, Texas, Espey developed a series of equations for generating unit hydrographs with an excess precipitation increment of 10 min (44). The parameters of these equations are as follows (Figure 7.26):

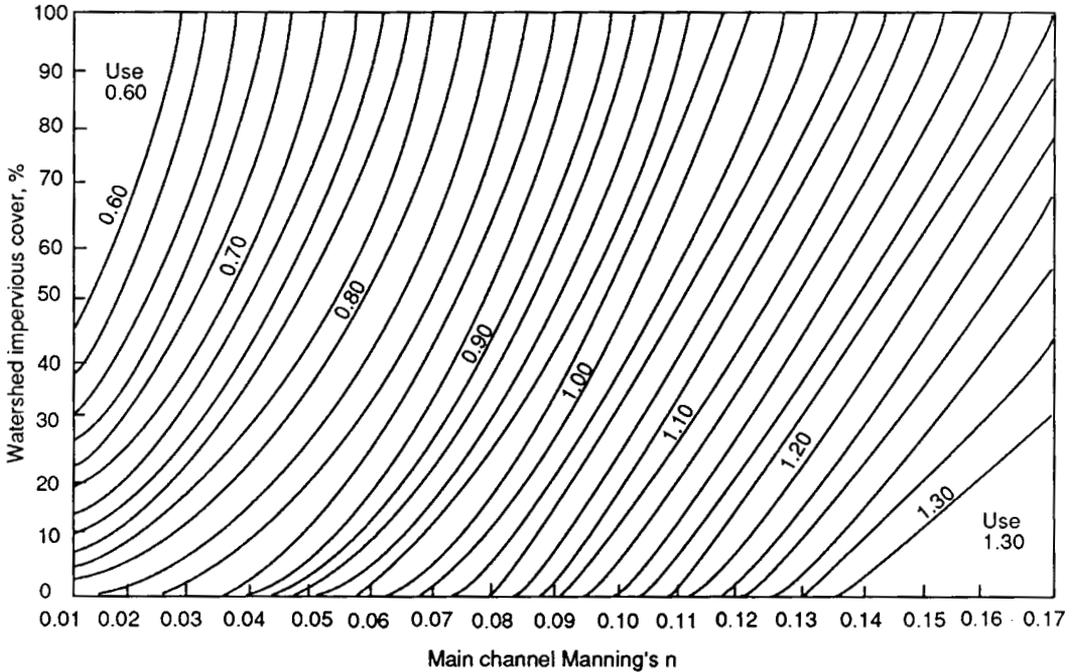


FIGURE 7.26 Watershed conveyance factor F (44).

T_r = time of rise, min (equivalent to T_p in SCS)

Q = peak discharge, ft^3/s (equivalent to q_p in SCS) ($1 \text{ ft}^3/\text{s} = 0.02832 \text{ m}^3/\text{s}$)

T_B = Time base, min (equivalent to T in SCS)

W_{50} = The time, min, between the two points on the unit hydrograph at which the discharge is $0.5Q$

W_{75} = The time, min, between the two points on the unit hydrograph at which the discharge is $0.75Q$.

These parameters are calculated by the following equations:

$$T_R = 3.1 L^{0.23} S^{-0.25} I^{-0.18} F^{1.57} \quad (7.29)$$

$$Q = 31,620A (0.96T_R) - 1.07 \quad (7.30)$$

$$T_B = 125,890AQ^{0.95} \quad (7.31)$$

$$W_{50} = 16,220A^{0.93} Q^{-0.92} \quad (7.32)$$

$$W_{75} = 3240A^{0.79} Q^{-0.78} \quad (7.33)$$

where L = length of the main stream, ft, from mouth to divide ($1 \text{ ft} = 0.3 \text{ m}$)

S = slope of main stream, ft/ft, from mouth to the point on the main stream that is $0.8L$ from the mouth

I = percent imperviousness

F = watershed conveyance factor from Figure 7.26

A = drainage area, mi^2 ($1 \text{ mi}^2 = 2.59 \text{ km}^2$)

Development of the unit hydrograph is accomplished by drawing a beginning hydrograph that meets the requirements of Eqs. (7.29) through (7.33). This hydrograph is then adjusted until the volume equals 1 in or cm over the entire drainage area (1 in or cm applied over the drainage area gives a volume of water in USCS and SI units, respectively). Storm outflow hydrographs are developed using superposition as described under unit hydrographs.

RIVER AND RESERVOIR ROUTING

Routing is the procedure used to predict the lagging and attenuation of a flood wave passing through a given river reach, reservoir, or detention pond. Routing techniques may be classified as either hydrologic or hydraulic. Hydrologic routing techniques, also known as storage routing, involve the balancing of inflow, outflow, and storage through the continuity equation. An additional relationship between storage and discharge must be established in order to employ hydrologic routing techniques. Hydrologic routing applications include flood prediction, evaluation of flood control measures, watershed simulation, reservoir design, and evaluation of the impact of urbanization. Hydrologic routing is the method most frequently used to size spillways for small, intermediate, and large dams. Many computer models are available which convert known amounts of precipitation into outflow hydrographs for a watershed and then route these hydrographs through complex river and reservoir networks using hydrologic routing procedures.

Hydraulic routing techniques are more complex and also more accurate than hydrologic methods. Cases that demand more sophisticated hydraulic techniques include backwater effects from downstream reservoirs and tributary inflows, as well as tidal bores, waves caused by sudden releases from reservoirs or dam failures, and occasionally flood waves caused by intense small-area storms, especially in channels of very flat slope (0.01 to 0.1%). These abrupt waves can generally be classified as dynamic waves. Dynamic waves are governed not only by the continuity equation but depend also on inertial influences. In contrast, kinematic waves are assumed to be free of inertial influences and are governed solely by the continuity equation and a stage-discharge function. A discussion of hydraulic routing techniques is beyond the scope of this chapter. The reader is referred to Chow (45), Henderson (46), or Viessman et al. (23) for a detailed explanation of these methods.

Hydrologic routing techniques can be readily performed manually. To perform a hydrologic routing, the continuity equation which relates inflow, outflow, and storage is solved

$$I - O = \frac{dS}{dt} \quad (7.34)$$

where I = inflow volume
 O = outflow volume
 S = storage volume
 t = time

By dividing t into n equal time increments dt , the continuity equation can be expressed in finite-difference form

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{t}$$

This form of the equation, together with either an analytical or assumed relationship between storage and discharge within the system, is the basis for most hydrologic routing techniques.

Channel Routing

Two hydrologic channel routing methods are presented in this section. The Muskingum method, presented first, is widely used and has been incorporated into many hydrological simulation models. A second method, based on the convex theory, has been used extensively by the SCS. Both methods can be calibrated to any stream or river reach. Routing with the convex method is simpler than the Muskingum method.

Muskingum Method. Hydrologic channel routing techniques are based on the premise that storage within a stable reach depends primarily on inflow, outflow, and the hydraulic characteristics of the channel. Storage within the reach at a given time can be expressed as

$$S = \frac{b}{A} [xI^{m/n} + (1-x)O^{m/n}] \quad (7.36)$$

where a and n reflect the mean stage-discharge relationship at the reach; b and m reflect the mean stage-volume characteristics; and x defines the relative weighting of inflow and outflow in determining storage in the reach. The Muskingum method assumes that $m/n = 1$, and lets $b/a = K$ (47). Equation (7.36) then takes the form

$$S = K[xI + (1-x)O] \quad (7.37)$$

the storage time constant K is the ratio of storage to discharge and has units of time. Its value is usually close to the travel time through the reach. The value of x ranges from 0 to 0.5, averaging about 0.2 for most natural streams. Setting x equal to 0.5 results in a pure translation of the flood wave in a uniform channel, while if $x = 0$, less time lag occurs along with greater attenuation of the flood wave.

Change in storage can be expressed from Eq. (7.37) as

$$S_2 - S_1 = K[x(I_2 - I_1) + (1-x)(O_2 - O_1)] \quad (7.38)$$

Equation (7.37) and the finite difference form of the continuity equation are combined to produce the Muskingum routing equation.

$$O_2 = C_0 I_2 = C_1 I_1 + C_2 O_1 \quad (7.39)$$

where

$$C_0 = \frac{-Kx + 0.5 \Delta t}{K - Kx + 0.5 \Delta t} \quad C_1 = \frac{Kx + 0.5 \Delta t}{K - Kx + 0.5 \Delta t} \quad C_2 = \frac{K - Kx - 0.5 \Delta t}{K - Kx + 0.5 \Delta t}$$

The coefficients K and t must have the same time units, and coefficients C_0 , C_1 , and C_2 should sum to 1.0. The routing operation is accomplished by solving Eq. 7.39 for successive time increments with O_2 becoming O_1 for the next period.

The Muskingum procedure involves computation of cumulative storage; therefore, the outflow at any time can only be determined by routing from the last known value at 0. If the routing period begins before the arrival of the flood wave, it can often be assumed that $I = 0$ for the initial conditions.

As stated previously, K is commonly estimated as the travel time through the reach, and x is typically assigned a value of 0.2. However, if inflow and outflow hydrograph records are available for the reach, better estimates of K and x can be determined. The method involves plotting the instantaneous storage volume S versus the weighted discharge $[xI + (1-x)O]$ for various values of x . The Muskingum method assumes that S and $[xI + (1-x)O]$ are linearly related by Eq. (7.37). The best value of x , therefore, is the one that gives the

most linear plot, i.e., the narrowest loop. The reciprocal of the slope of this plot is K from Eq. (7.37). Instantaneous values of storage are determined as S_2 in the finite-difference form of the continuity equation. S_1 can be assumed equal to O for the first time increment because only the slope of the plot and not the intercept is desired.

The Muskingum method of routing assumes a uniform, unbroken surface profile and that K and x remain constant throughout the reach. If these assumptions are severely violated, it may be necessary to work with shorter reaches or turn to a more sophisticated approach. A more complete discussion and examples of channel routing using the Muskingum method can be found in Refs. 45 through 48.

Convex Method. The convex method of stream flow routing is the procedure currently used in the SCS computer simulation model. The method is based on the following routing equation:

$$O_2 = (1 - C)O_1 + CI_1 \quad (7.40)$$

where C is the routing coefficient ($0 \leq C \leq 1$). This is the working equation for the method. Equation (7.40) can be solved for C to yield

$$C = \frac{O_2 - O_1}{I_1 - O_1} \quad (7.41)$$

It can be determined by similar triangles that

$$\frac{O_2 - O_1}{t} = \frac{I_1 - O_1}{K} \quad (7.42)$$

where K is the reach travel time. Therefore,

$$C = \frac{\Delta t}{K} \quad (7.43)$$

Several useful methods exist for estimating the routing coefficient C . Using Eq. (7.43), Δt can be estimated as one-fifth of the time to peak of the unit hydrograph; reach travel time can be estimated using a channel velocity derived from Manning's equation. A more accurate method would be to estimate the reach travel time by on-site measurement. With K known, a value of Δt could be selected which would then determine C . Studies have also shown that the routing coefficient C is approximately equal to twice the weighting factor x used in the Muskingum method. A final alternative is to use the empirical relationship

$$C = \frac{V}{V + X} \quad (7.44)$$

where V = channel velocity, ft/s or m/s
 $X = 1.7$ ft/s (0.52 m/s)

Once the value of C has been established by any of these methods, the computations for the outflow hydrograph proceed rapidly.

With C and K from Eq. (7.43) established, there is only one permissible routing interval, Δt . This routing interval may be inconvenient if it is an awkward fraction of an hour or if it does not fit the given hy-

drograph. Too large a routing interval will not accurately define the inflow hydrograph, and too small an interval will needlessly increase the amount of work required. Generally, the selected time interval should not exceed one-fifth of the time from the beginning of rise to the peak discharge of the inflow hydrograph. The end point of a time interval should fall at or near the inflow peak time and any other large change in flow rate. The value of t may be adjusted to t^* to meet these conditions; however, other adjustments must be made in order for the method to remain accurate. When a rough approximation of the routing effect is desired and manual computation is performed, C may be modified to C^* by the following equation:

$$C^* = 1 - (1 - C) \frac{[\Delta t^* + 0.5\Delta t]}{1.5 \Delta t} \quad (7.45)$$

where C^* is the modified routing coefficient for use with Δt^* .

Channel routing with the Convex Method produces a slightly different outflow hydrograph than methods based upon the continuity equation. This is chiefly because outflow resulting from a flood wave begins one routing interval Δt after inflow begins, which can be expected because the wave requires time to traverse the reach. Also, the peak rate of outflow does not fall on the recession limb of the inflow hydrograph, but as in all routing methods, maximum storage within the reach occurs when the inflow hydrograph intersects the outflow hydrograph.

Inflow I_2 does not appear in the working equation for the Convex Method as it does for other techniques. For this reason, the Convex Method can be used to predict an outflow rate at the end of a time interval without knowing what the inflow rate will be at that time. This predictive feature is more important for large rivers than for small streams because the routing interval for reaches of such streams is usually short. A more complete discussion of the Convex Method with examples can be found in Ref. 10.

Reservoir Routing

A flood wave passing through a reservoir is both delayed and attenuated. Water is released from storage gradually as pipe flow through principal spillways, in which case the discharge equation takes the form

$$O = CAH^x$$

where C = the pipe discharge coefficient
 A = the cross-sectional area of the pipe
 H = the head above the outlet
 x = an exponent (theoretically $\frac{1}{2}$)

During larger flows, water is released over emergency spillways, typically a weir, in which case the discharge equation takes the form

$$O = CYH^x$$

where C = the discharge coefficient for the spillway (approximately 3.0)
 Y = the length of the spillway crest
 H = head above the crest
 x = an exponent (theoretically $\frac{3}{2}$)

If a horizontal pool surface is assumed, which is valid for most reservoirs, storage and discharge depend only on the water surface elevation at the dam or spillway. This assumption is not valid for cases where the

backwater effect causes a significant percentage of the temporary storage to occur as wedge storage above a horizontal plane extending upstream from the water surface elevation at the dam. In most cases a level pool surface can be assumed. This simplifies the routing procedure because then both storage and discharge depend only on stage and can be considered independent of inflow.

The storage indication method is the most commonly used technique for routing hydrographs through detention ponds and reservoirs. To use this technique, stage-discharge and stage-storage curves must be developed (Figures 7.27 and 7.28). The stage-discharge curve is obtained by calculating the combined discharge through any orifices, weirs, deep inlets, or channels for various stages.

Flow passing through a drop inlet or morning glory spillway experiences three phases of flow control: weir, orifice, and as backwater from the discharge conduit. Flow entering the inlet will always be the minimum value of the three possible flow conditions.

Weir flow in a drop inlet can be calculated using the equation:

$$Q = XD_r H^{3/2} \quad (7.46)$$

where $Q = 9.739$ (5.377 for SI units)

X = head measured from the crest of the inlet, ft or m

D_r = diameter of the inlet, ft or m

H = discharge through the drop inlet, ft³/s or m³/s

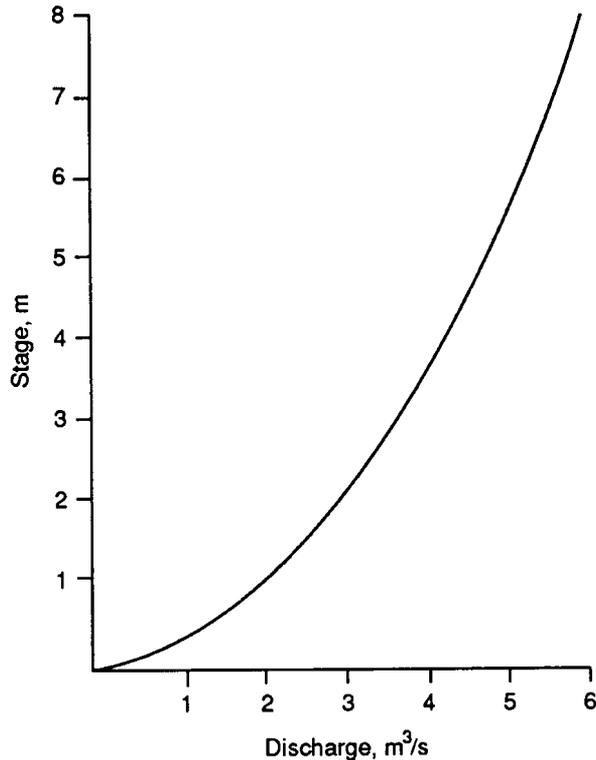


FIGURE 7.27 Stage: discharge curve.

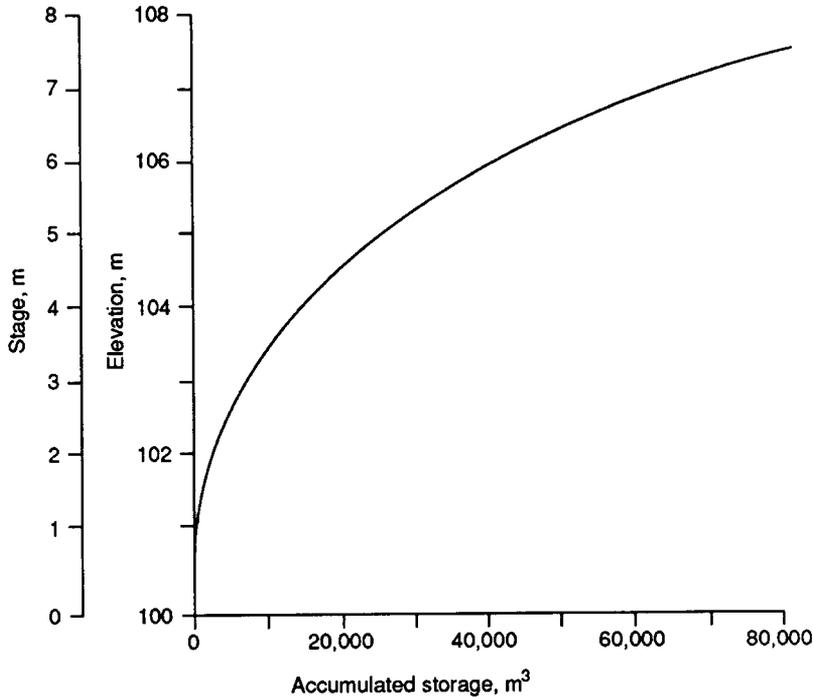


FIGURE 7.28 Stage-storage curve.

Orifice flow can be calculated using the equation

$$Q = XD_r^2 H^{1/2} \quad (7.47)$$

where $X = 3.783$ (2.088 in SI units)

The stage-storage curve is obtained by: (1) measuring the area enclosed by each topographic contour, (2) calculating the storage between contours, then (3) calculating the cumulative storage.

A storage-indication curve (Figure 7.29) should then be developed by plotting

$$\frac{2S}{\Delta T} \text{ versus } O$$

where S = accumulated storage, ft^3 or in^3 , at stage h

O = discharge in ft^3/s or m^3/s at stage h

T = routing time increment, s

The routing time increment should be selected to include at least five points on the rising limb of the inflow hydrograph. It is usually most convenient to use the same T as the method for generating the inflow hydrograph (in TR-55, the time increment is 0.1 h).

The routing calculation is commonly performed using the procedure outlined below and illustrated in Table 7.14. (for the first iteration, S is commonly assumed equal to the normal pool volume and O equal to the baseflow rate.)

STORMWATER MANAGEMENT

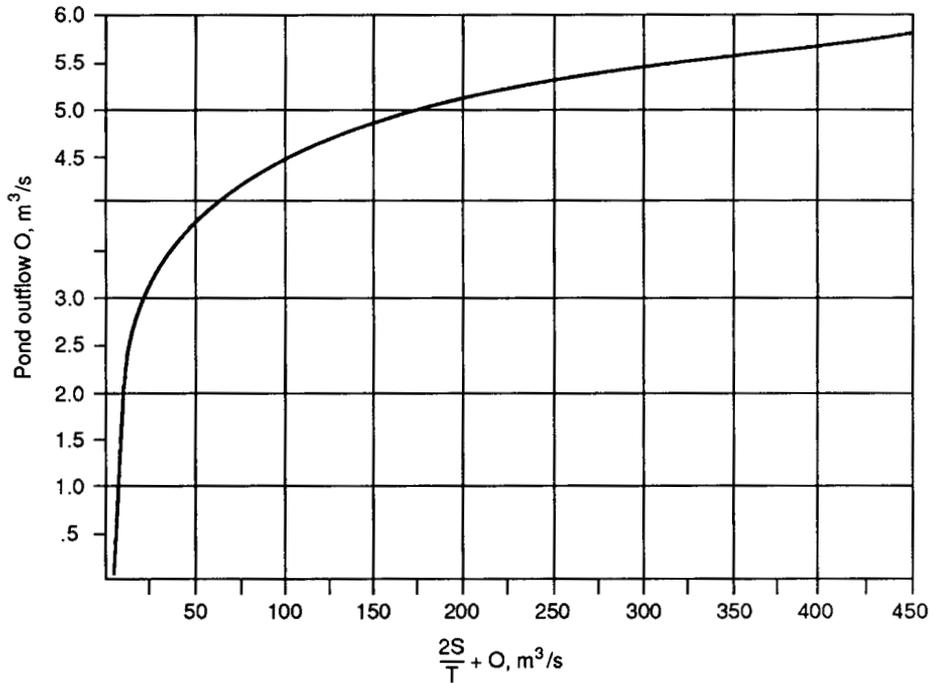


FIGURE 7.29 Storage indication curve.

TABLE 7.14 Example Hydrograph Routing Calculations

Time T , h	Inflow hydrograph $I, m³/s$	$\frac{2S}{T} - O$, $m³/s$	$\frac{2S}{T} - O$, $m³/s$	Outflow hydrograph $O, m³/s$	Comments
0	0				For first iteration, assume $\left(\frac{2S}{T} + O\right)_{T=0} = 0$
0.1	3.9	1.9	3.9	1.0	
0.2	13.4	13.2	19.2	3.0	
0.3	22.8	42.0	49.4	3.7	
0.4	25.6	131.0	140.4	4.7	
0.5	56.9	252.9	263.5	5.3	
0.6	36.4	335.2	346.2	5.5	
0.7	17.8	378.2	389.4	5.6	
0.8	11.6	396.2	407.6	5.7	
0.9	10.1	406.5	417.9	5.7	
1.0	9.2	414.2	425.8	5.8	
1.1	8.5	420.3	431.9	5.8	
1.2	6.5	423.7	435.3	5.8	
1.3	5.6	424.0	435.8	5.9	Peak discharge equals 5.9 m³/s
1.4	5.3	423.3	434.9	5.8	
1.5	5.2	422.2	433.8	5.8	
1.6	5.0	420.8	432.4	5.8	

Note: 1 ft³/s = 0.028 m³/s

1. Calculate quantity $\left(\frac{2S}{\Delta T} + O\right)$ for first iteration.

$$\left(\frac{2S}{T} + O\right)_{T=t+1} = \left(\frac{2S}{T} + O\right)_{T=t} + (I)_{T=t} + (I)_{T=t+1}$$

For the first iteration of Table 7.14, $t = 0$ h, $t + 1 = 0.1$ h

2. Find pond outflow using the storage-indication curve (Figure 7.29).

$$\left(\frac{2S}{T} + O\right) = 3.9 \text{ m}^3/\text{s} \text{ (138 ft}^3/\text{s)}$$

from routing curve, outflow $O = 1.0$

3. Calculate $\left(\frac{2S}{T} - O\right)_{0.1} = (3.9) - 2 \times 1.0 = 1.9 \text{ m}^3/\text{s} \text{ (67 ft}^3/\text{s)}$

$$\left(\frac{2S}{T} - O\right)_{T=t+1} = \left(\frac{2S}{T} + O\right)_{T=t+1} - (2 \times O)_{T=t+1}$$

$$\left(\frac{2S}{T} - O\right)_{0.1} = \left(\frac{2S}{T} + O\right)_{0.1} - (2 \times O)_{0.1}$$

$$\left(\frac{2S}{T} - O\right)_{0.1} = (3.9) - 2 \times 1.0 = 1.9 \text{ m}^3/\text{s}$$

4. Repeat process for $T = t + 2$

NONPOINT SOURCE POLLUTION

Water pollution control has traditionally focused on the impacts of pollutant discharges from point sources, such as industrial and municipal sewage systems. With the reductions in point source problems, other water pollutant sources have become the subject of closer technical and political scrutiny.

These other sources, called *nonpoint sources*, generate pollutants during the stormwater runoff process. Runoff erodes or washes off pollutants from a wide, diffuse area and transports them to a receiving water. Urbanization, agriculture, and silviculture are good examples of activities that cause nonpoint source pollutants.

A definition of nonpoint source pollution includes these elements:

- Nonpoint sources are diffuse, cover substantial areas, and act either in response to human activity or as “background pollution” from natural lands.
- Nonpoint source pollution is related to land management, geologic, and hydrologic variables which can change from day to day or year to year. Only the land management factors can be controlled by society.
- Nonpoint sources are generated and transported as part of the hydrologic cycle. Surface runoff transports eroded soil particles from pervious areas. It also picks up and transports pollutants deposited on impervious areas. Groundwater transports pollutants from septic tanks and landfills.

Point versus Nonpoint Sources

One reason for the lack of nonpoint source control lies in the different nature of nonpoint sources compared to point sources. Point sources are relatively easy to treat and to monitor for compliance. Conventional pollution control processes are designed for relatively concentrated, steady flows that usually vary by within less than one order of magnitude. Nonpoint sources are much more diffuse (except perhaps in dense urban areas) and are driven by random, intermittent precipitation events. The flow rate and quality of water often vary over several orders of magnitude. End-of-pipe solutions for nonpoint sources are ineffective and are not cost-effective.

Types of Nonpoint Sources. The many types of nonpoint sources produce an additional variable that affects the generation and control of nonpoint source pollutants. The EPA has developed a general list of nonpoint sources that includes urban runoff, construction, hydrologic modification, silviculture, mining, agriculture, irrigation return flows, solid waste disposal, atmospheric deposition, streambank erosion, and individual sewage disposal. From this group, the major national problems were defined as urban runoff and agricultural activities (50).

Urban runoff contains almost every type of water pollutant, including suspended solids, bacteria, heavy metals, oxygen-demanding substances, nutrients, and oil and grease. The sources of these pollutants are animal wastes, vehicles, street construction and salting, and erosion. Densely populated areas produce the highest loads.

The primary agricultural pollutants are pesticides, sediments, nutrients, organic material, and pathogens. The amount of pollutant generation is related to tillage practices and fertilizer and pesticide applications.

Silviculture activities produce large amounts of sediments from deforestation and from construction of logging roads. Oxygen-demanding organics and nutrients are carried to receiving waters with the sediments. The change in water quality after logging is often dramatic because of the extremely low background pollutant export from forested areas.

Other nonpoint sources include streambank erosion and atmospheric deposition. Streambank erosion is prevalent in watersheds where annual runoff volumes have increased over historical volumes. Urbanized areas and basins exposed to large-scale forestry activities are the most common examples of basins experiencing increased runoff. Atmospheric deposition was designated by the Council on Environmental Quality (CEQ) in 1979 as a major nonpoint pollution problem with respect to PCBs, lead, and phosphorus (50). Atmospheric deposition in the form of acid rain was recognized as one of the major environmental problems of the 1980s.

Magnitude of Pollutant Sources. The EPA reported that 87% of the 246 major water basins in the country are affected by nonpoint source pollutants (50). Table 7.15 indicates the estimated contributions of various nonpoint source pollutants. Other indications of the magnitude of the nonpoint source problem are (49):

- Sediment from nonpoint sources is estimated to be 360 times the point source load, and agricultural activities contribute over 50% of this nonpoint load.
- Biochemical oxygen demand (BOD) and nutrient loads from nonpoint sources are five to six times loads from point sources.
- BOD loads from urban runoff are probably equal to or larger than point source BOD loads.
- Nonpoint sources are responsible for about 80% of the total nitrogen and 50% of the total phosphorus load to the nation's receiving waters.
- Nonpoint sources produce over 98% of the fecal and total coliform counts in receiving waters.
- Toxic metal concentrations have been found to be higher in urban runoff than in sewage effluent.

TABLE 7.15 Annual Nonpoint Source Loadings (51)^{a,b}

Nonpoint source category	Sediment	BOD	Nitrogen	Phosphorus	Acids ^c	Salinity ^d
Crop land	1870	9	4.3	1.56	—	57.3
Pasture and range	1220	5	2.5	1.08	—	—
Forest	256	0.8	0.39	0.089	—	—
Construction	197	—	—	—	—	—
Mining	59	—	—	—	3.1	—
Urban runoff	20	0.5	0.15	0.019	—	—
Rural roadways ^e	2	0.004	0.0005	0.001	—	—
Small feedlots	2	0.05	0.17	0.032	—	—
Landfills	—	0.3	0.026	—	—	—
Subtotal	3626	15.8	7.4	2.8	3.1	57.3
“Natural background”	1260	5.0	2.5	1.1	—	—
Total	4886	20.8	10.0	3.8	3.1	57.3

^aEstimated pollutant contributions to surface waters from selected nonpoint sources in the contiguous 48 states (average pollution loads in million tons per year).

^b207 million acres (83.8 Mha) in public lands (14% of contiguous U.S.) mostly in Rocky Mountain region, were excluded due to inadequacy of information.

^cAs CaCO₃.

^dFrom irrigation return flow.

^eDeposition from traffic-related sources.

Description of Nonpoint Sources

Three basic processes are responsible for the generation of pollutants during the runoff process:

- Atmospheric deposition of airborne pollutants occurs during wet and dry periods.
- Accumulation of pollutants occurs on impervious areas until they are washed away by stormwater runoff.
- Erosion acts on nonpervious areas, dislodging and transporting pollutants to receiving waters.

These processes are controlled by land use factors, meteorological factors, and geological and geographic factors. Of these factors, land use is the only one that can be controlled.

Most urban watersheds are dominated by the accumulation and wash-off process, depending mostly on the percentage of impervious area (a key land use factor). Rural watersheds are largely controlled by the erosion process, which is influenced primarily by landcover, soil types, and slope. Because of these differences, nonpoint sources are usually divided into two main groups, urban and rural sources, where different assessment methods and predictive tools are used.

Urban Sources. The Nationwide Urban Runoff Program (NURP) study was conducted to facilitate understanding of the nature of urban runoff from residential, commercial, and industrial areas and was the largest study of stormwater undertaken to date. One focus of the NURP study was to characterize the water quality of discharges from storm sewer systems that drain residential, commercial, and light industrial (industrial parks) sites. Stormwater samples from 81 residential and commercial properties in 22 urban/suburban areas nationwide were collected and analyzed during a 5-year period, between 1978 and 1983. The majority of samples collected in the study were analyzed for eight conventional pollutants and three metals (114, 115).

Data collected under the NURP study indicated that discharges from separate storm sewer systems draining runoff from residential, commercial, and light industrial areas carried more than ten times the level of total suspended solids (TSS) on an annual loading basis as discharges from municipal wastewater treatment plants providing secondary treatment. Runoff from residential and commercial areas carried somewhat higher annual loadings of chemical oxygen demand (COD), total lead, and total copper than secondary effluent.

Discharges associated with urban runoff are highly intermittent and short-term loadings may have shock-loading effects on receiving water, such as low dissolved oxygen levels. Also, NURP study findings showed that fecal coliform counts in urban runoff are typically in the tens to hundreds of thousands per hundred milliliter of runoff during warm weather conditions. Monitoring data summarized in the NURP study provide important information about urban runoff from residential, commercial, and light industrial areas. The NURP study also recognized that the quality of urban runoff can be adversely affected by several sources of pollution that were not directly evaluated, including illicit discharges, construction site runoff, and illegal dumping. In some cases, storm sewer outfalls discharging during substantial dry periods had pollutant levels high enough to significantly degrade receiving water quality.

Urban stormwater pollutant concentrations range from the quality of drinking water to that of raw sewage (Table 7.16). This is due to the many pollutant sources in the urban environment (Table 7.17).

These specific sources are related to either soil erosion or the accumulation–washoff process. Highly impervious areas have little exposed soil, thus erosion may be relatively minor. Pollutant accumulation, usually more significant than erosion in urban areas, is very rapid immediately after street sweeping or rain events for the first 2 or 3 days and then slows down (Figure 7.30).

The percentage of impervious area declines for less intense land uses. The degree of imperviousness in a basin can be determined directly for small basins from planning maps; estimates for large basins can be calculated using impervious approximations by land use (Table 7.5).

For less dense areas, erosion becomes more important and can overshadow the accumulation–washoff process. The increased runoff volumes and velocities from urban areas also increase erosion in natural

TABLE 7.16 Characteristics of Urban Stormwater (52)

Parameter	Range		
BOD	1	—	700 mg/L
TOC	1	—	150 mg/L
COD	5	—	3,100 mg/L
SS	2	—	11,300 mg/L
Total solids	200	—	14,600 mg/L
Volatile total solids	12	—	1,600 mg/L
Settleable solids	0.5	—	5,400 mg/L
Organic N	0.01	—	16 mg/L
PKN	0.01	—	4.5 mg/L
NH ₃ N	0.1	—	2.5 mg/L
NO ₃ N	0.01	—	1.5 mg/L
Soluble PO ₄	0.1	—	10 mg/L
Total PO ₄	0.1	—	125 mg/L
Chlorides	2	—	25,000 mg/L
Oils	0	—	110 mg/L
Phenols	0	—	0.2 mg/L
Lead	0	—	1.9 mg/L
Total coliforms	200	—	146 × 10 ⁶ /100 ml
Fecal coliforms	55	—	112 × 10 ⁶ /100 ml
Fecal streptococci	200	—	1.2 × 10 ⁶ /100 ml

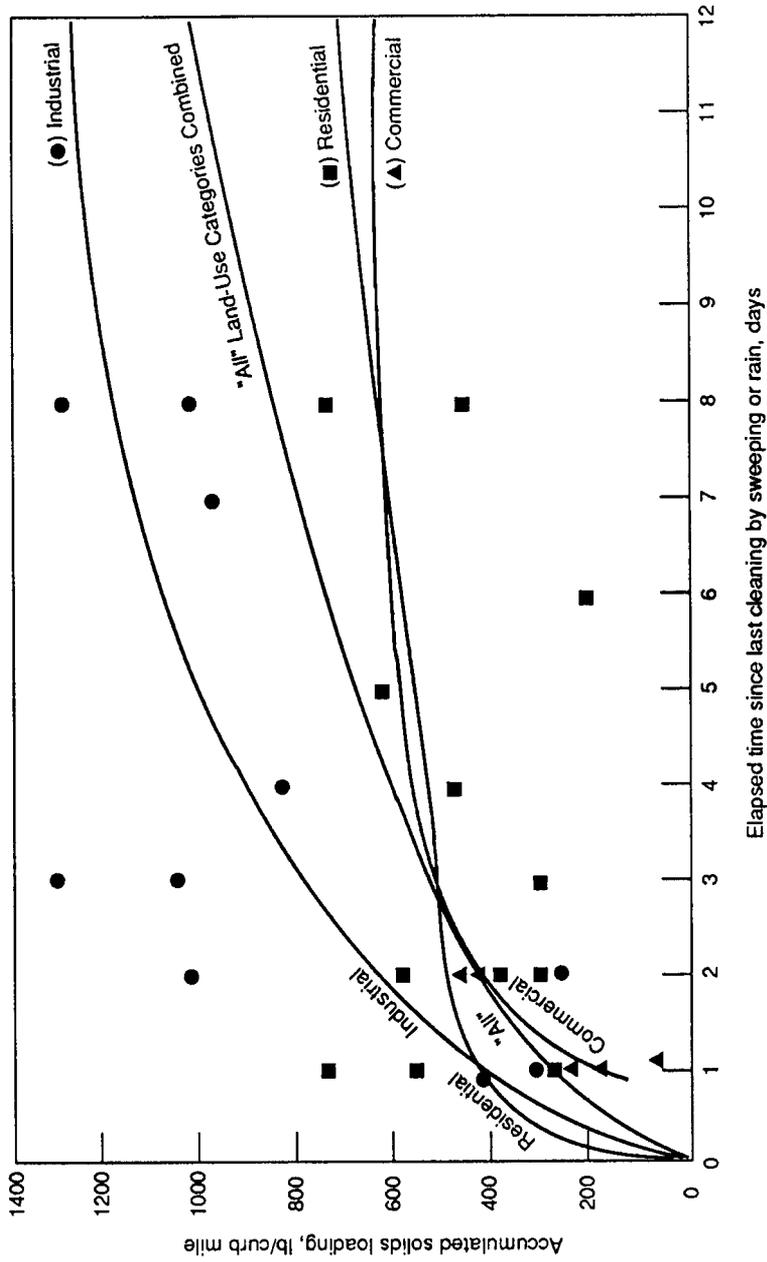


FIGURE 7.30 Accumulation of pollutants by land use (54).

TABLE 7.17 Primary Sources of Urban Runoff Pollutants

Source	Pollutant				
	BOD	Suspended solids	Nutrients	Heavy metals	Bacteria
Animal droppings	X		X		X
Atmospheric fallout	X		X	X	
Land erosion	X	X		X	
Lawn runoff	X		X		
Pavement runoff	X	X		X	

streams in the drainage network. These streams were naturally formed to carry lower flow rates for shorter durations; stream expansion via erosion is the result of higher runoff rates.

Urban areas produce a higher volume of runoff per volume of rainfall than nonurban areas. This higher runoff–rainfall factor in urban basins is a general mechanism for higher pollutant loads. An urban basin with a pollutant concentration similar to a rural basin will have a higher pollutant load because of the higher runoff volume. The rural basin has the capability to soak up much of the precipitation and prevent it from picking up pollutants and eroding soil as it flows through the basin.

Nonpoint pollutant loads are usually reported on a mass per area per time basis for various land uses (e.g., pounds per acre per year or kilograms per square meter per year). Table 7.18 summarizes reported export rates of nutrients, sediment, and heavy metals. These loads represent average annual loads and should be viewed as long-term estimates of pollutant export. The actual nonpoint source load during any year can vary greatly from the reported averages because of rainfall, soil cover, or numerous other factors.

Rural Sources. Rural sources are usually divided into two groups: agriculture and silviculture. Agriculture presents a larger, more complex problem than silviculture because of the variety of sources. Agriculture sources include erosion from pastures and cropland, runoff of agricultural chemicals (such as nutrients, pesticides, and herbicides), and runoff of animal wastes.

The principal pollutants are sediment, pesticides (68,000 metric tons of pesticide were used annually by U.S. agriculture in 1977), nutrients (44 million metric tons of fertilizers were used in 1976), organic material, and pathogens (49). Pollutant loading rates from various types of agricultural lands are reported in Table 7.19.

Several factors affect erosion from agricultural areas: soil, climate, topography, and the type of crop and land management practices. The Soil Conservation Service (SCS) actively promotes erosion control through the use of groundcover, progressive tillage techniques, and the prevention of gully and channel erosion. De-

TABLE 7.18 Pollutant Export Rates for Urban Land Uses* (55)

	Sediment	Total phosphorus	Total nitrogen	Lead
Residential	620–2300	0.4–1.3	5.0–7.3	0.06
Commercial	50–830	0.1–0.9	1.9–11.0	0.17–1.10
Industrial	400–1700	0.9–4.1	1.9–14.0	2.2–7.0

*In kilograms per hectare per year; 1 kg/ha/year = 0.89 lb/acre/year.

TABLE 7.21 Average Annual Dustfall Values for Various Water Resources Regions (52)

Area	Geometric mean		Range	
	ton/mi ² /mo	MT/km ² /mo	ton/mi ² /mo	MT/km ² /mo
New England	8.2	2.87	0.5–152	0.18–53.2
Mid-Atlantic	5.5	1.92	0.3–241	0.1–84.4
Upper Colorado	143.3	50.2	69–281	24–98.4
Pacific Northwest	7.2	2.52	0.3–317	0.1–111
Lower Mississippi	62.0	21.7	18–270	6.3–95
Missouri basin	34.7	12.2	6–103	2.1–36
Lower Colorado	33.9	12	16–69	5.6–24
South Atlantic	5.0	1.75	1.2–296	0.42–104
Tennessee	4.2	1.47	1.1–17.2	0.38–6.0
Ohio	2.8	.98	1.7–5.9	0.6–2.1
Upper Mississippi	12.5	4.37	0.3–315.3	0.1–110.4
Great Lakes	32.0	11.2	2–206	0.7–72
Souris-Red Rainy	23.4	8.2	3.0–73	1.1–26
Rio Grande region	29.5	10.33	12–269	4.2–94
Texas Gulf region	32.4	11.34	8–116	2.8–41
Arkansas-White-Red	—	—	—	—
California	16.9	5.9	1–38	0.35–13.3
Great Basin	14.7	5.15	5–56.5	1.8–20

spite this effort, the U.S. Department of Agriculture estimates that 57 to 76 million acres (23 to 31 Mha), around 15 to 20% of the nation's agricultural land, is in need of sediment control measures (50).

Animal wastes can be significant contributors to nutrient and pathogen contamination. Feedlots have high pollutant loads caused by the flow of runoff through the feedlot area.

Atmospheric Deposition. Atmospheric deposition occurs in the form of precipitation or as dustfall. Average precipitation concentrations for urban and rural areas are presented in Table 7.20.

Acid rain, a significant problem caused by atmospheric deposition, is attributed to the presence of sulfates, sulfites, and nitrates from the burning of organic fuels, especially coal. Acidification in lakes draining basins with low buffer capacity has been observed in Sweden, Canada, and the northeastern United States. Acid rain adversely affects the biota of streams and lakes. Severe decreases in fish populations have resulted from acid rain. Terrestrial ecosystems are also adversely affected by acid rain.

Dustfall rates vary significantly from region to region and are largest in the central United States. Mean dustfall values for different water resource regions are listed in Table 7.21. Pollutant concentrations in rural dustfall are related closely to soil conditions; urban dustfall is related more to local air pollution problems. Dustfall from urban areas in Germany consisted of 0.05 to 0.2% of lead as a percent of dry weight, 0.5 to 1% of ammonia, and 0.5 to 1% of nitrate. Urban dustfall is water soluble (59).

Other Sources. Other important nonpoint sources include streambank erosion, hydrological modifications (channelization, dredging, dam construction), irrigation return flows, solid waste dumps, septic tanks, and atmospheric deposition (acid rain). Streambank erosion and hydrologic modifications are usually small areas of intensely disturbed land. Small-scale erosion control techniques applied over relatively small areas can greatly reduce the impact of these nonpoint sources. Irrigation return flows usually present salinity problems. Solid waste dumps and septic tanks are often governed by groundwater considerations as well as runoff processes. This makes these problems significantly more complex.

Water Quality Impacts

Stormwater runoff from lands modified by human activities can harm surface water resources, and, in turn, violate water quality standards by changing natural hydrologic patterns and by elevating pollutant concentrations and loadings. Stormwater runoff may contain or mobilize high levels of contaminants, such as sediment, suspended solids, nutrients, heavy metals, pathogens, toxins, oxygen-demanding substances, and floatables. Such contaminants are carried to nearby streams, rivers, lakes, and estuaries. Individually and combined, these pollutants can reduce water quality and threaten one or more designated beneficial uses. Often, an increased volume of runoff or contaminants can lead to violations of water quality standards.

Receiving waters are classified as conservative and nonconservative systems. Conservative water systems hold most pollutants entering from nonpoint and point sources. Reservoirs or lakes with very long hydraulic residence times are very efficient conservative systems. Nonconservative systems include rivers, estuaries, or reservoirs with very short residence times. These systems have the ability to quickly transport pollutants downstream of source areas.

Conservative Systems. Most lakes can be considered conservative systems. They are affected by nonpoint sources because pollutants accumulate in the water column and the bottom sediments. Sediments, nutrients, toxic substances, and acidity are the most important water quality parameters for lakes. Sedimentation is particularly important in drinking water reservoirs and is almost entirely responsible for reducing safe yields and shortening the useful life of a reservoir. High sediment loads in lakes can lead to excessive aquatic weed growth, destruction of aquatic breeding areas, high turbidity and premature loss of capacity.

Sediment control in lakes often consists of dredging accumulated bottom material. Dredging is a treatment for a symptom of a watershed problem, rather than a cure for lake problems. Watershed management practices designed to permanently control sediment loads should be used in conjunction with short-term solutions such as dredging.

Nonpoint nutrient loads are important in the lake eutrophication process. Nonpoint sources usually contribute a large fraction of the annual nutrient load to a lake.

Toxic materials can accumulate in lake systems. This material can affect aquatic life and concentrate in the food chain. Pesticides and PCBs (polychlorinated biphenyls), for example, are exported into the water environment almost exclusively by nonpoint sources.

Nonconservative Systems. Nonconservative systems, such as rivers, are usually impacted to a smaller degree by nonpoint sources. The most serious impacts occur when nonpoint sources drain into small streams where dilution is low. Wet-weather stream water quality is degraded, and accumulation of benthic pollutants can affect the stream during dry periods. Sediments, nutrients, and toxic materials affect streams in a similar fashion as lakes, although usually not as severely. Dissolved oxygen depletion is often caused by organic sediments deposited by nonpoint sources.

Other nonpoint source problems in receiving waters include bacteriological, oil and grease, and radiological contamination. These problems are usually local in nature and result from special nonpoint source situations.

Design of Field Monitoring Programs

Traditionally water quality monitoring has been performed during dry weather periods to establish baseline conditions and to determine the impacts of point sources. Wet-weather monitoring is now being performed to evaluate the impacts of nonpoint sources. The monitoring can focus on either wet-weather concentration problems or on nonpoint source loadings. A concentration analysis is designed to monitor the pollutants throughout the storm event to determine the frequency and duration of poor water quality. A pollutant loading study requires both wet and dry weather monitoring and is usually performed to calculate annual pollu-

tant budgets. While concentration studies are important for determining water quality violations, loading studies are generally more relevant to the study of nonpoint source impacts.

The actual monitoring data are used in several ways. Concentration data are used to

- Compare against water quality standards or criteria.
- Compare against dry-weather pollutant concentrations.
- Determine the frequency and duration of degraded water quality during wet-weather periods.

Loading data are usually expressed initially as pounds (kilograms) per storm event and are transformed to an annual load. These data are used to

- Develop statistical information for decision makers.
- Develop simple models that are used to predict average annual pollutant export rates.
- Calibrate or verify complex nonpoint source simulation models.

Manual versus Automatic Sampling. Flow data and water quality samples can be collected by field crews or by a combination of field crews and automatic equipment. Field crews are easier to utilize and are usually more reliable. Automatic equipment has the advantage of being able to operate in an immediate, unattended fashion and is able to collect more data with less skilled workers.

The rapid response time of urban basins usually requires the installation of automatic equipment. Runoff from a small [125 to 450 acre (50 to 150 ha)], highly impervious basin can begin in minutes after the start of a storm. Even large urban basins can react quickly; flow rate changes of 2000 ft³/s (50 m³/s) in 15 min have been observed in 20,000-acre (80-km²) urban basins.

Rural basins are generally larger and have slower runoff response times. The primary monitoring difficulty in rural basins is the large distance between monitoring stations. Access problems also occur frequently since good sampling sites are often located off of the main roads, in areas where access is difficult.

Number and Location of Stations. A nonpoint source sampling network should be designed to monitor receiving water impacts, measure loads going into a receiving water, or both. Stations intended to determine loadings from a geographic area (such as a watershed) should be located to include as large an area and contain as many of the areas of the major nonpoint pollutant sources as possible. Because of the expense or difficulty in monitoring some areas or land uses, compromises are often made. If small tributaries exhibit similar land use distribution, one can be monitored and used to estimate the second tributary's pollutant export characteristics. Simple pollutant export models can be used to estimate the significance of various basins. Small basins with relatively small nonpoint pollutant sources are often regarded as statistically insignificant.

Stations built to monitor in-stream impacts should be located following more traditional sampling schemes. The area of interest should be bracketed by upstream and downstream stations. A control station on a hydrologically similar but "undisturbed" watershed can be employed to determine baseline conditions.

Two types of monitoring stations are employed in a nonpoint source survey. Small catchment stations ranging from 12 to 125 acres (5 to 50 ha) in size are used to gather data on specific land uses or special areas. They are usually found on storm sewers, drainage ditches, or small tributaries. Data from these stations can then be used to represent the land use type for load calculations on other nonmonitored areas. The second type of station is built to monitor larger basins and measure nonpoint source loads impacting a receiving body. These larger basins generally have an area of more than 125 acres (50 ha) and are usually monitored on larger stream channels or rivers.

Flow Measurement. Flow measurement is probably the most important element in collecting accurate monitoring data (61–63). The most common method is to use a level recorder to measure stream stage, which is then used with a stage discharge relationship to calculate flow. In large streams, unique stage-discharge relationships are developed from stream gauging activities using current meters (64). A Price current meter, which consists of six cups that revolve in response to current, is often used for this purpose. Several gaugings at different flow rates (stream levels) are required to develop an accurate curve. A minimum of six stream flow measurements per year over a variety of flow conditions should be taken.

Smaller basins are usually equipped with a primary device such as a flume or weir to provide a stage–discharge relation (62). Primary devices produce a hydraulic transition between subcritical and supercritical flow by restricting the channel and/or increasing the channel slope.

Sharp-crested, broad-crested, and V-notch weirs can be used to measure flow, but they tend to trap sediment and pollutants in the upstream pool. Flumes do not exhibit this sedimentation problem and are generally superior for nonpoint source monitoring. The most popular flumes are HS, H, HL, cutthroat, and Parshall flumes for open-channel applications, and Palmer–Bowlus flumes for in-pipe flow measurement.

Another popular method used to develop a stage-discharge relation is Manning’s equation for open-channel flow. This relation should only be used as a last resort unless the roughness and energy slope terms can be calibrated against actual field data. Calibration can be performed by gauging the stream with current meters or employing a continuous tracer system to perform mass balances on the flow stream. The Manning equation is also useful to extrapolate existing stage-discharge curves to increase the monitoring range.

Flow meter devices to measure stage include float-operated, acoustic, and bubbler devices. Float devices are relatively inexpensive and simple to operate but require an in-stream stilling well to protect the float and to dampen water level surges. The expense of constructing a stilling well often negates the cost savings of the float device. Acoustic devices determine water level by measuring the travel time of an ultrasonic signal from the acoustic sensor to the water level. Bubbler systems use either a manometric system or a pressure transducer to measure the static pressure required to bubble air or nitrogen gas out of a small tube fixed in the stream.

Water Sampling Methods. If manual sampling procedures are to be used, extensive field training is essential to ensure the collection of an adequate sample (60). Sampling locations, equipment use, and general procedures must be understood by all field personnel. Field quality control measures should be implemented.

An analysis of water quality constituents in the stream cross section should be performed to determine the distribution across the width and from top to bottom of the stream. Samples should be tested for a suspended parameter (such as total suspended solids) and a soluble parameter (such as orthophosphate). The testing should be performed at both a small runoff event and a moderate to high flow event if possible. Vertical sampling should be performed using depth samplers or closeable bottles if the stream is more than 4 to 5 ft (1.2 to 1.5 m) deep. Figure 7.31 shows an experimental result of sediment distribution in a stream cross section flowing at 5 ft/s (1.5 m/s).

Manual and automatic water sampling procedures should be designed according to the results of the cross section analysis. Manual sampling should be performed using depth samplers (such as Kemmerer bottles) if nonsurface samples are required. If samples are being extracted from a large collection bottle or bucket, care must be exercised to avoid settling in the collection vessel. Frequent shaking or the use of a sample splitter is usually necessary due to the high concentrations of settleable solids in runoff samples.

Automatic equipment consists of a pump (usually peristaltic, positive displacement, or vacuum type) controlled by timers or flow meters (66). Important specifications for an automatic sampler are

- Sample velocity in the sampler line of greater than 3 ft/s (1 m/s) to prevent sedimentation
- 12-V dc power supply option
- Constant sample size over different sampling heads for rising and falling streams

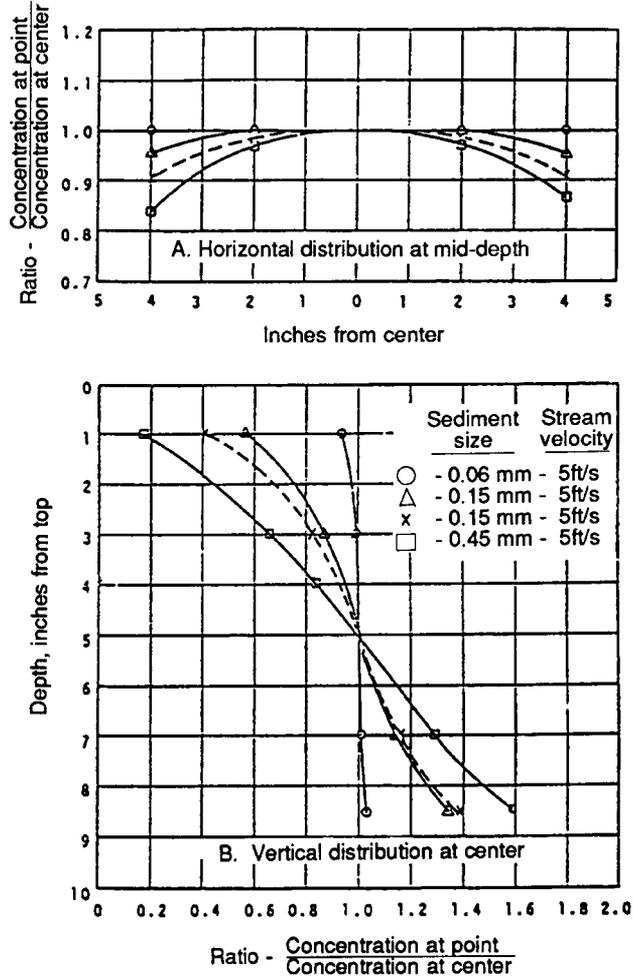


FIGURE 7.31 Sediment distribution at sampling station (65).

- Air purges of the sample line before and after sample collection
- Minimum 3/8-in (1-cm) sample line
- Ability to composite samples in one container or to collect at least 24 discrete samples

Two basic approaches can be applied to intensive monitoring of storm loads. Several discrete samples can be collected during the storm event and analyzed separately, or discrete samples can be composited into a single event sample for analysis (60).

The discrete sample approach allows examination of pollutant concentration variations during a storm event but requires a larger commitment of laboratory resources since more samples must be analyzed. Total storm loads can be determined by constructing pollutographs from the sample series and integrating the curves, either graphically or numerically. An event mean concentration (EMC) can be used to compare different storms or different stations on an average concentration basis.

Composite samples can be obtained by manually compositing grab samples or by automatically compositing with flow-proportional samples. Composite samples represent EMCs in a single sample. While this greatly reduces laboratory costs, it restricts the information gained of pollutant processes occurring during the storm.

The interval between sample collection (either discrete samples or portions of a composite) depends on the response time and duration of the storm. Generally, at least four samples on both the rising limb and six on the recession limb should be collected for proper resolution of nonpoint source loads in developed areas. Table 7.22 presents additional guidelines for measurement intervals for catchments of different sizes and perviousness.

Other Monitoring. Precipitation intensity, volume, and duration data are important for a monitoring program. Most studies use a combination of recording rain gauges (either tipping bucket or weighting gauges) and collecting gauges that record only total volume. The design of a location rain gauge network and the actual installation of gauges require consideration of meteorological and topographical factors to avoid monitoring errors. Wind errors can be significant; for example, a rain gauge undermeasures rain by about 20% at wind speeds of 20 mi/h (9 km/s) (2).

Monitoring of atmospheric deposition is performed using samplers that expose a wetfall bucket during precipitation events and expose a dryfall bucket during dry periods. These samplers are usually designed with a rain sensor that controls a movable lid that covers one bucket while the other bucket is exposed. Wet-fall samples are usually analyzed after every storm, and dryfall buckets are analyzed on a monthly or bi-monthly basis.

Analysis of Monitoring Data

Nonpoint source problems can be examined by several basic approaches. The simplest is to estimate watershed loadings by using national or regional loading data from basins similar to local conditions. This method can produce relatively large errors as local factors can greatly influence pollutant export.

The next level of complexity is the application of a desktop model to predict basin loads. The Universal Soil Loss Equation (USLE) has been used for many years to estimate loadings from agricultural fields. Modifications have been made to extend the loading prediction to entire watersheds and to determine the loadings of other pollutants carried on the soil. Urban area models are available to assess the loadings from accumulation–washoff of pollutants on city streets. These models are largely based on the type of urban land use or on the street density in the basin.

Desktop models are more sophisticated than simple loading factors and have the ability to assess different options. They can be used to predict the success of best management practices, such as street sweeping in urban areas or different tillage methods in rural areas. The desktop model is still a relatively rough predic-

TABLE 7.22 Maximum Water Quality Measurement Intervals (66)

Catchment size, acres (ha)	Desirable maximum measurement interval, min	
	Highly impervious catchment	High pervious catchment
50 (20)	3	4
100 (40)	4	7
600 (240)	7	20
3000 (1200)	15	30

tive tool and has the safest application in comparing nonpoint sources in basins or in planning management practices.

To assure accurate information, watershed monitoring is required. Runoff flow is measured and water samples are collected and analyzed. The data are then used with simple statistics to analyze concentration trends or to calculate pollutant loads from a watershed. The monitoring data can be used to calibrate and verify the results of the desktop models and to design stormwater management practices.

The final level of complexity (and expense) is the application of monitoring data to a sophisticated nonpoint source computer model. This allows the most accurate interpretation of the mechanisms involved in the generation of nonpoint sources. The evaluation of alternatives can be performed using computer-simulated processes. Many models exist for different types of basins, levels of complexity, and nonpoint source problems; the main requirement is the use of monitoring data to calibrate the model. Without calibration data, the most sophisticated model will usually yield results no more accurate or reliable than a simple desktop model.

Event Mean Concentration. EMCs are useful for comparing average storm concentrations and for calculating annual loadings. They are calculated by integrating the pollutograph (instantaneous load versus time) and the hydrograph (67).

Statistical Analysis. Simple statistical analyses of concentration data are usually applied to nonpoint source concentration data because most databases are small and have low confidence limits. Many of the statistical concepts from hydrology are used since hydrological events are the driving force behind nonpoint sources.

Nonpoint source data are usually log-normal distributed because the distribution of hydrologic events is usually log-normal. The skewness and kurtosis of nonpoint source data can be calculated to determine the degree of normality or the normality of the log transformed data. Geometric means of log-normal data are probably more representative than arithmetic means. Statistical tests that depend on normal distributions (such as the *T*-test or analysis of variance) should use log transformations of the data.

Concentration–duration plots (concentration versus percentage time value equalled or exceeded) can be used to determine the frequency of certain concentrations occurring during wet weather. EMC can be used to represent average storm concentrations, or peak concentration values can be used if discrete sampling was performed. These plots can also be constructed for individual storms with discrete sampling data to determine the frequency of a particular pollutant level.

The frequency data for an EMC, a peak concentration, or a particular concentration level of interest can be used with hydrologic data describing the frequency of wet weather to calculate pollutant recurrence intervals. Return periods for various pollutant concentrations can then be estimated, similar to return periods for rainfalls and stream flows. This information has been developed from the EPA's Nationwide Urban Runoff Program (1982) and is particularly valuable for nontechnical decision makers.

Other statistical methods which have been employed with nonpoint source data include the *t*-test, analysis of variance (ANOVA), and Duncan's test. The *t*-test can be used to indicate the significance of a difference between two sample means (such as data from two different land uses). The ANOVA procedure determines if significant differences occur in a sample population. Duncan's test examines the differences between sample subsets.

Storm Loads. Data from nonpoint source monitoring studies are usually reduced to a pollutant load per storm basis. An EMC is multiplied by the volume of runoff. After several storms are sampled, these load per storm data are then used to estimate an annual load from the basin. This is done by assuming the monitored storms are a representative sample of the total number of storms that usually occur during a year (53).

Annual Loads. Simple models can also be employed to estimate annual loads from monitoring data. Regressions of total load versus total runoff from a family of storms provide a slope in concentration units

(mass per volume) and can be used to predict pollutant load for a specific quantity of runoff. Some modifications of this technique involve abstracting out base flows from the storm runoff volumes or low-flow loads from storm loads to better represent only the runoff process. Many researchers have found good relationships using log-load versus log-runoff volume regressions reflecting the log-normal distribution of the data (68, 69).

Annual loads are often normalized with respect to the basin area to present loading data with units of mass per area per time (kilograms per hectare per year or pounds per acre per year being the most common). This technique allows a direct comparison of pollutant export from basins with different land uses. This calculation can be applied to small single-land-use basins as well as large mixed-land-use watersheds. Other common methods used to normalize data are to express loads as a function of curb length, population density, drainage density, or as a function of particular land uses (53).

Nonpoint Source Models

A variety of nonpoint source predictive models are available. They range in complexity from simple loading function and desktop procedures to sophisticated hydrological–water quality models run on large mainframe computers. The selection and use of a model depends on the resources and the nature of the nonpoint source problem.

Models are usually used to estimate loadings from different nonpoint sources and to evaluate different alternatives. The distribution of loads from various sources in a watershed can be evaluated to identify critical areas that are responsible for water quality impacts. Models can also be used to predict the impacts of land use changes or the implementation of management practices.

Loading Functions. The simplest model is to apply loading factors to specific land uses in the watershed being studied. These loading factors are usually expressed as mass of pollutants per unit area per time (e.g., grams per square meter per year or pounds per acre per year). The annual nonpoint source loading for a particular pollutant for a specific land use is calculated by selecting a representative pollutant loading from Tables 7.18 and 7.19 or from other scientific literature. The selected loading is then multiplied by the number of square meters (or acres) of that specific land use in the watershed. This general procedure is illustrated below:

$$L_A = (LU_A)(LF_A) \quad (7.48)$$

where L_A = annual nonpoint source loading for land use A (mass/time)

LU_A = area of land use A in watershed being studied (area)

LF_A = loading factor for land use A (mass/area·time)

The annual pollutant loading for the total watershed, expressed as mass per year (e.g., kilograms per year or pounds per year) is calculated by summing all the annual pollutant loadings for each land use in the watershed. This general procedure is illustrated below:

$$L_w = \sum_{i=1}^n (LU_i)(LF_i) \quad (7.49)$$

where L_w = total annual pollutant loading for entire watershed (mass/time)

I = various land uses in watershed

The main problem with this approach is that it does not properly consider variations in topography, soil conditions, meteorological conditions, climate, and land use activities. Each watershed has its own charac-

teristics and cannot be directly compared to other watersheds even if they are similar. Obviously, the more similar two watersheds are, the more accurate this approach will be if data from one watershed are transferred to the other. Significant differences can occur in the amount and quality of stormwater runoff based on localized conditions, such as slope, geology, soil type, specific watershed activities, and the size of sub-basins within the watershed.

Loading factors should only be used to provide a very rough ballpark estimate of the annual pollutant loadings from a watershed. Whenever possible, more sophisticated methodologies, such as desktop and computer modeling, should be used to assess the magnitude of nonpoint sources.

Desktop Models. More sophisticated desktop models can be applied for more accurate results. These models usually have larger input data requirements and require more time and effort to use. These models are often developed from regression equations; nonpoint source loads are calculated as a function of one or more parameters. Some of the more common independent variables include land use distribution, land-use-related factors, rainfall, runoff volume, or a combination of factors.

Other desktop models employ simple equations developed from empirical data to describe actual processes such as rainfall energy, soil erodability, and management practices. The careful selection of coefficients is responsible for much of the sensitivity of these types of models.

Universal Soil Loss Equation. Erosion modeling has traditionally been performed using the USLE (71). The basic equation predicts upland soil erosion (erosion from a field-sized area) for various time periods. This equation is frequently modified to include a sediment delivery ratio to predict loads at the watershed outlet:

$$A = RK(LS)CPS_d \quad (7.50)$$

where A = calculated soil loss, mass/area/time period

R = rainfall erosion factor/time period

K = soil erodability factor, mass/rainfall erosion factor

LS = topographic (slope-length) factor

C = cropping management factor

P = erosion control practice factor

S_d = sediment delivery ratio

Average annual rainfall erosion factors R representing the erosion energy during an entire year are shown in Figure 7.32. The annual distribution of erosive rains during a year varies greatly for different regions as shown in Figure 7.33. For many areas of the United States much of the annual erosional energy can occur in two or three months. Erosion control practices are particularly important during this period.

The soil erodability factor K is an experimentally determined factor representing erodability per rainfall erosion unit. The general magnitude of K for different soils is presented in Table 7.23. Values of K can be found in most soil surveys.

The topographic slope-length (LS) factor represents the combined effects of slope and overland flow. The topographic factor can be determined from Figure 7.34.

The cropping management factor C reflects the groundcover conditions of the soil. Values for general land use categories are presented in Tables 7.24 and 7.25. Selected values for construction sites range from 1.0 for no ground cover, 0.05 for temporary seedling (90% after 60 days), to 0.13 for 1 ton/acre (0.29 for 1 mg/ha) of hay mulch.

The erosion control practice factor P represents land treatment factors, such as contouring, terracing, sedimentation basins, and other measures. The P factors for general land use categories are presented in Table 7.26.

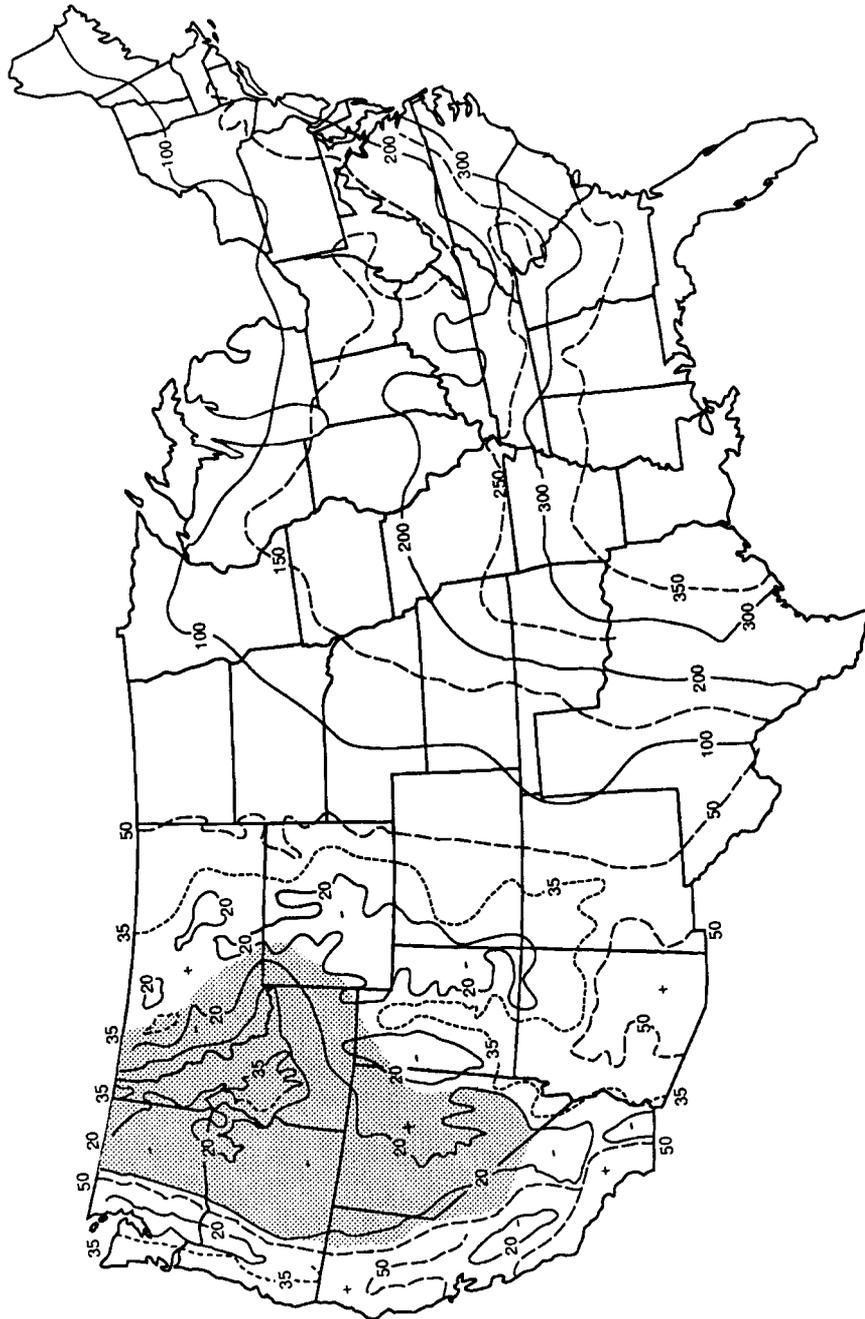


FIGURE 7.32 Values of Rainfall Factor R (in tons/acre; multiply by 2.24 for tons/ha) (72).

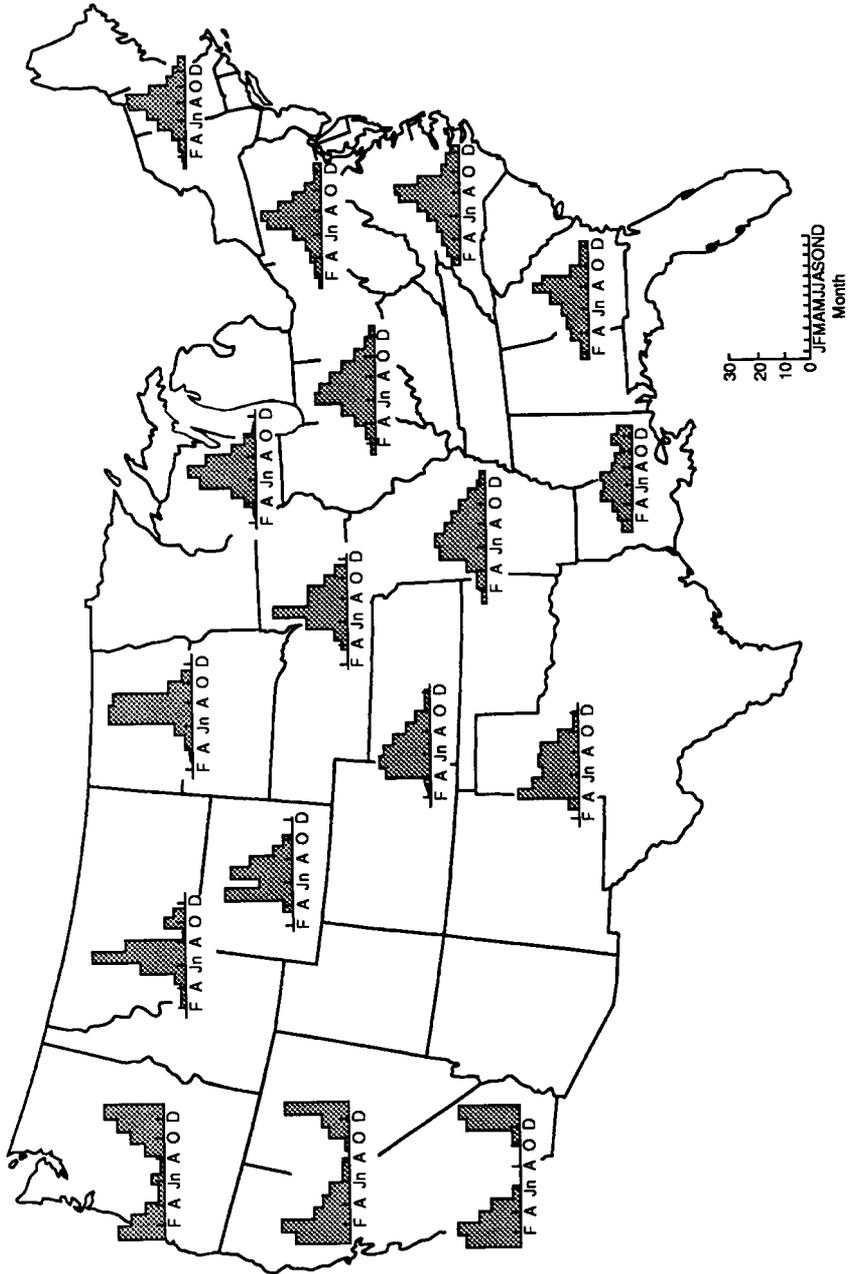


FIGURE 7.33 Monthly distribution of erosive rainfall as percentage of annual (72).

TABLE 7.23 Indications of the General Magnitude of the Soil-Erodability Factor, K (72)

Texture class	Soil Erodability Factor, K^*		
	Organic matter content, %		
	0.05	2	4
Sand	0.05	0.03	0.02
Fine sand	0.16	0.14	0.10
Very fine sand	0.42	0.36	0.28
Loamy sand	0.12	0.10	0.08
Loamy fine sand	0.24	0.20	0.16
Loamy very fine sand	0.44	0.38	0.30
Sandy loam	0.27	0.24	0.19
Fine sandy loam	0.35	0.30	0.24
Very fine sandy loam	0.47	0.41	0.33
Loam	0.38	0.34	0.29
Silt loam	0.48	0.42	0.33
Silt	0.60	0.52	0.42
Sandy clay loam	0.27	0.25	0.21
Clay loam	0.28	0.25	0.21
Silty clay loam	0.37	0.32	0.26
Sandy clay	0.14	0.13	0.12
Silty clay	0.25	0.23	0.19
Clay		0.13–0.29	

*The values shown are estimated averages of broad ranges of specific soil values. When a texture is near the borderline of two texture classes, use the average of the two K values. For specific soils, use of SCS K -value tables will provide much greater accuracy.

The sediment delivery ratio S_d is the ratio of sediment delivered at a watershed outlet to the gross upland erosion. A comprehensive model is not available to accurately predict delivery ratios for different watersheds. Various models relating stream order, drainage density, soil type, and watershed size have been developed using data from different watersheds or areas. Figure 7.35 relates sediment delivery ratio to drainage density determined from topographic maps (74).

A modification of the USLE for small (construction) sites is the Revised Universal Soil Loss Equation (RUSLE):

$$A = RK(LS)CP$$

The U.S. Environmental Protection Agency has proposed that RUSLE may be used to predict stormwater discharges associated with small construction sites. These sites would have soil disturbance from clearing, grading, and excavating activities but would not be expected to adversely affect water quality. In these cases, the annual soil loss rate for the period of construction for a site would be expected to be less than 2 tons/acre/year (4.5 Mg/ha/yr). The annual soil loss rate is calculated assuming the constants of no ground cover and no runoff controls in place. R values published by the U.S. Department of Agriculture (USDA) are applied (116).

Nonpoint Calculator Model. The Midwest Research Institute (MRI) developed a “nonpoint calculator” which uses regression equations between in situ soil and pollutant loadings for oxygen-demanding substances, nutrients, and some heavy metals (74). The generalized function is listed below.

$$Y(P)_E = aY(S)_E C_s(P) r_p \quad (7.51)$$

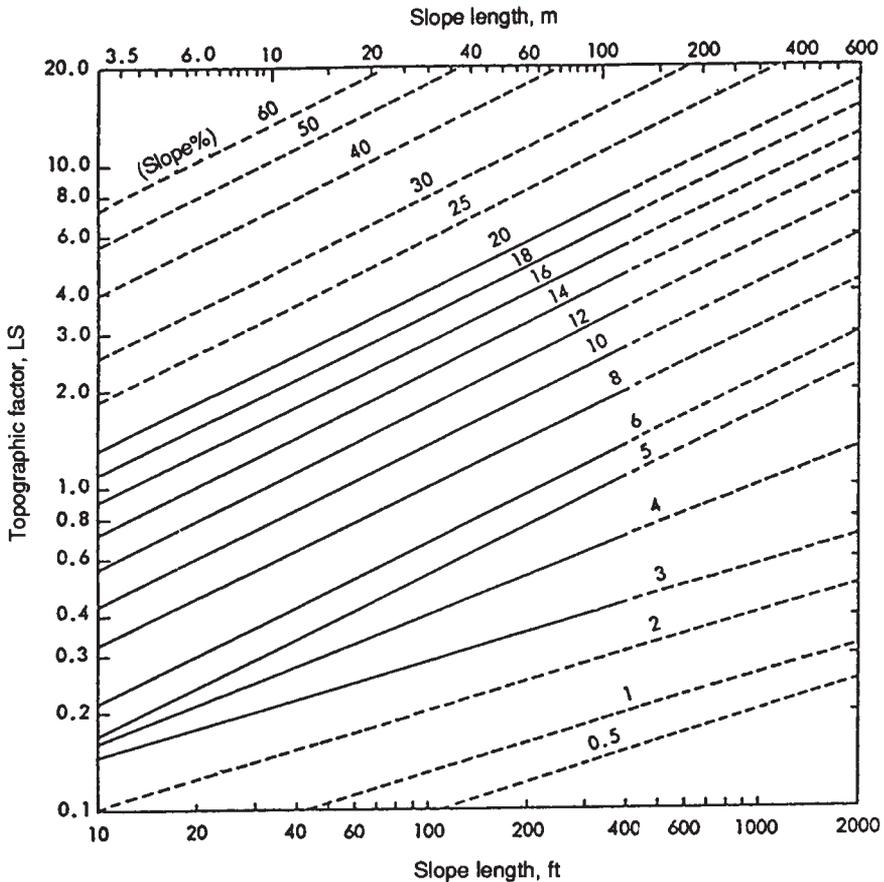


FIGURE 7.34 Sample plot of topographic factor (LS) versus slope and slope length. (The dashed lines represent estimates for slope dimensions beyond the range of lengths and steepnesses for which data are available.) (73)

where $Y(P)_E$ = pollutant P load to streams

a = dimensional constant

$Y(S)_E$ = sediment loading to stream (from USLE)

$C_s(P)$ = concentration of pollutant in soil profile

r_p = enrichment ratio for pollutant P

More detailed information on these functions is presented in Tables 7.27 and 7.28.

The best results are obtained when long-term annual loading estimates are used. However, water quality impacts during certain times of the year are more important, thus the technique can be used for shorter time periods by varying R (rainfall) and C (soil cover) values when calculating $Y(S)_E$. Agricultural land uses have varying R and C values throughout the year. Generally, these values are maximal from mid-June to mid-July.

$C_s(P)$ estimates are most reliable when local soil sampling data are available. In the absence of such data, studies have found that percent phosphorus and nitrogen found in surface soil ranges from 0.0 to 0.3. Percent organic matter may be estimated by multiplying nitrogen values by 20. Enrichment ratios r_p should be calcu-

TABLE 7.24 C Factors for Pasture, Rangeland, and Idle Land (73)*

Type and height of raised vegetative canopy [†]	Canopy cover [‡] , %	Type [§]	Ground cover that contacts the surface, %					
			0	20	40	60	80	95–100
No appreciable canopy		G	0.45	0.20	0.10	0.042	0.013	0.003
		W	0.45	0.24	0.15	0.090	0.043	0.011
Canopy of tall weeds or short brush(0.5m fall height)	25	G	0.36	0.17	0.09	0.038	0.012	0.003
		W	0.36	0.20	0.13	0.082	0.041	0.011
	50	G	0.26	0.13	0.07	0.035	0.012	0.003
		W	0.26	0.16	0.11	0.075	0.039	0.011
	75	G	0.17	0.10	0.06	0.031	0.011	0.003
		W	0.17	0.12	0.09	0.067	0.038	0.011
Appreciable brush or bushes (2 m fall height)	25	C	0.40	0.18	0.09	0.040	0.013	0.003
		W	0.40	0.22	0.14	0.085	0.042	0.011
	50	G	0.34	0.16	0.085	0.038	0.012	0.003
		W	0.34	0.19	0.13	0.081	0.041	0.011
	75	C	0.28	0.14	0.08	0.036	0.012	0.003
		W	0.28	0.17	0.12	0.077	0.040	0.011
Trees but no appreciable low brush (4 m fall height)	25	G	0.42	0.19	0.10	0.041	0.013	0.003
		W	0.42	0.23	0.14	0.087	0.042	0.011
	50	U	0.39	0.18	0.09	0.040	0.013	0.003
		W	0.39	0.21	0.14	0.085	0.042	0.011
	75	G	0.36	0.17	0.09	0.039	0.012	0.003
		W	0.36	0.20	0.13	0.083	0.041	0.011

*All values shown assume: (1) random distribution of mulch or vegetation, and (2) mulch of appreciable depth where it exists.

[†]Average fall height of water drops from canopy to soil surface, in meters (1 m = 3 ft).

[‡]Portion of total area surface that would be hidden from view by canopy in a vertical projection (a bird's-eye view).

[§]G = Cover at surface is grass, grasslike plants, decaying compacted duff, or litter at least 5 cm (2 in) deep. W = Cover at surface is mostly broadleaf herbaceous plants (as weeds) with little lateral root network near the surface and/or undecayed residue.

TABLE 7.25 C Factors for Woodland (73)

Stand condition	Tree canopy, % of area	Forest litter, [†] % of area	Undergrowth [‡]	C factor
Well stocked	100–75	100–90	Managed [§]	0.001
			Unmanaged [§]	0.003–0.011
Medium stocked	70–40	85–75	Managed [§]	0.002–0.004
			Unmanaged [§]	0.01–0.04
Poorly stocked	35–20	70–40	Managed [§]	0.003–0.009
			Unmanaged [§]	0.02–0.09

*When tree canopy is less than 20%, the area will be considered as grassland or cropland for estimating soil loss.

[†]Forest litter is assumed to be at least 2 in deep over the percent ground surface area covered.

[‡]Undergrowth is defined as shrubs, weeds, grasses, vines, etc., on the surface area not protected by forest litter. Usually found under canopy openings.

[§]Managed = grazing and fires are controlled, unmanaged = stands that are overgrazed or subjected to repeated burning.

TABLE 7.26 *P* Values for Erosion Control Practices on Croplands (73)

Range of slope	Erosion control practice				
	Up- and downhill	Cross-slope farming without strips	Contour farming	Cross-slope farming with strips	Contour strip-cropping
2.0–7	1.0	0.75	0.50	0.37	0.25
7.1–12	1.0	0.80	0.60	0.45	0.30
12.1–18	1.0	0.90	0.80	0.60	0.40
18.1–24	1.0	0.95	0.90	0.67	0.45

lated from local data as well; however, MRI suggests using a range of 1.0 to 5.0, with nitrogen enrichment ratios generally two to three times greater than the phosphorus ratio. Availability factors for nitrogen and phosphorus are generally less than 15%. Values used for nitrogen and phosphorus that have been used in the past are 6 and 10%, respectively.

SCS Curve Number Method. The SCS curve number method has been modified to calculate pollutant loads using the following equation:

$$L = CQ \quad (7.52)$$

where L = pollutant loading (mass – time – 1)

C = pollutant concentration (mass – volume – 1)

Q = stormwater flow calculated using the SCS curve number method (mass – time – 1)

This method can be used to calculate both particulate and soluble pollutant loadings.

Computer Models. Many computer models are available to simulate nonpoint source processes. Several researchers have summarized main features of existing computer models (52, 53, 58, 75).

These references illustrate the different applications for models and the different hydrologic/water quality processes which are simulated. A model can be classified using several criteria:

1. Type of modeling application
 - a. Land use type
 - b. Basin size
 - c. Basin complexity (single catchment versus multiple catchments)
 - d. Temporal resolution (annual or single event, average versus continuous record)
2. Type of hydrologic–hydraulic–water quality process
 - a. Ability to generate hydrographs
 - b. Ability to simulate other hydrologic processes (snowmelting, groundwater, etc.)
 - c. Runoff routing method (ability to simulate backwater, storage, etc.)
 - d. Erosion generation method
 - e. Sediment transport method
 - f. Chemical pollutant and sediment relationships
 - g. Treatment processes (storage, best management practices, etc.)

Huber and Heaney catalogued several models for the EPA and commented on the selection process (Figure 7.36) (75).

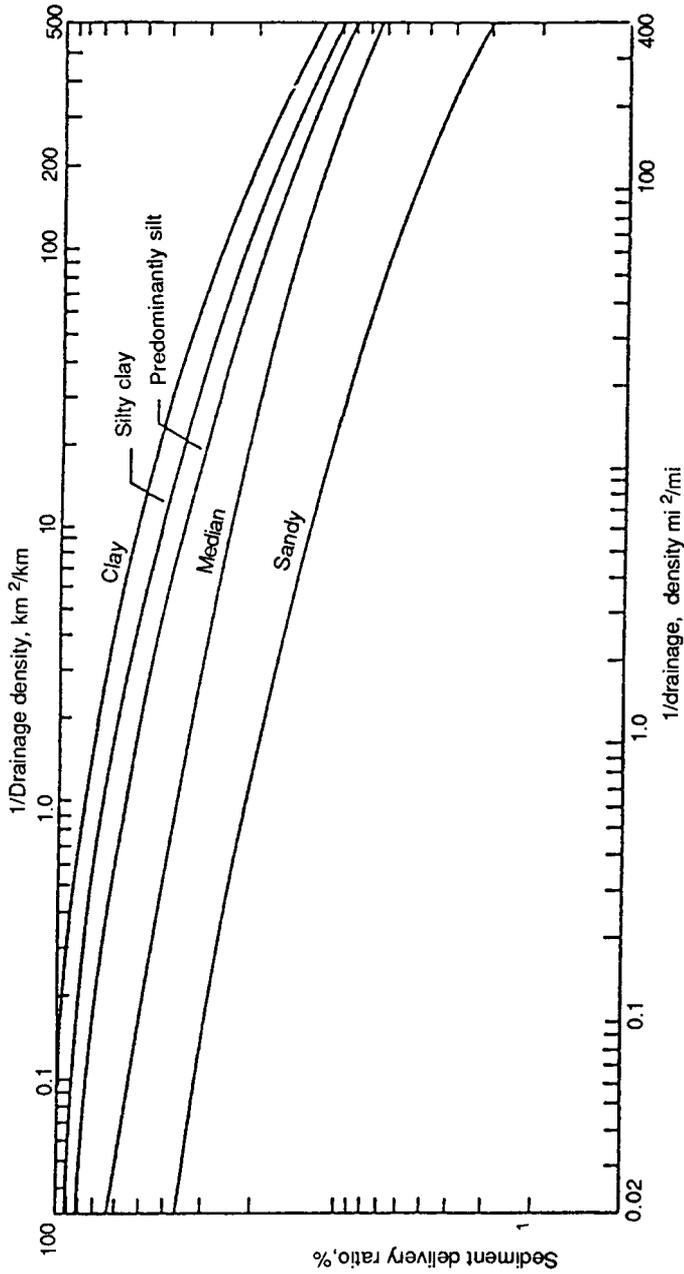


FIGURE 7.35 Sediment delivery ratio for relatively homogenous basins (74).

TABLE 7.27 Loading Functions Summary (74)

Loading type or source	Loading function	Description
Sediment (agriculture)	$Y(S)_E = \sum_{i=1}^{n_i} (ARKLLSCP S_i)_i$	Applicable to agricultural land; may be modified with multipliers to represent silviculture, construction, surface mining
Nitrogen	$Y(NT)_E = aY(S)_E C_s(NT) r_{NT}$ $Y(NA)_E = Y(NT)_E f_N + Y(M)_{Pr}$ $Y(N)_{Pr} = A Q(QR)Q(Pr) N_{Pr} b$ $Y(NA) = Y(NT)_E f_N + Y(N)_{Pr}$	The first loading function represents total nitrogen yield from sediment. The second loading function represents total available nitrogen in sediment. The third function represents the precipitation-borne nitrogen loading that is transported via overland flow. The last function is total available nitrogen.
Phosphorus	$Y(PT) = aY(S)_E C_s(PT) r_{PT}$ $Y(PA) = Y(PT) f_p$	The first loading function represents total phosphorus yield. The second loading function represents total available phosphorus. There is no phosphorus in precipitation.
Organic matter	$Y(OM)_E = aY(S)_E C_s(OM) r_{OM}$	Can be used to estimate BOD loadings.
Pesticides (Herbicides, Insecticides, Fungicides)	$Y(HIF) = a_n C_s(HIF) - Y(S)_E r_{HIF}$	Based on average pesticide residual (nonpeak loads). Used for insoluble pesticide, if average soil concentration known.
Salinity	$Y(HIF) = \sum_{i=1}^n Q_i C_i a$	Used for water-soluble and insoluble pesticide loadings.
Sediment	$Y(TDS)_{IRF} = aAC(TDS)_{GW}(IRR + Pr - CU)$ $Y(TDS)_{IRF} = a[Q(str)]_B C(TDS)_B - Q(str)_A$ $C(TDS)_A - Y(TDS)_{BG} - Y(TDS)_{Pr}$ $Y(TDS)_{IRF} = (TDS)_{IRF} A$	Source-to-stream methodology. Stream-to-source methodology. Requires arcal salt-loading rates based upon accumulated results of stream monitoring program.
	$Y(S)_E = \sum_{i=1}^{n_i} (ARKLLSCPMS_i)$	M is a multiplier indicating the effect of different types of disturbances. Nutrient loads are assumed to be only inorganic.
	$Y(AMD) = M[Ka(I_{AU} + I_{IU} + I_{AS} + I_S) - Kb Q(R)C(AUK)_{BG}]$ $Y(AMD) = aAQ(R) [C(SO_4) - C(SO_4)_{BG} - C(SO_4)_{Pr}]$ $Y(AMD) = aQ(str) [C(SO_4) - C(SO_4)_{BG} - C(SO_4)_{Pr}]$	The appropriate equation to be used for calculating acid mine drainage depends upon the region of the country. The first two loading functions reflect a source-to-stream approach; the next two reflect a stream-to-source estimating procedure.
	$Y(HM)_s = a_n C_s(HM) Y(S)_E$	Estimating procedure for heavy metals and radioactivity loads are identical except that heavy metals go in as ppb and radioactivity as picocuries/liter.
Heavy metals/radioactivity	$Y(HM) = a \sum_{i=1}^n Q_i C(HM)_i$	
Feedlots	$Y(HM) = aAQ(R) [C(HM) - C(HM)_{BG}]$	
	$Y(HM) = aQ(str) [C(HM) - C(HM)_{BG}]$	
Terrestrial disposal	$Y(i)_{FL} = aQ(FL)C(i)_{FL} FL_d A$ $Y(i)_{LF} = aC(i)_{LF} Q(LF) LF_d A$	Overland runoff approach; not related to sediment. Subsurface/groundwater return flow, in the form of leachate.

TABLE 7.28 List of Variables and Abbreviations Used in Table 7.27

Variable	Definition	Variable	Definition
<i>A</i>	Source area, ha	\bar{Q} (OR)	Overland runoff, cm/year
AMD	Acid mine drainage	\bar{Q} (P)	Total precipitation flow rate, cm/year
AS; AU	Active surface or underground mine	\bar{Q} (Perc)	Percolation flow rate, cm/year
BG	Background source	\bar{Q} (R)	Direct runoff, cm/year
<i>C</i>	Cover management factor	\bar{Q} (Str)	Stream flow rate, L/s
$C(i)$	Concentration of pollutant <i>i</i> in sediment	$\bar{Q}(t)$	Runnoff over a period of time <i>t</i>
$C(i)_{source}$	Concentration <i>C</i> of pollutant <i>i</i> in source <i>L</i>	<i>R</i>	Rainfall erosivity factor
$C(Alk)_{BG}$	Concentration of background alkalinity, mg/L	R_p	Rainfall erosivity factor due to rainfall
<i>D</i>	Overland distance between erosion site and receptor water, ft	R_s	Rainfall erosivity factor due to snowmelt
DD	Drainage density, km ⁻¹	r_{NT}	Enrichment ratio for nitrogen (ratio of concentration of nitrogen in sediment to that in soil)
<i>E</i>	Annual average erosion rate, MT/ha/year	r_{OM}	Enrichment ratio for organic matter (ratio of concentration of organic matter in sediment to that in soil)
f_N	Ratio of NA:NT in eroded sediment	r_{PT}	Enrichment ratio for phosphorus (ratio of concentration of phosphorus in sediment to that in soil)
f_P	Ratio of PA:PT in eroded sediment	<i>S</i>	Slope gradient factor; also sediment
FL	Small feedlot source	S_d	Sediment delivery ratio (ratio of amount of sediment delivered to a stream to the amount of on-site erosion)
FL_d	Feedlot delivery ratio	TDS	Total dissolved solids
HIF	Herbicide, Insecticide, Fungicide; and pesticide	<i>U</i>	Composite topographic factor for irregular slopes
HM	Heavy metals or radioactivity	$Y(AMD)$	Acid mine drainage loading, kg/year
<i>I</i>	Load index for acid mine drainage	$Y(HIF)$	Total pesticide loading, kg/year
IRF	Irrigation return flow	$Y(HM)$	Heavy metal loading, kg/year
IRR	Irrigated water added annually to crop root zone, cm/year	$Y(i)_{FL}$	Loading of pollutant <i>i</i> from small feedlots, kg/year
IS; IU	Inactive surface or underground mine	$Y(i)_{LF}$	Loading of pollutant <i>i</i> from landfills, kg/year
<i>K</i>	Soil erodibility factor	$Y(i)_U$	Loading of pollutant <i>i</i> from urban areas, kg/year
<i>L</i>	Slope length factor	$Y(N)_{Pr}$	Nitrogen loading from precipitation runoff, kg/year
LF; LF _{<i>d</i>}	Landfill, landfill delivery ratio	$Y(NA)$	Available nitrogen loading, kg/year
LS	Topographic factor	$Y(NT)_E$	Total nitrogen loading from erosion, kg/year
NA	Available (or mineralized) nitrogen	$Y(OM)_E$	Organic matter loading, kg/year
N_{Pr}	Nitrogen yield rate per unit area from precipitation, kg/ha/year	$Y(PA)$	Available phosphorus loading, kg/year
NT	Sum of nitrogen of all chemical forms	$Y(PT)$	Total phosphorus loading, kg/year
OM	Organic matter	$Y(RAD)$	Loading of radioactive substances, microcuries/year
OR	Overland runoff	$Y(S)_E$	Sediment loading from surface erosion, MT/year
<i>P</i>	Percolation rate, cm/year	$Y(S)_U$	Loading of street solids from urban areas, kg/year
<i>P</i>	Conservation practice factor	$Y(TDS)_{BG}$	Salinity (TDS) load from background, kg/year
Pr	Annual average precipitation, cm/year, storm precipitation, cm	$Y(TDS)_{IRS}$	Salinity (TDS) load in irrigation return flow, kg/year
PA	Available phosphorus	$Y(TDS)_{Pr}$	Salinity (TDS) load from point sources, kg/year
PD	Population density, number/ha	$Y(i)_{BG}$	Yield of pollutant <i>i</i> from background, kg/year
PT	Total phosphorus; also point source		
$\bar{Q}; \bar{Q}$	Runoff due to a storm event, cm		
$\bar{Q}(FL)$	Feedlot runoff, cm/year		

The most commonly used models are the EPA's SWMM (76) and SWMM Level 1, the Corps of Engineers STORM (77), and the Hydrologic Simulation Program—Fortran (HSPF) (78). Summaries of SWMM Level 1, STORM, and HSPF prepared by Heaney and Huber (75) are presented in Tables 7.29 to 7.31, respectively.

The modeling process begins by collecting and reducing the required input data. Data needs include land use and management practice data, meteorological data, soils data, and topographic data. A long-term planning study will usually require average precipitation data for the model. Other applications may require the use of a design storm using statistically based input data. A model can also be used to examine a particular storm event or series of events. Actual precipitation data can be used to calibrate and verify the model.

More detailed modeling projects attempt to calibrate and verify the modeling effort. The calibration process involves adjusting the discretionary model parameters to produce output data corresponding to actual hydrologic or water quality data from a field monitoring program. The verification procedure compares output data from a calibrated model against a second independent set of field data. The data used for calibration and verification should be distinct and separate elements to determine the accuracy and robustness of the model.

One final step often performed during the modeling effort is a sensitivity analysis of all of the input parameters. A series of computer runs are made, each with only one parameter's value adjusted, first up and then down, usually by 10 to 20%. Computer runs are generated to determine how sensitive the model is to each parameter. This information is then used with confidence limit estimates for the various input parameters to estimate the accuracy of the output data.

COMBINED SEWER OVERFLOWS

Historically, sewer systems were constructed for the purpose of conveying stormwater runoff away from urban areas. Disposal of domestic wastes into early sewer systems was prohibited. In the early nineteenth century, the association between unsanitary living conditions and epidemics of various diseases was recognized, and ordinances were changed to allow the introduction of domestic wastes into the sewer system. This created the first combined sewers and led to the development of the water carriage waste disposal system.

A combined sewer system is characterized by a single-pipe network that collects and transports domestic wastewater, stormwater runoff, and in some cases industrial wastewater. Combined sewers are typically placed at a greater depth than storm sewers in order to service basements. New construction of combined sewer systems has been almost completely eliminated, except in areas where location of storm sewer outfalls is severely limited. Many large cities in the United States are still partially served by combined sewers (Figure 7.37), typically in the older areas. In 1974, it was estimated that 29% of the total sewered population was served by combined sewers. The use of combined sewers is more common in the northeast and Great Lakes areas.

Many separate sanitary sewers essentially behave as combined sewers during wet periods due to excess infiltration which results in greatly increased flows many times higher than occur during dry weather. Old sanitary sewers were often constructed of stone or brick and were not watertight even when new. Holes and cracks develop with time in most sanitary sewer lines due to factors such as differential settling and damage by tree roots. Defective sewer joints are probably a more significant source of infiltration than cracks or holes. Sewer joints constructed prior to the 1950s were rigid, and effective gasket materials were not used. New materials and technology have reduced the problem of infiltration in more recent systems; however, overloading of sanitary sewer lines due to excess infiltration remains common. Considerations which apply toward combined sewage also relate to heavily infiltrated municipal sewage.

During periods of dry weather, flow in combined sewers is composed of domestic and perhaps industrial wastes, and street drainage from watering of lawns and washing operations. Flow rates during dry periods

TABLE 7.29 Summary of EPA Stormwater Management Model: Level I (SWMM-Level 1) (75)

Characteristic	Summary data
Applicable land drainage area	Urban
Time properties	Averages seasonal or annual inputs and outputs
Space properties	Small to large single catchment
Physical processes	(1) Runoff estimated as a function of population density and precipitation; (2) dry-weather flow quantity estimated by coefficient method; (3) wet-weather quality estimated as a function of land use, precipitation, population density, type of sewerage system, and street sweeping frequency; (4) effectiveness of storage or treatment evaluated by deriving production function, regionalized parameters are used; (5) street-sweeping production function is included; (6) sewer-flushing production function is included
Chemical processes	(1) Quality constituents are represented as conservative substances; (2) BOD ₅ , SS, VS, PO ₄ , and N are represented
Ecological processes	None
Economic analysis	Provides the least costly combination of storage and treatment and other management practices
Mathematical properties	Deterministic model with graphical or analytical solution of algebraic equations
Input data requirements	Precipitation, land use, population density, type of sewerage system, street-sweeping frequency, treatment efficiencies, control costs
Ease of application	(1) Nonproprietary model; (2) two documentation reports available through EPA or University of Florida; (3) documentation includes example problems; (4) user support available through University of Florida and EPA workshops; (5) used extensively in nationwide assessment, needs surveys, and EPA 208 studies throughout the United States
Output and output format	No specified output format
Linkages to other models	Part of overall EPA SWMM package
Labor needs	Environmental engineer with some background in economics
Costs	(1) Nominal setup costs; (2) no computer-related expenses involved
Model accuracy and sensitivity	(1) Relatively crude model; (2) provides a rough check on the reasonableness of simulated results for a given area; (3) neglecting treatment in storage may mislead users into specifying two separate control units when a single storage and treatment unit could be used
Other comments	(1) Popular model that has been used extensively; (2) user support available; (3) model developed by the University of Florida, Department of Environmental Engineering, under EPA sponsorship

TABLE 7.30 Summary of STORM Model (75)

Characteristic	Summary data
Applicable land drainage area	(1) Urban; (2) general nonurban
Time properties	(1) Continuous simulation with hourly time step; (2) input hourly precipitation and daily air temperature; (3) output hourly, event, and/or annual summaries; (4) run for extended periods, usually a year or more
Space properties	Small to large single catchment
Physical properties	(1) Rainfall/snowmelt runoff estimated by runoff coefficient method or SCS method; (2) dry-weather flow quantity estimated by coefficient method, daily and hourly variation factors included; (3) wet-weather quality estimated using linear pollutant buildup with washoff rate proportional to runoff; (4) erosion estimated by USLE to interaction of "erosion" with other pollutants; (5) dry-weather quality estimated by coefficient method; (6) no treatment is assumed to occur in storage; (7) treatment is simply a hydraulic accounting of flow through capability, no explicit pollutant removal
Chemical processes	(1) Quality constituents represented as conservative substances; (2) suspended and settleable solids, BOD, total nitrogen, and orthophosphate are represented
Ecological processes	Coliforms are represented
Economic analysis	None
Mathematical properties	Deterministic model with iterative solution of algebraic equations
Input data requirements	(1) Hourly precipitation and daily air temperature ; (2) land use including runoff parameters; (3) pollutant accumulation and washoff rates; (4) land surface erosion parameters; (5) treatment rate and storage volume
Ease of application	(1) Nonproprietary model; (2) updated user's manual available from Hydrologic Engineering Center (HEC), U.S. Army Corps of Engineers, Davis, California; (3) documentation includes hardware and software requirements, brief subroutine description, data input specifications, default data, and test problems; (4) user support is available through HEC; (5) used extensively in EPA 208 and other studies throughout the United States.
Output and output format	(1) Four types of output: quantity analysis, quality analysis, pollutograph analysis, land surface erosion analysis; (2) quantity and quality reports include annual statistics of rainfall or snowmelt runoff, pollutant washoff, and the quantity, quality, and frequency of overflows; (3) land surface erosion report shows average annual values for sediment production and delivery
Linkages to other models	(1) Linked to SWMM—RECEIV by Mata Systems; (2) linked to simplified SWMM RECEIVE by Medina
Labor needs	(1) Environmental engineer; (2) systems programmer
Costs	(1) Model available at nominal cost (about \$50) from HEC; (2) precipitation and air temperature data tapes available from U.S. Weather Service for about \$100; (3) land use data usually available at little cost; (4) one-year simulation costs about \$1.00
Model accuracy and sensitivity	(1) STORM is used as a preliminary model to provide rough estimates of runoff quantity and quality, and required sizes of storage and treatment; (2) neglecting the treatment in storage may mislead user into specifying two separate control units when a single storage and treatment; unit could be used
Other comments	(1) A very popular model that has undergone extensive testing and refinement; (2) excellent user support available

TABLE 7.31 Summary of Hydrocomp Simulation Program (HSP) (75)

Characteristic	Summary Data
Applicable land drainage area	(1) Urban; (2) agriculture; (3) forest
Time properties	(1) Continuous simulation with hourly time step, run for extended periods, usually a year or more; (2) time step of input data: (a) precipitation (hourly), (b) air temperature, dew point, solar radiation, and wind velocity (daily); (3) time units on output: hourly event, and/or annual summaries
Space properties	(1) Multiple catchments, overland flow, open channel, and sewer routing with modified kinematic wave formulation using Manning's equation; (2) does not explicitly handle surcharging, does flag if surcharging occurs; (3) reservoir routing can be included
Physical processes	(1) Rainfall/snowmelt runoff estimated by detailed accounting of water balance in upper and lower soil storage zones and deep groundwater zone, incorporates interception, evaporation, and evapotranspiration; (2) dry-weather flow quantity estimated by coefficient method; (3) wet-weather quality estimated using linear pollutant buildup with washoff rate directly proportional to runoff, separate washoff functions are used for each residuals discharge; (4) dry-weather quality estimated by coefficient method; (5) storage routing, no treatment simulation or treatment in storage
Chemical processes	Quality constituents represented as conservative substances; suspended and settleable solids, BOD, total nitrogen, and orthophosphate are represented
Ecological processes	Coliforms are represented
Economic analysis	None
Mathematical properties	(1) Deterministic model with iterative solution of algebraic equations; (2) mathematical representation based on physical concepts but formulations are based largely on empirical relationships
Input data requirements	Job specification; hourly precipitation; max-min daily air temperature; daily radiation, cloud cover, dew point, and wind; daily or semimonthly evaporation; HSP calibration parameters, e.g., infiltration, depression storage, pollutant decay rates
Ease of application	(1) Proprietary model of Hydrocomp, Inc., Palo Alto, Calif.; (2) current user's manual available for quantity (Jan. 1976) and quality (April 1977); (3) documentation includes detailed description of the model components and underlying equations and assumptions; (4) extensive user support available through Hydrocomp which also publishes periodic newsletters and conducts short courses; (5) HSP has evolved from the Stanford Watershed Model, which was originally developed for hydrologic simulation of nonurban catchments; it is probably the most detailed and complete rainfall and runoff model and the most widely used quantity simulator model; quality modeling is similar to other models and has been used in recent EPA 208 studies, e.g., Chicago; (6) model is especially well suited for continuous simulation where detailed hydrologic analysis is needed
Output and output format	(1) Flexible output format from quantity and quality analysis including tabular and graphical arrays, routine statistical analysis and frequency analysis; (2) monthly, annual, and period of record summaries available

TABLE 7.31 Summary of Hydrocomp Simulation Program (HSP) (75) (continued)

Characteristic	Summary Data
Linkages to other models	(1) Linked to Hydrocomp's receiving water model; (2) related models have been developed for agricultural areas, e.g., the Agricultural Runoff Model (ARM)
Labor needs	Environmental engineer familiar with hydrology
Costs	(1) Proprietary model of Hydrocomp, Inc.; (2) initial cost to become familiar with program is several hundred dollars to participate in short course, user costs depend on requested level of professional services
Model accuracy and sensitivity	(1) HSP is very well suited to problems where continuous simulation is needed with emphasis on quantity; (2) its quality version has the same residuals discharge buildup and washoff relation as SWMM and STORM
Other comments	(1) The quantity model has been used extensively; (2) HSP has a very flexible data handling capability; (3) excellent user support is available

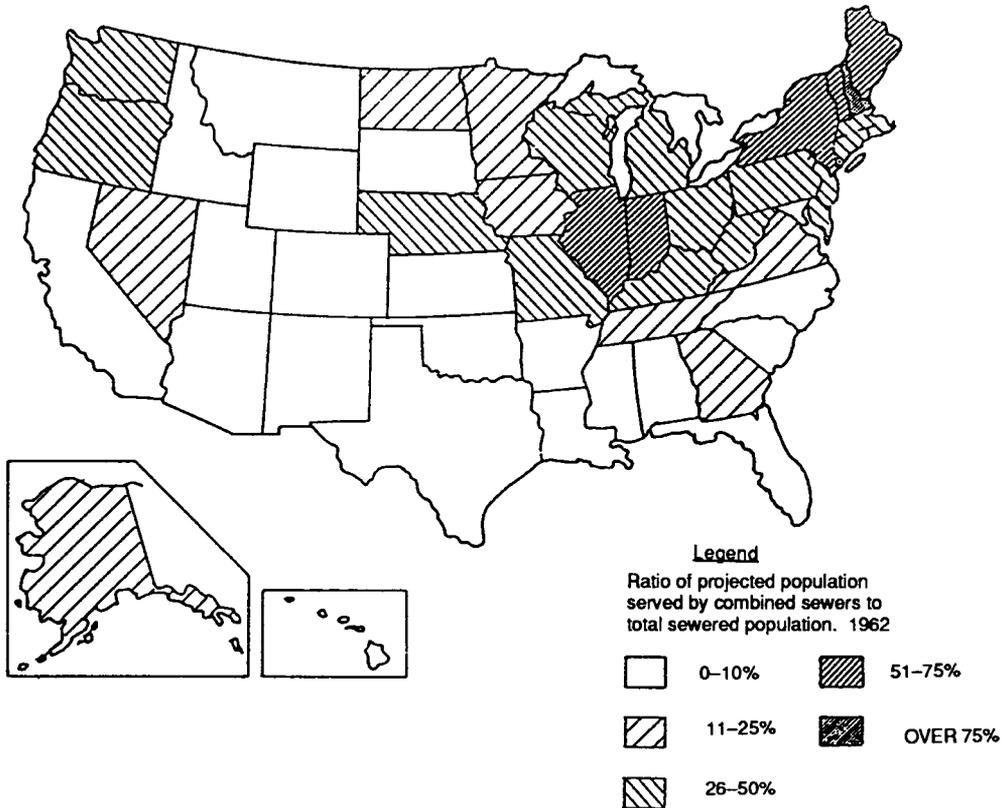


FIGURE 7.37 Relative use of combined sewers by states (76).

are generally less than 5% of the hydraulic capacity of a combined sewer. Combined sewers flow into regulators which divert dry-weather flows to treatment facilities through interceptor lines. Since combined sewers carry both sewage and stormwater in the same conduit, flow rates are greatly increased during periods of stormwater runoff. Regulators are designed to divert a portion of the combined sewage and stormwater flow into interceptors for conveyance to treatment facilities. The flow in excess of the interceptor capacity (or that permitted by the regulator) is either stored or discharged directly into receiving waters as combined sewer overflow. Combined sewer overflow occurs when the ratio of stormwater flow to dry-weather flow exceeds an allowable dilution ratio, typically between 4:1 and 8:1. If interceptors deliver combined sewage to the treatment facility at a rate exceeding the treatment rate, a portion of the flow is bypassed either to storage or directly to receiving waters representing additional combined sewer overflow.

Water Quality Characteristics

Combined sewer overflow contains untreated sewage and can exert a severe impact on receiving water bodies. Water quality characteristics of combined sewer overflows are highly variable due to the method used for diversion and volume of stormwater runoff. Due to the variability in combined sewer overflow, “average” water quality characteristics cannot be described. Various studies have attempted to compare typical combined sewage concentration to typical municipal sewage concentrations. Table 7.32 summarizes several of these comparisons. Table 7.33 compares these typical combined sewage concentration levels to other forms of sewage and nonpoint source characteristics.

Since combined sewer overflow is composed of domestic sewage and stormwater runoff, the resulting water quality would be expected to lie somewhere between these two components. This is often not the case, especially during the initial period of stormwater flow or “first flush.” During the first flush, typical concentrations are many times higher than those in the extended overflow and are often substantially higher than average typical concentrations of untreated dry weather sewage (Table 7.34). During dry periods, deposits of silt and sludge accumulate in sewer joints and improperly sloped sewer lines. These deposits are high in suspended solids, BOD₅, grease, and organic matter, and undergo decomposition often under anaerobic conditions. The initial surge of stormwater runoff scours these deposits from the depressions and conveys them through the system as part of the first flush. The duration of the first flush can last from a few minutes to hours depending upon the size of the system and path tracked by the storm.

Catch basins are also a significant source of pollutants during the first flush. These devices are installed at the point when stormwater enters the sewer system for the purpose of removing grit and coarse debris during storms and/or providing water seals to impound sewage odors. Catch basins form standing pools where collected material, leaves, and other organic matter can undergo decomposition if not regularly cleaned following storms. Water quality in catch basins can approach that of untreated sewage without proper maintenance, resulting in increased pollutant loadings during the first flush.

Water Quality Control

The construction of new separate sanitary sewers to replace existing combined sewers is often impractical because of the enormous cost, limited effectiveness, inconvenience to the public, and extended time required for implementation. Efforts at controlling combined sewer overflows are frequently directed at improved separation devices and containment for subsequent treatment.

Improved regulators can significantly improve the water quality characteristics of combined sewer overflows. Early regulators were typically weirs installed in combined sewers downstream from a vertical or horizontal orifice that was connected to the interceptor. Relatively new devices, such as the swirl regulator–concentrator, achieve both quality and quantity control by inducing hydraulic flow patterns that separate and concentrate solids from the main flow. Many of these devices can be retrofitted to existing systems.

TABLE 7.32 Comparison of Quality of Combined Sewage for Various Cities^a (79)

Type of wastewater, location, year, ref. no.	BOD, mg/L		COD, mg/L		DO, mg/L Avg	SS, mg/L		Total coliforms, MPN/100 ml		Avg total nitrogen, mg/L as N	Avg total phosphorus, mg/L as P
	Avg	Range	Avg	Range		Avg	Range	Avg	Range		
Typical untreated municipal	200	10–300	500	250–750	—	200	100–350	5×10^7	$1 \times 10^7 - 1 \times 10^9$	4.0	10
Typical treated municipal											
Primary effluent	135	70–200	330	165–500		80	40–120	2×10^7	$5 \times 10^6 - 5 \times 10^8$	35	7.5
Secondary effluent	25	15–45	55	25–50		15	10–30	1×10^3	$1 \times 10^2 - 1 \times 10^4$	30	5.0
Selected combined											
Atlanta, Ga., 1969	100	48–540	—	—	8.5	—	—	1×10^7	—	—	—
Berkeley, Calif., 1968–69 ^c	60	18–300	200	20–600	—	100	40–150	—	—	—	—
Brooklyn, N.Y., 1972	ISO	86–428	—	—	—	1051	132–8,759	—	—	—	1.2 ^b
Bucyrus, Ohio, 1968–69	120	11–560	400	13–920	—	470	20–2,440	1×10^7	$2 \times 10^5 - 5 \times 10^7$	13	3.5
Cincinnati, Ohio, 1970	200	80–380	250	190–410	—	1,100	500–1,800	—	—	—	—
Des Moines, Iowa, 1968–69	115	29–158	—	—	—	295	155–1,166	—	—	12.7	11.6
Detroit, Mich., 1965	153	74–685	115	—	—	274	120–804	—	—	163 ^d	4–9
Kenosha, Wis., 1970	129	—	464	—	—	458	—	2×10^6	—	104 ^d	5.9
Milwaukee, Wis., 1969	55	26–182	177	118–765	—	244	113–848	—	$2 \times 10^5 - 3 \times 10^7$	3–24	0.8 ^b
Northampton, U.K., 1960–62	150	80–350	—	—	—	400	200–800	—	—	10 ^e	—
Racine, Wis., 1971	119					439	—	—			
Roanoke, Va., 1969	115	—	—	—	—	78	—	7×10^7	—	—	—
Sacramento, Calif., 1968–69	165	70–328	238	59–513	—	125	56–502	5×10^6	$7 \times 10^6 - 9 \times 10^8$	—	—
San Francisco, Calif., 1969–70	49	1.5–202	155	17–626	—	68	4–426	3×10^6	$2 \times 10^6 - 2 \times 10^6$	—	—
Washington, D.C., 1969	71	10–470	382	80–1760	—	622	35–2,000	3×10^6	$4 \times 10^6 - 6 \times 10^6$	3.5	1.0

^aData presented here are for general comparisons only. Since different sampling methods, number of samples, and other procedures were used, the reader should consult the references before using the data for specific planning purposes.

^bOnly orthophosphate.

^cInfiltrated sanitary sewer overflow.

^dOnly ammonia plus organic nitrogen (total Kjeldahl).

^eOnly ammonia.

TABLE 7.33 Comparison of Typical Magnitudes of Concentrations from Nonpoint Sources and Sewage (58)

Source	Suspended solids, mg/L	BOD ₅ , mg/L	COD, mg/L	Total N, mg/L	Total P, mg/L	Lead, mg/L	Total coliform, no./100 ml
Precipitation	11-13	12-13	9-16	1.2-1.3	0.02-0.04		
Background levels	5-1000	0.5-3.		0.05-0.5	0.01-0.2	0.1	
Agricultural cropland		7	80	9	0.02-1.7		
Animal feedlots	30	1000-11,000	31,000-41,000	920-2100	290-380		
Urban stormwater	100-10,000(630)	10-250 (30)	20-600	3-10	0.6	0.35	10 ³ -10 ⁸
Combined sewers	100-2000(410)	20-600(115)	20-1000	9-10	1.9	0.37	10 ⁵ -10 ⁸
Municipal sewage, untreated	100-330 (200)	100-300 (200)	250-750	40	10		10 ⁷ -10 ⁹
Municipal sewage, treated	10-30	15-30	25-80	30	5		10 ² -10 ⁴

Note: Numbers in parentheses are typical values.

TABLE 7.34 Comparison of Quality Characteristics from First Flushes and Extended Overflows of a Combined Sewer (79)

Characteristic	First flushes, mg/L*	Extended overflows, mg/L*
COD	500–765	113–166
BOD ₅	170–182	26–53
SS	330–848	113–174
VSS	221–495	58–87
Total N	17–24	3–6
Coliforms [†]	—	—

*At 95% confidence level.

[†]Coliform 1.5×10^5 to $310 \times 10^7/100$ mL.

Combined sewer overflows can be controlled by providing storage facilities to collect excess overflow, particularly the first flush, and release the stored volume gradually after some treatment to receiving waters or to a centralized treatment facility. Storage provides flow equalization, attenuation, and in some cases, transmission. Storage facilities respond to intermittent and random storm behavior and are relatively unaffected by changes in flow and water quality. Some degree of treatment is provided by storage facilities where sedimentation occurs, and the removal of deposited materials must be considered in the design. An economy of scale exists for construction and operation of storage facilities. Computer models such as STORM and SWMM are capable of computing required or existing storage capacities. Generally, however, storage facilities are large, expensive, and dependent on other treatment facilities for processing retained water.

Storage facilities can be both in-line and off-line, and include excess sewer capacity storage, surface basins, underground vaults and tunnels, and underwater collapsible tanks. In-line sewer storage is feasible where conduits are large, deep, and flat, and interceptor capacity is high. System controls such as in-line sewer storage are directed toward maximum utilization of existing facilities and should be among the first alternatives considered. Controls of this type can often be attained at a fraction of the cost of constructing new facilities.

Other methods used to improve water quality of combined sewer overflow focus on controlling deposition in combined sewers during dry weather, thereby reducing the impact of the first flush. This can be done by periodic cleaning of the sewer system and maintaining flow velocities above the critical range at which organics settle out. Collected material should be removed from catch basins after storms. Existing sewer lines can be cleaned by increasing water volume temporarily to scour accumulated deposits. The ratio of flush flow to dry-weather (sewage) flow should not exceed the overflow ratio. New sewers should be installed at sufficient slope to assure flow velocities which prevent solids from settling out. Proper selection of sewer pipe will decrease the hydraulic roughness and increase flow velocities. In existing sewer systems, hydraulic roughness can be controlled by frequent cleaning, installation of sewer linings, or injection of a polymer gelled slurry.

Treatment

Many treatment alternatives are available for processing of combined sewage. Due to the irregular occurrence of storms, some form of storage is generally utilized to reduce the cost of a treatment facility. There is a trade-off between storage capacity and treatment rate. A storage facility can be designed to intercept the

first flush only, or the entire stormwater discharge, with a certain probability that combined sewer overflow will occur at a specified recurrence interval. Storage capacity and treatment rate are determined from the general storage equation and from available historic hydrological data.

Combined sewage can be processed using physical, chemical, or biological treatment. Many systems use a combination of these types for effective treatment. Table 7.35 summarizes removal efficiencies for several processes.

Biological treatment processes are not well suited for treating combined sewage due to problems in maintaining a biomass. Wet-weather combined sewage can have low organics concentrations which reduce the effectiveness of biological processes, and the biomass is sensitive to toxic substances often contained in urban stormwater runoff. Biological treatment processes are upset by the irregular occurrence of storm events and varying water quality. It is difficult and expensive to keep the biomass alive between events, therefore a combined sewer overflow treatment unit is ideally located adjacent to a sewage treatment facility which provides biomass during storm events. Fewer problems with keeping the biomass alive are encountered with lagoons.

Disinfection of combined sewer overflows can be achieved by the addition of chlorine, which first must oxidize organics and reduce components such as iron, manganese, and ammonia. Up to 40 mg of Cl_2/L may be required for effective bacteria kill. Even higher dosages are necessary to control viruses. Disinfection is almost always performed in combination with other treatment processes.

TABLE 7.35 Efficiencies of Various Stormwater Treatment Processes (58)

Process	Efficiency, %				
	Suspended solids	BOD ₅	COD	Total P	Total Kjeldahl N
Physical-chemical					
Sedimentation					
without chemicals	20–60	30	34	20	38
with chemicals	68	68	45		
Swirl	40–60	25–60	50–90		
Screening					
microstrainers	50–95	10–50	35	20	30
drum screens	30–55	10–40	25	10	17
rotary screens	20–35	1–30	IS	12	10
static screens	5–25	0–20	13	10	8
Dissolved air flotation	45–85	30–89	55	55	35
High-rate filtration*	50–80	20–55	40	50	21
Magnetic separation†	92–98	90–98	75		
Biological‡					
Contact stabilization	75–95	70–90		50	50
Biodisks	40–80	40–80	33		
Trickling filters	65–85	65–85			
Oxidation ponds	20–57	10–17		22–40	57
Aerated lagoons	92	91			
Facultative lagoons	50	50–90			

*With chemical addition.

†Only bench scale.

‡Applicable only to combined sewer overflows.

STORMWATER MANAGEMENT SYSTEMS

The effects of urbanization on the hydrologic cycle have been outlined in previous sections. As land is developed and paved, the quantity of surface runoff increases as well as the severity of downstream flooding and channel erosion. Development also increases nonpoint sources of pollution and degrades water quality.

Various alternatives for managing the impacts of stormwater are described in this section. These alternatives include both conveyance systems and structural management practices.

Philosophy and Goals of Stormwater Management

The optimum stormwater management system is based on the presumption that the natural drainage system is a resource that can enhance the environment when used wisely. Past practice has been to enclose natural channels in underground conduits whenever possible and move stormwater off-site as quickly as possible. Although this method can be effective in preventing flooding, it is very costly and destroys the natural environment, leaving a sterile, often unaesthetic system in its place. Modern systems of stormwater management, instead, try to use the natural system to the greatest extent while allowing the construction of facilities necessary for modern-day life. The goals of an optimum stormwater management system might be the following:

1. To minimize the danger and inconvenience of stormwater to individuals and society as a whole
2. To maintain and enhance the aquatic and terrestrial environment
3. To accomplish the previous two goals at the minimum cost to individuals and society as a whole

These goals can be accomplished by encouraging the following:

1. Minimize increases in surface runoff by preferentially constructing impervious surfaces where the soils have a naturally low infiltration rate and by constructing special controls to infiltrate excess runoff in more permeable areas
2. Maximize the amount of storage within the system by using structural detention measures, such as detention basins and parking lot storage, and by using natural materials in preference to structural controls
3. Minimize encroachment upon natural channels and floodplains through greenbelts and buffer strips
4. Combine stormwater management facilities with other facilities, such as parks, to minimize costs

The philosophy and goals of stormwater management are evident in the following case study. Land development provisions prepared for water quality protection in Georgia are presented in Table 7.36. Intended for new developments, the provisions provide guidance for modifying and developing local development regulations and are grouped into four categories: (1) overall measures of development, (2) streets and pavements, (3) drainage, and (4) construction process.

Stormwater Management Planning

Stormwater management planning includes structural and/or nonstructural best management practices (BMPs). The use of preventive measures, including nonstructural BMPs, is generally a low-cost means to abate water quality impacts. For industrial sites, stormwater pollution prevention planning formally began with the Phase I NPDES permitting program. For Phase II NPDES permitting, minimum criteria were established for all local governments with separate storm sewer systems. Watershed planning by local, regional, or state authorities brings a more comprehensive basis for assessing and protecting water quality (117).

TABLE 7.36 Land Development Provisions to Protect Georgia Water Quality (*121*)

Overall Measures of Development

Density zoning is regulation of development intensity by the quantity of development on a site as a whole, not by minimum lot size. It gives flexibility to adapt to site-specific topography and drainage, locating streets, homes and lots in ways that are at once economical, environmentally protective, and appropriate to local markets.

Stream buffers are reservations of undeveloped land adjoining stream channels. Undisturbed buffer vegetation filters inflowing runoff, prevents channel erosion, and creates habitats for functioning ecosystems. Siting construction away from drainage courses avoids the costs associated with flood damage and poor drainage.

Limited impervious cover controls the proportion of a site that can be covered in impervious roofs and pavements without treating the runoff. Limiting unmitigated impervious cover controls the generation of runoff and pollution at the source, while allowing development of any type and intensity.

Land use combination blends different, but mutually supportive, land use types in the same zoning districts. Certain types of commercial and office uses can be combined with residential uses, reducing dependence on automobiles and the pavements they require, and the consequent auto emissions and runoff

Paths for biking and walking allow individuals the choice of nonautomotive transportation. Biking and walking reduce automobile use, the quantity of pavement, and the quantity of pollutants generated from paved surfaces. In most of Georgia, the moderate terrain and mild climate favor biking and walking.

Infill zoning allows relatively high-density, mixed-used development or redevelopment where it would be compatible with an existing neighborhood. The local concentrations of runoff and pollutants are, of course, high. But infill development limits impervious cover and auto usage in the region as a whole. It accommodates some growth without destroying pristine areas, and without requiring large quantities of pavement to support routine automobile use.

Streets and Pavements

Limited street width and curbing limits street development to that needed for each street's specific function. This limits both runoff and construction costs. Narrow pavements encourage cautious driving, and eliminate the "speedway" feel of wide streets. They do not hinder emergency access when they are correctly applied only to streets with little traffic and little on-street parking.

Limited pavement in turn-arounds eliminates unnecessary pavement areas at the ends of cul de-sacs. In the centers of turn-arounds, pavement is unusable for vehicles. Replacing it with vegetated soil reduces runoff and provides infiltration and treatment. It reduces construction costs, but requires provision for maintenance.

Limited amount of parking eliminates unused portions of parking areas. In commercial and office areas, parking areas have been oversupplied. Limiting parking limits paved areas and runoff. It reduces construction costs and land consumption.

Porous pavement materials replace impervious pavements so the underlying soil can absorb rainfall and treat pollutants. Porous pavement materials can economically provide safer driving surfaces than the impermeable materials they replace. However they should be avoided on steep slopes.

Drainage

Drainage in vegetated swales carries, stores, treats, and infiltrates runoff in contact with permeable soil. Where vegetated swales replace curbs or drainage pipes, they reduce construction costs.

Swale "biofiltration" velocity control assures effective runoff treatment and infiltration by prolonging contact with soil and vegetation. Although the quantity of treatment is small, in a few large storms, the cumulative long-term effect of many small storms is vital.

TABLE 7.36 Land Development Provisions to Protect Georgia Water Quality (121) (continued)**Drainage** (continued)

Treatment of “hot spots” assures runoff treatment specifically at a few small, highly concentrated runoff and pollutant sources, such as dumpster pads and gasoline stations. This secures “point” treatment, even where treatment of runoff from other impervious surfaces cannot receive the same degree of careful attention.

Inlet labeling identifies swales and drainage inlets to the public, and indicates their purpose. This inhibits dumping of pollutants and educates the public about the environmental systems around them.

Construction Process

Limited clearing, grading and disturbance confines construction work to those areas that construction actually requires. This preserves existing trees and pervious soils that attenuate, treat, and infiltrate rainfall and runoff.

Nonstructural BMPs are preventive actions that involve management and source controls. For example, nonstructural BMPs include policies and ordinances that result in protection of natural resources and prevention of runoff. These include requirements to limit growth to identified areas, protect sensitive areas such as wetlands and riparian areas, reduce imperviousness, maintain open space, and minimize disturbance of soils and vegetation.

Structural BMPs include storage practices (wet ponds and extended-detention outlet structures), filtration practices (grassed swales, sand filters, and filter strips), and infiltration practices (infiltration basins, infiltration trenches, and porous pavement). These systems are developed as appropriate to address local concerns and to apply new control technologies as they become available.

For water quality planning and impact analysis, pollutant load reductions obtained through the implementation of BMPs for stormwater management should be reflected in total maximum daily load (TMDL) analyses. A TMDL analysis is required by the Clean Water Act for waters that will not achieve water quality standards after implementation of technology-based point source controls. A TMDL analysis includes the determination of the relative contributions of pollutants from point, nonpoint, and natural background sources. More specifically, an allowable TMDL is the sum of the individual waste-load allocations for existing and future point sources (including stormwater), waste-load allocations for existing and future nonpoint sources (including diffuse runoff and agricultural stormwater), and natural background materials, with a margin of safety incorporated.

Maximum extent practicable (MEP) is defined as the technology-based control standard currently used in the existing municipal stormwater program against which permit writers and permittees assess whether or not an adequate level of control has been proposed in the stormwater management program.

Stormwater Pollution Prevention Plans. The NPDES program for industrial stormwater discharges formalized on-site stormwater management programs with the requirement for a stormwater pollution prevention plan (SWPPP). The basic objectives of these plans are to:

- Quantify sources of water pollution from stormwater discharges
- Identify best management practices to prevent or reduce the impact of these discharges
- Provide practical guidance to effectively implement the plan, including compliance with any applicable permit conditions.

The SWPPP process begins with the selection of a pollution prevention team that includes representatives from significant operations areas and of different management levels. This team identifies areas of concern

for potential harmful discharges, develops procedures for best management practices, develops implementation strategies, including training as well as construction, evaluates the impacts of future facility operations on stormwater management, and monitors the overall effectiveness of the SWPPP.

Detailing for the SWPPP begins with an assessment phase to identify potential sources of pollution and a detailed site map to locate these sources and other site characteristics. The identification tasks include actual site field inspections and collection and review of available monitoring, planning, and other pertinent data. Site features and potential pollution sources for the site map are listed in Table 7.37.

Existing environmental management documents will provide important basic data and historical information that will become a part of the final SWPPP. For example, the spill prevention control and countermeasure plan (SPCC) should help to define areas of past and potential spills and routes of runoff from various areas. This will lead to a further review of current practices and possibly the installation of new or improved best management practices.

After the assessment of potential stormwater pollution problems, structural and nonstructural best management practices are evaluated for use in final SWPPP development. Structural BMPs include storage practices, such as extended-detention outlet structures; filtration practices, such as grassed swales; and infiltration practices, such as porous pavements. Nonstructural BMPs are preventive actions that involve management and source controls, including site-specific policies and procedures.

When considering an individual activity of a facility, the BMPs should first look to source control. The following is a priority listing of BMPs:

- Alter the activity by changing the process
- Enclose the activity to prevent exposure to rainfall and potential runoff
- Cover the activity to reduce runoff if the cost for enclosure is prohibitive
- Segregate the activity to a small area, making source control more feasible
- Discharge stormwater to the process wastewater collection and treatment works
- Discharge runoff from small, frequent storms to the sanitary sewer system
- Discharge runoff from small, frequent storms to a sump for subsequent treatment/disposal
- Discharge runoff from small, frequent storms to a stormwater treatment system

Focusing on nonstructural opportunities, the SWPPP program for a facility as a whole should consider the priority listing of BMPs in Table 7.38.

TABLE 7.37 Stormwater Pollution Prevention Site Mapping

-
- Property boundaries
 - Natural drainage features and boundaries
 - Proximity to surface water body
 - Unique features, such as on-site wetlands or public use areas
 - Impervious areas (pavement and buildings) and location of connections to drainage system
 - Stormwater collection system
 - Structural control measures, such as berms, oil/water separators, and detention facilities
 - Fuel and other liquid storage tanks
 - Fuel and product transfer areas
 - Processing areas
 - Material storage areas
 - Vehicle maintenance and fueling sites
 - Sites of previous leaks and/or spills
 - Areas for facility expansion
-

TABLE 7.38 Facility-Wide Best Management Practices

-
- Good housekeeping for source control and pollution prevention
 - Preventive maintenance of stormwater management facilities
 - Visual site inspection of active and passive stormwater management facilities
 - Visual inspection and maintenance of all facility equipment and machinery
 - Spill prevention and response procedures
 - Sediment and erosion prevention before, during, and after construction
 - Training in response to spills and stormwater
 - Record keeping and reporting of monitoring and management data
 - Nonstormwater discharge monitoring to ensure illicit discharges do not occur
 - Management of runoff by diversion, infiltration, recycle, reuse, or treatment
-

The effective implementation of the SWPPP requires periodic reviews and assessment by the pollution prevention team, review and evaluation of monitoring data and associated corrective actions, monitoring and maintenance of selected control measures, and timely updates of the plan and site map.

Local Government Stormwater Planning. Most local and regional governments should develop, implement, and enforce stormwater management plans. The plan objectives should include a program designed to protect water quality by reducing the discharge of pollutants from the local storm sewer system to the maximum extent practicable. In the United States, the Environmental Protection Agency has identified six minimum control measures for stormwater management programs subject to the NPDES permit program. Permit obligations of a small system for a minimum control measure may be performed by another governmental or other entity. Since the permit holder remains responsible for compliance, there should be a legally binding agreement with operating entity that minimizes any uncertainty regarding compliance with the NPDES permit.

These minimum control measures generally are applicable to all entities operating a storm sewer system and are: (1) public education and outreach on stormwater impacts, (2) public involvement/participation, (3) illicit discharge detection and elimination, (4) construction site stormwater runoff control, (5) postconstruction stormwater management in new development and redevelopment, and (6) pollution prevention/good housekeeping operations (118).

Public Education and Outreach on Stormwater Impacts. Local governments should carry out a public education program to distribute educational materials about the impacts of stormwater discharges on local water bodies and the steps to reduce stormwater pollution, such as ensuring proper septic system maintenance, limiting the use and runoff of garden chemicals to appropriate amounts, properly disposing of used motor oil or household hazardous wastes, and becoming involved in local stream restoration activities.

Other possible outreach programs should encourage citizens to participate in a local public service program by performing such services as roadside litter pickup and storm drain stenciling or highlight the potential public health risks to children if exposed to pollution when playing near storm drains. In addition, some programs should be directed toward targeted groups of commercial, industrial, and institutional entities likely to have significant stormwater impacts to explain their impacts on stormwater pollution, such as preceding information to restaurants on the impact of grease clogging storm drains and to garages on the impacts of used oil discharges.

Public Involvement/Participation. Public involvement is an integral part of an effective stormwater program. The public should participate as a partner in developing, implementing, and reviewing the stormwater management program. Opportunities for local citizen participation in program development and implementation could include serving as citizen representatives on a local stormwater management panel, attending public hearings, working as citizen volunteers to educate other individuals about the program,

helping in program coordination with other preexisting programs, or participating in volunteer monitoring efforts.

Illicit Discharge Detection and Elimination. Inflows from aging sanitary sewer collection systems are considered part of the illicit discharge-related problem for storm sewer systems. Sanitary sewer systems frequently develop leaks and cracks resulting in discharges of pollutants to receiving waters through separate storm sewers. These pollutants include sanitary waste and sewer main construction materials, such as asbestos cement, brick, cast iron, and vitrified clay. Local governments have long recognized the problems of stormwater infiltration/inflow into sanitary sewer collection systems, because this type of infiltration/inflow often disrupts the operation of the municipal wastewater treatment plant. However, the reverse problem of sewage exfiltration out of the sanitary sewer collection system into the storm water collection system can occur during dry weather periods.

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Discharges from stormwater drainage systems often include wastes and wastewater from nonstormwater sources. Results from a 1987 study conducted in Sacramento, California, revealed that slightly less than one-half of the water discharged from a separate storm sewer system was not directly attributable to precipitation runoff (119). A significant portion of these dry-weather flows results from illicit and/or inappropriate discharges and connections to the municipal separate storm sewer system. Illicit discharges enter the system through either direct connections, such as wastewater piping either mistakenly or deliberately connected to the storm drains, or indirect connections, such as infiltration into the storm drain system or spills collected by drain inlets.

Various forms of illicit or inappropriate discharges may enter a storm water collection system directly or indirectly and may be a continuous or intermittent discharge. The discharge characteristics, or contamination category, may range from a leaking potable water line to a toxic chemical hold tank spill. Table 7.39 provides a simple overview of typical pollutant sources and their most likely characteristics.

A simple flow sheet for the initial steps in the search for illicit and inappropriate discharges is shown in Figure 7.38. More specific investigation steps include:

- Drainage area mapping
- Visual inspection
- Tracer identification
- Field survey and data collection
- Analyses of data collected
- Categorization of outfalls
- Investigation and remediation
- Pollution prevention program

Construction Site Stormwater Runoff Control. Stormwater discharges generated during construction activities cause an array of water quality impacts, as the biological, chemical, and physical integrity of the waters can change significantly. In part, water quality impairment results because many pollutants are preferentially absorbed onto mineral or organic particles found in fine sediment washed from construction sites. The interconnected process of erosion, sediment transport and delivery is the primary pathway for introducing key pollutants, such as nutrients, metals, and organic compounds, into aquatic systems. Also, construction

TABLE 7.39 Potential Pollutant Entries into a Storm Sewer System (119)

Potential Source	Storm Drain Entry		Flow Characteristics		Contamination Category		
	Direct	indirect	Continuous	Intermittent	Toxic	Pathogenic/ Nuisance	Clear
Residential Areas:							
Sanitary Wastewater	X	x	X	x	X	x	
Septic tank effluent			X	x	X	x	
Household chemicals	x	X		X	X		
Laundry wastewater	X			X		X	
Excess landscaping watering		X		X	x	x	X
Leaking potable water pipes		X	X				X
Commercial Areas:							
Gasoline filling station	X	x		X	X		
Vehicle maintenance repair	X	x		X	X		
Laundry wastewater	X		X	x	x	X	
Construction site dewatering		X	X	x		X	
Sanitary wastewater	X	x	X		X		
Industrial Areas:							
Leaking tanks and pipes	x	X	X	x	X		
Miscellaneous process waters	X	x	X	x	X	x	x

X = most likely condition; x = may occur; blank = not very likely.

^a = direct, e.g., by piping; indirect, e.g., infiltration and spills.

sites may contribute other pollutants, such as used materials, landscape chemicals, sanitary wastes, or concrete truck washout.

In watersheds experiencing intensive construction activity, the localized impacts of water quality may be severe because of high pollutant loads, primarily sediments. Siltation is the second largest cause of impaired water quality in rivers and lakes. Introduction of coarse sediment (coarse sand or larger) or a large amount of fine sediment also is a concern because of the potential of filling lakes and reservoirs as well as clogging stream channels. Large inputs of coarse sediment into stream channels initially will reduce depth and minimize habitat complexity by filling pools (121).

Stormwater discharges from construction sites that occur when the land area is disturbed can severely affect designated uses. This particularly applies to waters designated for public water supply, recreation, and propagation of fish and wildlife. The siltation process may affect all three designated uses by:

- Depositing high concentrations of pollutants in public water supplies
- Decreasing the depth of a water body, which can result in its limited use by boaters, swimmers, and other recreational enthusiasts
- Directly impacting the habitat of fish and other aquatic species, which can limit their ability to reproduce

Excess sediment can cause a number of other problems, such as increased turbidity and reduced light penetration in the water column as well as more long-term effects associated with habitat destruction and increased difficulty in filtering drinking water supplies.

Generally, properly implemented construction site ordinances are effective in reducing these pollutants. In many areas, however, the effectiveness of ordinances in reducing pollutants is limited due to inadequate enforcement or incomplete compliance with such local ordinances, BMPs are not always properly main-

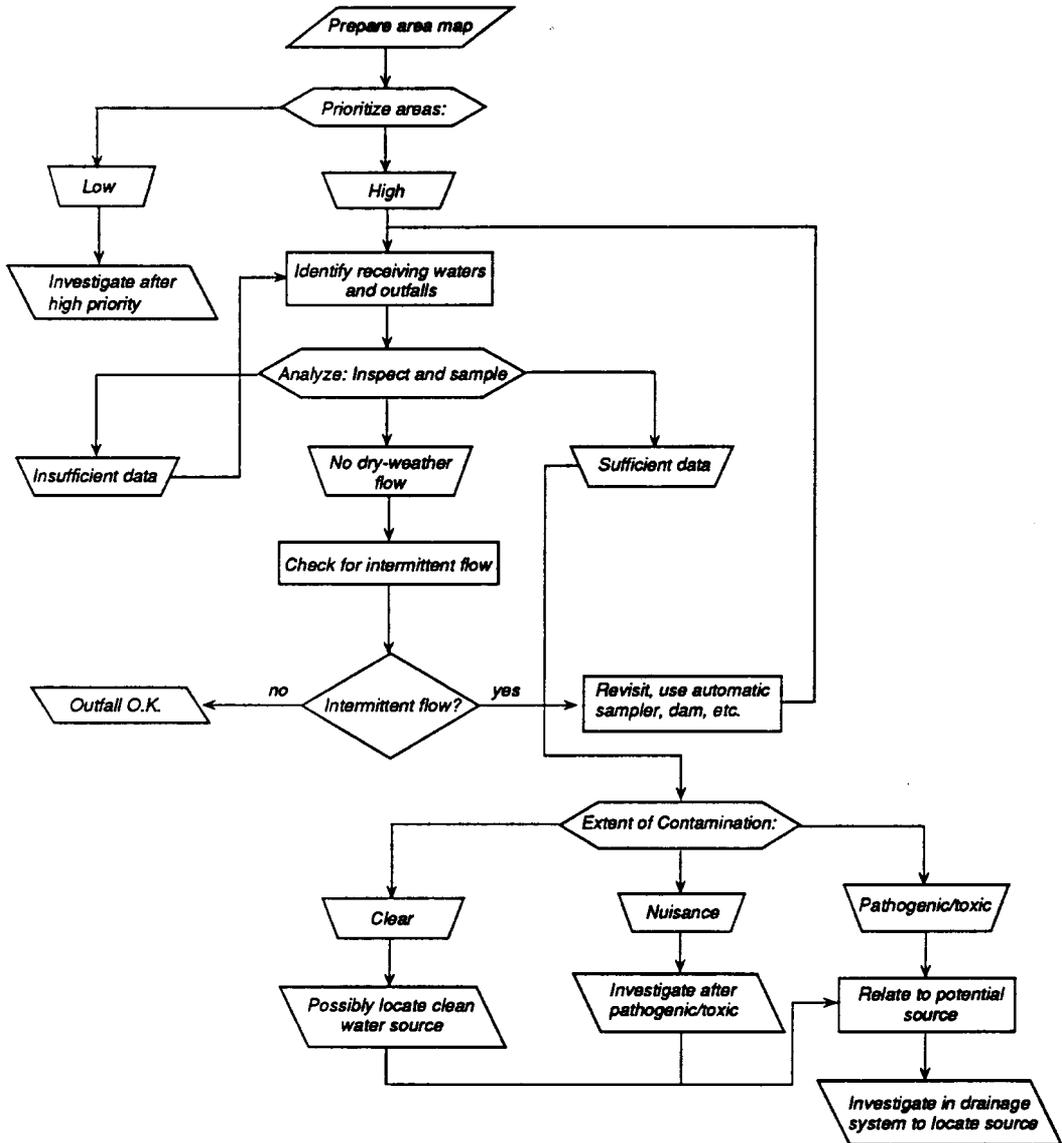


FIGURE 7.38 Illicit discharge investigation procedures (119).

tained. For example, sediment traps and sediment basins may fill and silt fencing may break or be overtopped.

Owners or operators of regulated small municipal separate storm sewer systems should develop, implement, and enforce a pollutant control program to reduce pollutants in stormwater runoff from construction activities disturbing one or more acres. An ordinance or other regulatory mechanism is needed to control erosion and sediment to the maximum extent practicable. The program also needs to ensure control of other

waste, such as discarded building materials, concrete truck washout, and sanitary waste, at construction sites that could adversely impact water quality. Of course, the program should include requirements for construction site owners or operators to carry out appropriate BMPs, such as silt fences, temporary detention ponds and hay bales; provisions for preconstruction review of site management plans; procedures for receipt and consideration of information provided by the public; regular inspections during construction; and penalties to ensure compliance.

Postconstruction Stormwater Management in New Development and Redevelopment. Planning and designing for the minimization of pollutants in stormwater discharges is the most cost-effective approach to stormwater quality management. Reducing the discharge of pollutants after the discharge enters a storm sewer system is not only generally more expensive but also is less efficient than preventing or reducing the discharge of pollutants at the source. Considering water quality impacts from the beginning stages of projects, new development, and possibly redevelopment, allows opportunities for more water quality sensitive projects. For example, minimization of impervious areas, maintenance or restoration of natural infiltration, wetland protection, use of vegetated drainage ways, and use of riparian buffers have been shown to reduce pollutant loadings in stormwater runoff from developed areas.

Pollution Prevention/Good Housekeeping Operations. Municipalities should develop and implement an operation and maintenance/training program with the ultimate goal of preventing or reducing pollutant runoff from municipal operations. The program considerations are:

- Maintenance activities, maintenance schedules, and long-term inspection procedures for structural and other stormwater controls to reduce floatables and other pollutants discharged from the separate storm sewers
- Controls for reducing or eliminating the discharge of pollutants from streets, roads, highways, municipal parking lots, maintenance and storage yards, and waste transfer stations, including programs that promote recycling and pesticide use minimization
- Procedures for the proper disposal of waste removed from the separate storm sewer systems and areas listed, including dredge spoil, accumulated sediments, floatables, and other debris
- Ways to ensure that new flood management projects assess the impacts on water quality and examine existing projects for incorporation of additional water quality protection devices or practices.

Watershed Stormwater Planning

An integrated watershed approach for stormwater and other discharges focuses on coordinated public and private sector efforts to address the highest-priority water quality problems within hydrologically defined geographic areas. This approach is a decision-making process reflecting a common strategy for information collection and analysis and a common understanding of the roles, priorities, and responsibilities of all stakeholders within a watershed.

The Environmental Protection Agency's watershed strategy integrates the NPDES program functions into a broader watershed protection approach and highlights areas for coordination with stakeholders to promote implementation of the approach. The NPDES Watershed Strategy is based on the principles presented in Table 7.40.

Water quality monitoring programs should take advantage of existing monitoring programs of both public and private entities. Typical watershed monitoring objectives are to:

- Characterize water quality and ecosystem health in a watershed overtime
- Determine causes of existing and future water quality and ecosystem health problems in a watershed and develop a watershed management program

TABLE 7.40 NPDES Watershed Strategy (118)

-
- Watershed protection approaches vary in terms of specific elements, timing, and resources, but all should share a common emphasis and insistence on integrated actions, specific action items, and measurable environmental and programmatic milestones.
 - A TMDL analysis is used to assess point, nonpoint, and background pollutant impacts on water quality within the watershed.
 - Related activities within a basin or watershed must be coordinated to achieve the greatest environmental benefit and most effective level of stakeholder involvement.
 - Actions relating to restoration and protection of surface water, groundwater, and habitat within a basin should be based upon an integrated decision-making process, a common information base, and a common understanding of the roles, priorities, and responsibilities of all stakeholders within a basin.
 - Staff and financial resources are limited and must be allocated to address environmental priorities as effectively and efficiently as possible.
 - Program requirements that interfere or conflict with environmental priorities should be identified and revised to the extent possible.
 - Accurate information and high quality data are necessary for decision making and should be collected on an incremental basis; interim decisions should be made based on available data to prevent further degradation and promote restoration of natural resources.
-
- Assess progress of the watershed management program or effectiveness of pollution prevention and control practices
 - Support documentation of compliance with permit conditions and/or water quality standards

Common Drainage Systems and Their Alternatives

Storm drainage systems are intended to carry off excess runoff at the minimum total cost. Designs are normally prepared so that extremely large storms may exceed the capacity of the system, but the more common storms will be carried within the system to minimize inconvenience. There are various systems in common use to accomplish this. They range from conventional storm sewer systems to culvert and swale systems.

Curb-and-Gutter Storm Sewer System. Conventional storm sewer systems consist of combinations of street gutters, inlets, storm sewers, and other appurtenances to collect and transport surface runoff off-site quickly. In heavily developed areas, roof drains are commonly connected directly to the sewers. In suburban areas, yards are usually graded to prevent ponding and to speed runoff to inlets. These systems can be very effective in removing runoff, but they can cause problems downstream by increasing peak flow rates. The advantages of conventional storm sewer systems are that they allow narrower road right of ways, create a traffic barrier on both sides of the roadway, and are compatible with conventional sidewalks.

Conventional curb-and-gutter storm sewer systems also have several disadvantages. During large storms, runoff is confined to the road surface and can create a traffic hazard. Since runoff is conveyed directly to sewers, no infiltration can occur and runoff volumes are increased. Any pollutants deposited on the road surface will be washed directly into storm sewers and will not have a chance to settle out. These pollutants may adversely affect water quality in streams and impoundments.

The design of storm sewer systems depends on their intended use. Roadways with a large amount of traffic require a greater level of protection. Typical design standards were listed previously.

Conventional storm sewer systems include the following elements:

- Surface swales and street gutters to collect surface runoff
- Underground closed conduits to convey runoff off-site
- Inlets to connect between these subsystems

Surface swales and street gutters are designed to minimize the encroachment of water onto roadways and property. Runoff water in street gutters is generally limited to a velocity of 10 ft/s (3.0 m/s) and a maximum depth of 6 in (15 cm) for safety reasons. The capacity of gutters may be estimated from Eq. (7.53), which is a modification of Manning's equation (81).

$$Q = \frac{X}{n} S_x^{1/2} y^{8/3} \quad (7.53)$$

where Q = Flow rate, ft³/s (m³/s)

S_x = Reciprocal of the pavement cross section, ft/ft (m/m)

n = Manning's coefficient (normally 0.015 for pavement)

s = Longitudinal gutter slope, ft/ft (m/m)

y = Depth of flow at curb, ft (m)

x = 0.562 (0.378 in SI system)

The maximum flow rate in street gutters is limited by placement of inlets. Cross-street flow should be avoided wherever possible. In highly concentrated business areas where there is a large amount of pedestrian traffic, inlets may be necessary at each corner. In general, ponding water or gutter flow greater than 2 ft (0.6 m) wide is difficult for pedestrians to cross.

Generally, there are three types of inlet systems: curb-opening inlets, grated inlets, and combination inlets. Each type of inlet may be installed with or without a depression of the gutter and may be a single or a multi-unit. Depressed inlets have a substantially higher capacity. Brief descriptions of each inlet type follow (81):

- *Curb-opening inlets* consist of a vertical opening in the curb through which the gutter flow passes.
- *Grate inlets* consist of an opening in the gutter covered by one or more grates.
- *Combination inlets* consist of both a curb-opening and a grate inlet acting as a unit.

Inlets may be placed on a continuous grade or at sumps. When placed on a continuous grade, the inlet may be designed to capture only a portion of the flow (usually greater than 90%) for greater economy. The portion bypassing should be accounted for at the next inlet. Inlets in sumps should be designed to capture 100% of the flow. Inlets are sized using commonly available nomographs (81). A typical nomograph is shown in Figure 7.39.

The closed conduits used to convey runoff away from the site are typically corrugated metal, precast concrete, or poured-in-place concrete. Other materials, such as vitrified clay, cast iron, and built-up pipe, have been used in the past. Where soils are corrosive, corrugated metal pipe is commonly coated with asphalt to extend the life of the pipe.

Underground conduits are generally laid with a constant vertical and horizontal alignment. Where changes in alignment are necessary, manholes are usually required. Access to the system through inlets, manholes, or headwalls is usually required every 300 to 400 ft (110 to 125 m) to allow maintenance. A minimum pipe diameter of 15 in (38 cm) is commonly required for pipes passing under public roadways, although a 12-in (30-cm) diameter may be allowed for pipes in other areas. A minimum cover of 18 in (46 cm) between the top of pipe and finished grade is commonly required to protect the pipe.

Detailed guidelines for the design of underground systems may be found in Ref. 33, including procedures for calculation of hydraulic grade lines and structural requirements.

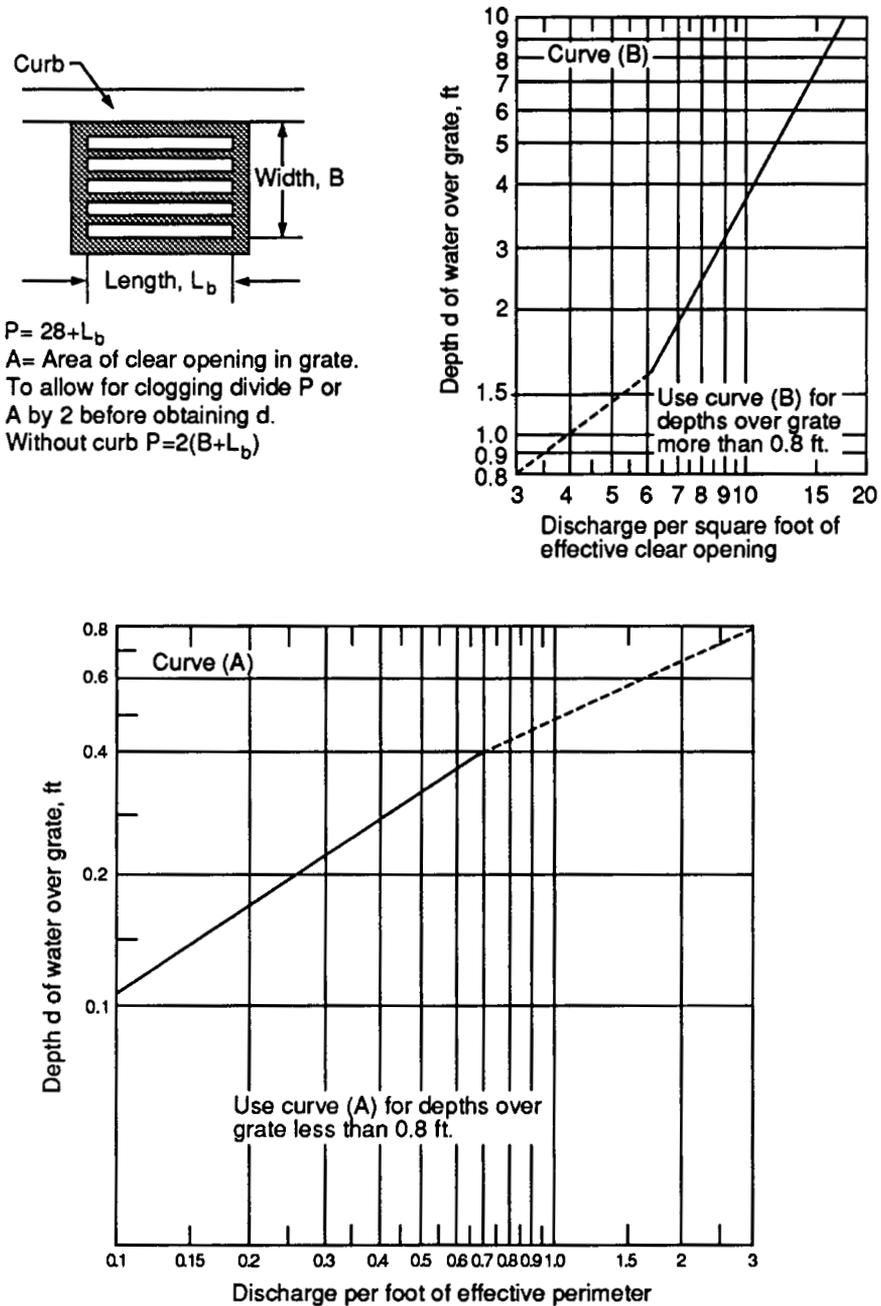


FIGURE 7.39 Hydraulic capacity of grate inlets in sump (81).

Culverts and Swales. Culvert and swale systems are most useful where land is not at a premium. These systems consist of an extensive network of open channels to collect and convey runoff; they are occasionally crossed by culverts. These systems use the capacity of natural surfaces to reduce runoff velocities, increase infiltration, and remove certain contaminants present in stormwater. The hydraulic capacity of a culvert can be limited by the capacity of the inlet to accept flow and/or the capacity of the pipe barrel to transport the flow. When flow capacity is limited by the inlet, the culvert is said to flow under inlet control. When flow capacity is limited by friction in the pipe barrel, the culvert is said to flow under outlet control. For culverts under inlet control, only the culvert diameter, inlet shape, and headwater depth affect flow capacity. For culverts under outlet control, the barrel roughness, barrel slope, and tailwater depth also affect flow capacity. For culverts in inlet control, it is frequently most economical to improve the inlet shape until flow is limited by the pipe barrel (82). Typical design nomograph for inlet and outlet control are shown in Figures 7.40 and 7.41 (83).

Swales can be used on grades from 1 to 10%. Design criteria for culvert and swale systems are less precise than for underground systems due to the difficulty in assessing an accurate value of Manning's n . Generally, side slopes of channels should not be greater than 3:1 to enable mowing.

Velocity is the critical parameter in the design of grass-lined channels, as vegetative covers are subject to damage due to erosion. Careful consideration must be given to the type of cover selected for use. Table 7.41 lists permissible velocities for different types of grass and soil erodability.

The higher velocities are permitted only where the sward is of very high quality. The amount of effort directed toward maintenance of the vegetative cover will also influence the selection of species. Table 7.42 lists permissible velocities for other types of natural channels. The type of grass selected for a lining must be adapted to the geographic area, and it must be compatible with other plant species in the landscape. The grass must also be tolerant to frequent submersion.

Grass-lined channels can be designed using Figure 7.42. A typical cross section is selected along with the type of grass cover. The degree of retardance is determined from Table 7.43 for the maximum anticipated grass height. An approximate hydraulic radius R and velocity V to carry the design flow are assumed and used to determine Manning's n . This n is then used to calculate the actual R and V . The process is repeated until the velocity agrees to the required accuracy. Once the proper channel dimensions are determined, this process should be repeated for the minimum grass height to determine the maximum anticipated velocity. Allowable velocities can be determined from Table 7.42.

Alternative Drainage Systems. Experience developed over the past 30 years has shown that there are significant problems with past stormwater control methods. In recent years, two new stormwater control concepts have come into common use: the natural drainage system concept and the major–minor system concept. These concepts are summarized below; details of these concepts are provided in Refs. 85 to 87.

Natural Drainage Concept. The natural drainage concept was first used during the planning of The Woodlands development outside Houston, Texas (85). One of the prime attractions of The Woodlands site was the existence of the only extensive natural forest in the region. The major complications of this project were the naturally poor drainage and frequent flooding of the area. Conventional drainage techniques could have been used but would have reduced infiltration, probably destroying the trees, which are a major attraction of the area. The technique selected attempted to minimize the disturbance caused by development and allow the natural drainage system to control surface runoff. The natural drainage concept was applied in the following manner (85, 86):

1. An extensive inventory of vegetation and soils was performed to identify natural infiltration rates. Intense development was concentrated in areas with a naturally low infiltration capacity while areas with a high infiltration capacity were preserved as open space.
2. No underground storm sewers were used. The existing drainage system was used to the fullest extent possible. Where drainage channels needed to be constructed, wide, shallow swales lined with native vegetation were used instead of cutting deep, narrow ditches.

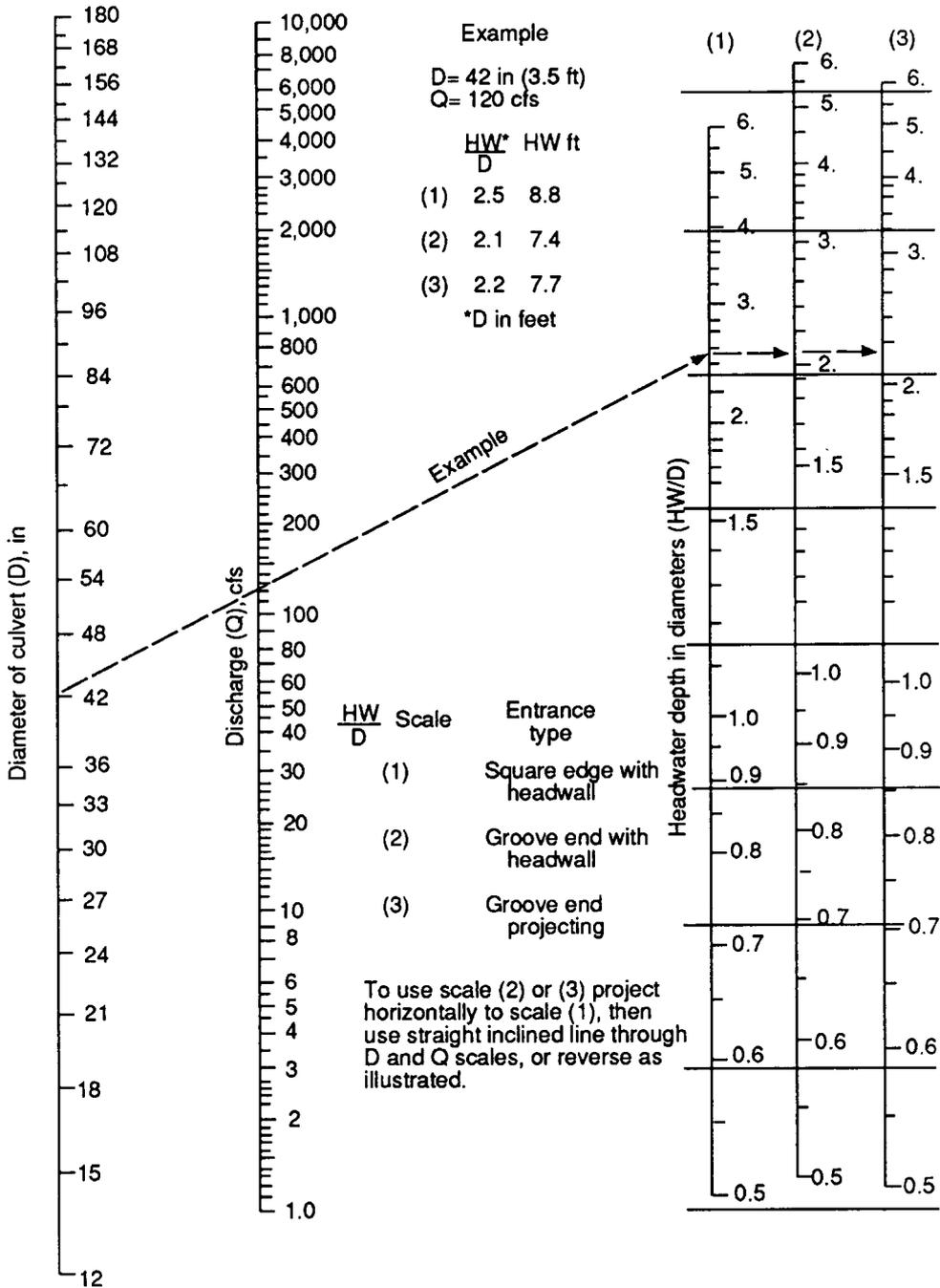


FIGURE 7.40 Headwater depth for concrete pipe culverts with inlet control (83).

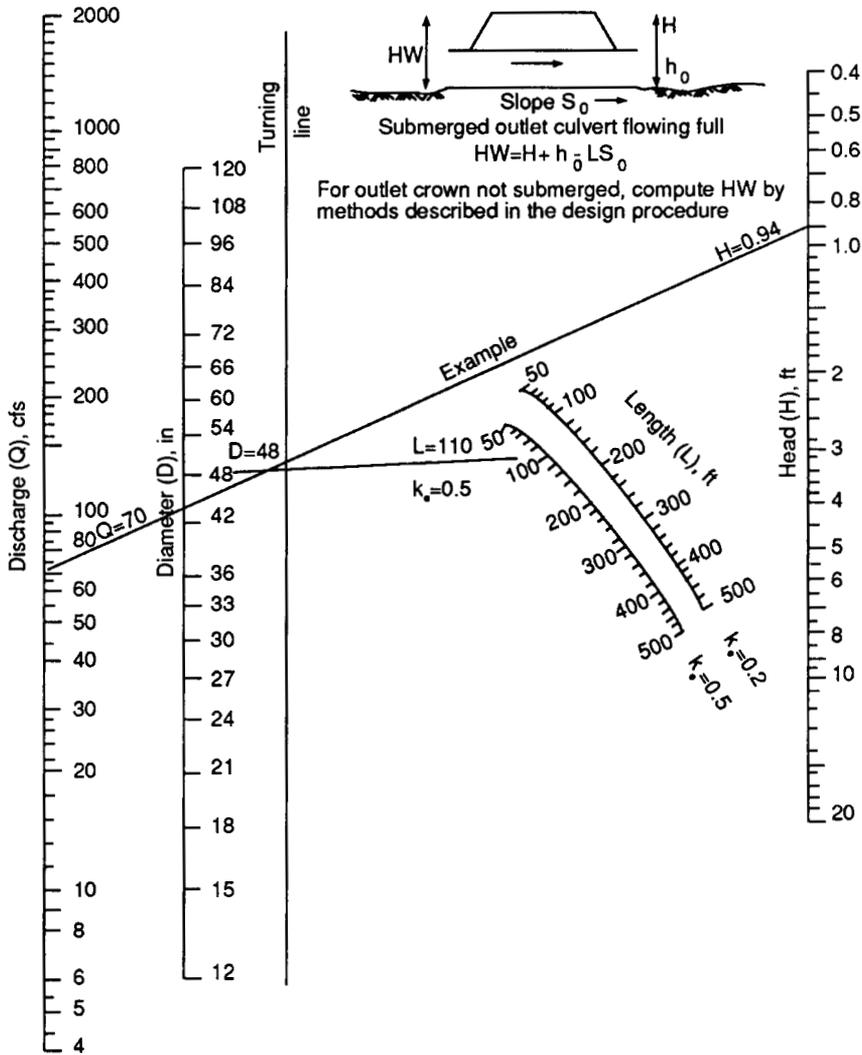


FIGURE 7.41 Head for concrete pipe culverts flowing full (83).

3. Site grading was minimized. No grading was allowed that would damage trees. Homes were constructed on pier-and-beam foundations to minimize damage to trees and prevent drainage problems.
4. In commercial areas where paving was required, detention ponds, infiltration pits, and porous paving were used extensively.

The development of The Woodlands demonstrated the feasibility of the natural drainage concept. The beginning phases of the project have been completed, many at a high density (6 individual homes per acre, 2.4 per hectare) and have sustained major storms (9 in (23 cm) over 5 hours) with no damage while adjacent developments were awash. The developer, Woodlands Development Corporation, expected to save

TABLE 7.41 Velocities for Vegetation Lined Channels (31)

Cover	Slope ^c range, %	Permissible velocity ^{a,b} , ft/s	
		Erosion-resistant soils	Easily eroded soils
Bermudagrass	0–5	8	6
	5–10	7	5
	over 10	6	4
Bahia	0–5	7	5
Buffalograss	5–10	6	4
Kentucky bluegrass	over 10	5	3
Smooth brome			
Blue grama			
Tall fescue			
Grass mixtures	0–5 ^c	5	4
Reed canarygrass	5–10	4	3
<i>Lespedeza sericea</i>	0–5 ^d	3.5	2.5
Weeping lovegrass			
Yellow bluestem			
Redtop			
Alfalfa			
Red fescue			
Common lespedez ^e	0–5 ^f	3.5	2.5
Sudangrass			

^aUse velocities exceeding 5 ft/s only where good covers and proper maintenance can be obtained.

^b1 ft/s = 0.30 m/s.

^cDo not use on slopes steeper than 10% except for vegetated side slopes in combination with a stone, concrete, or highly resistant vegetative center section.

^dDo not use on slopes steeper than 5% except for vegetated side slopes in combination with a stone, concrete, or highly resistant vegetative center section.

^eAnnuals—use on mild slopes or as temporary protection until permanent covers are established.

^fUse on slopes steeper than 5% is not recommended.

TABLE 7.42 Velocities for Roadside Drainage Channels (38)

Type of lining soils (earth, no vegetation)	Permissible velocity	
	m/s	ft/s
Fine sand (noncolloidal)	0.16	2.5
Sandy loam (noncolloidal)	0.16	2.5
Silt loam (noncolloidal)	0.91	3.0
Ordinary firm loam	1.07	3.5
Fine gravel	1.52	5.0
Stiff clay (very colloidal)	1.52	5.0
Graded, loam to cobbles (noncolloidal)	1.52	5.0
Graded, silt to cobbles (noncolloidal)	1.68	5.5
Alluvial silts (noncolloidal)	1.07	3.5
Alluvial silts (colloidal)	1.52	5.0
Coarse gravel (noncolloidal)	1.83	6.0
Cobbles and shingles	1.68	5.5
Shales and hardpans	1.83	6.0

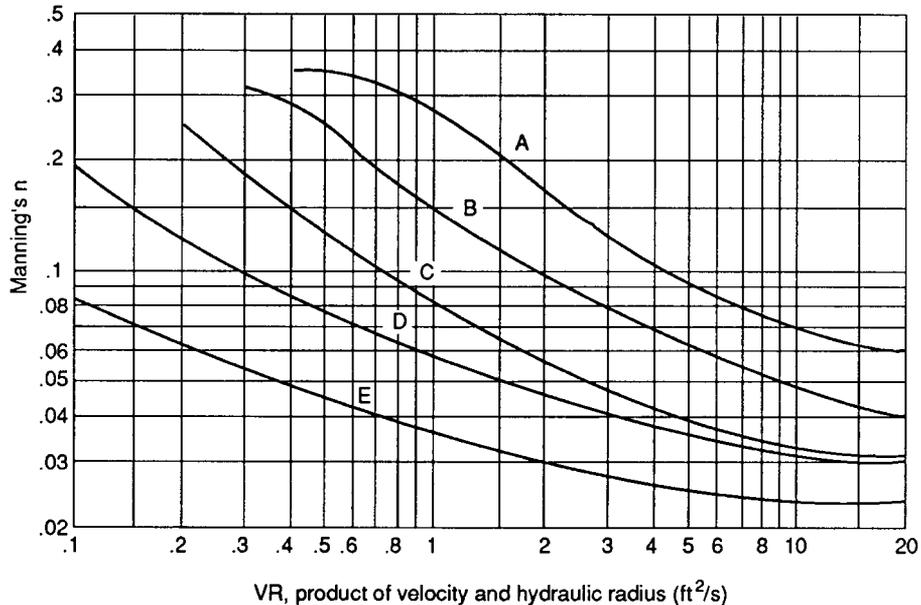


FIGURE 7.42 Manning's n related to velocity, hydraulic radius, and vegetal retardance ($1 \text{ ft}^2/\text{s} = 0.0929 \text{ m}^2/\text{s}$) (84).

\$14,500,000 of the \$18,500,000 originally estimated for a conventional storm sewer system by using the natural drainage concept (85).

Major–Minor Systems Concept. This concept was first used in the Denver area to eliminate serious flood damage while minimizing the cost of the storm drainage system (38). The minor drainage system, consisting of underground storm sewers and/or swales and culverts, is designed to transport the more frequent storms (5- or 10-year design frequency) and to minimize inconvenience to the public. The major system consists primarily of surface grading, shallow swales, and large natural channels. This system is designed to accept some inconvenience but to eliminate significant flood damage during large storms (typically 100-year design frequency).

When laying out the major and minor systems, the minor system is designed first. Inlets and conduits are sized for the minor system design storm. The major system design storm is then applied to the basin, and overland relief is provided where the capacity of the minor system is exceeded. Typical guidelines for design of the on-site major drainage system are as follows (87):

1. Areas should be graded and/or buildings located or constructed in such a manner that if a complete failure of the storm sewer system occurs, no building will be flooded by the design flow.
2. Key areas to watch are sump areas, relatively flat areas, and areas where buildings are located below streets and/or parking lots.
3. Where culverts cross large channels [drainage area greater than 30 acres (12 ha)], the low point of the roadway should be directly over the culvert. Where this is not convenient, an overland relief channel should be provided to return floodwater to the channel.
4. Use the 100-year frequency storm to compute runoff for the major drainage system.
5. For the first trial, use the same time of concentration values that were used in designing the minor

TABLE 7.43 Classification of Vegetation Cover as to Degree of Retardance (31)

Retardance	Cover	Condition
A	Reed canarygrass	Excellent stand, tall (average 36 in)
	Yellow bluestem ischaemum	Excellent stand, tall (average 36 in)
	Smooth bromegrass	
	Bermudagrass	Good stand, mowed (average 12–15 in)
	Native grass mixture (little bluestem, blue grama, and other long and short midwest grasses)	Good stand, tall (average 12 in)
B	Tall fescue	Good stand, unmowed
	<i>Lespedeza sericea</i>	Good stand, unmowed (average 18 in)
	Grass–legume mixture—timothy, smooth bromegrass, or orchard grass	Good stand, not woody, tall (average 19 in)
	Reed canarygrass	Good stand, uncut (average 20 in)
	Tall fescue, with bird’s foot trefoil or lodino	Good stand, mowed (average 12–15 in)
	Blue grama	Good stand, uncut (average 18 in)
	Bahia	Good stand, uncut (average 13 in)
	Bermudagrass	Good stand, uncut (6–8 in)
	Redtop	Good stand, mowed (average 6 in)
	Grass–legume mixture—summer (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, headed (15–20 in)
C	Centipede grass	Good stand, uncut (6–8 in)
	Kentucky bluegrass	Very dense cover (average 6 in)
	Bermudagrass	Good stand, headed (6–12 in)
	Red fescue	Good stand, cut to 2.5-in height
	Buffalograss	Good stand, headed (12–18 in)
D	Grass–legume mixture—fall, spring (orchard grass, redtop, Italian ryegrass, and common lespedeza)	Good stand, uncut (3–6 in)
	<i>Lespedeza sericea</i>	Good stand, uncut (4 to 5 in)
	Bermudagrass	After cutting to 2-in height. Very good stand before cutting.
E	Bermudagrass	Good stand, cut to 1.5-in height.
	Bermudagrass	Burned stubble.

drainage system and assume the minor system completely inoperable. If no building will be flooded based on these assumptions, then the analysis can be considered complete.

6. If buildings will be flooded based on the assumptions used in item 5, then the designer should perform more precise hydrologic and hydraulic computations. He or she should design the minor system, overland relief swales, and/or surface storage in such a way that no building will be damaged by flooding. Special precautions should be taken to prevent blockage of the minor systems.
7. The minor storm drainage system should not be oversized as a design for the major system. The major drainage system should be in the form of grading of the area and/or locating and constructing buildings in such a manner that overland relief swales and/or surface storage will accomplish the objective.

Infiltration. Infiltration measures reduce surface runoff by recharging stormwater into the ground. Infiltration can also decrease the cost of a conventional drainage system, improve water quality, and increase dry-weather stream flows.

There are essentially three ways to improve the infiltration capacity of a site: (1) use materials that allow water to directly infiltrate into the soil; (2) divert excess runoff to large porous areas during the storm event; and (3) collect excess runoff in a storage chamber and allow runoff to percolate into soil after the storm has passed. All infiltration techniques use one or more of these methods to infiltrate excess stormwater into the soil.

Infiltration Design Criteria. Soil permeability and depth to bedrock are the primary limitations to the widespread use of infiltration. Soil permeability requirements vary, but 0.6 in/h (1.5 cm/h) is normally required at a minimum. This permeability should be measured on site by percolation tests typically used to design septic tank systems. The “perc” test should be run on the soil horizon with the minimum permeability. The soil horizon of minimum permeability can be determined by excavating a small pit to visually examine the soil profile. Examination of the profile will also indicate the presence of bedrock or a seasonally high water table. The minimum depth to bedrock should be 5 ft (1.5 m).

A saturated soil condition should normally be assumed when a bearing surface is constructed over infiltration areas. In the absence of field data, a California Bearing Ratio (CBR) of 2.0 is frequently assumed.

Infiltration measures should be designed to allow bypassing of runoff during extreme storms or when the facility clogs. This bypass is normally provided by overland relief.

The required hydraulic capacity for infiltration practices depends on a number of factors including the intended use, total quantity of rainfall, time distribution of rainfall, and ratio of rainfall to surface runoff. Infiltration measures are most commonly used for pollutant removal and control of storms less than a 10-year design frequency. Control of storms greater than the 10-year storm is frequently accomplished by supplementing infiltration practices with detention measures that reduce the required volume of underground storage.

The design frequency and duration of rainfall must be known to determine the required storage volume. Design frequency will normally be specified by a government agency. Common practice is to determine the required storage volume based on one rainfall duration and the time of concentration at the site. This procedure often results in an infiltration practice with too little storage volume. If a practice is designed with too little storage, the storage volume will be filled during the beginning of the storm and the hydrograph peak may pass completely unaffected. This problem can be eliminated by calculating the required storage volume for several rainfall durations. This can be accomplished using the SCS 24-h storm distributions. These distributions are constructed to include all rainfall durations from 5 min to 24 h. The development of the SCS 24-h distributions is described in the section on rainfall. Techniques for generating hydrographs may be found in the runoff processes subchapter under TR-55.

Pollutant Trap Efficiency. The pollutant trap efficiency of infiltration measures has been reported to be high (up to 100% pollutants trapped) for the fraction of runoff that actually enters the soil. Zero pollutant removal is typically assumed for runoff that bypasses the practice. In order to determine the optimum storage volume for pollutant trap efficiency, the Northern Virginia Planning District Commission (NVPDC) estimated annual pollutant trap efficiencies for various combinations of soil type, land use and storage volume (88). The trap efficiencies were determined using a continuous simulation model run for a typical year rainfall in the Washington, D.C., area. Estimated trap efficiencies are shown in Tables 7.44 to 7.49. These trap efficiencies were calculated based on the following pollutant removal rates for water that actually enters the soil profile:

Total suspended solids: 99%
BOD: 90%
Total phosphorus: 65%
Total nitrogen: 45%
Extractable lead: 95%
Extractable zinc: 95%

Different soil permeabilities will result in differing drawdown periods for the same storage volume. The NVPDC's modeling produced the following drawdown times for infiltration practices:

One-day drawdown: 3.5 in/h (8.9 cm/h)
Three-day drawdown: 1.2 in/h (3.1 cm/h)

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TABLE 7.44 Projected Average Annual Pollutant Removal Rates for Infiltration Practices: Large Lot Single-Family Land Use (88)

Storage capacity, in (cm)	Drawdown time, days	Annual sediment removal, %	Annual total P removal, %	Annual total N removal, %	Annual BOD removal, %	Annual lead removal, %	Annual zinc removal, %
0.20 (0.51)	1	94	57	42	81	92	89
	3	89	52	37	74	90	85
	5	86	49	35	68	88	82
	7	79	44	32	61	49	76
0.10 (0.25)	1	73	44	34	58	87	72
	3	62	35	27	42	81	62
	5	59	31	24	37	77	58
0.07(0.18)	1	64	38	30	48	83	64
	3	54	29	24	34	76	55
	5	46	24	19	26	65	45
	7	38	19	16	21	55	38
0.05 (0.13)	1	53	31	26	36	77	54
	3	46	24	20	23	67	46
	5	36	18	15	19	53	36
	7	30	14	12	15	39	26
0.02 (0.05)	1	37	20	17	25	55	38
	3	28	12	10	15	40	27
	5	21	9	7	11	28	19
	7	13	6	5	7	19	13

TABLE 7.45 Projected Average Annual Pollutant Removal Rates for Infiltration Practices: Medium-Density, Single-Family (88)

Storage capacity, in (cm)	Drawdown time, days	Annual sediment removal, %	Annual total P removal, %	Annual total N removal, %	Annual BOD removal, %	Annual lead removal, %	Annual zinc removal, %
0.37 (0.94)	1	97	60	45	85	93	93
	3	94	56	42	80	90	89
	5	91	54	40	76	87	87
	7	88	51	38	70	84	84
0.21 (0.53)	1	88	53	41	72	86	85
	3	81	47	36	60	79	78
	5	79	44	34	57	78	76
	7	75	42	32	53	70	53
0.15 (0.38)	1	81	48	38	62	80	79
	3	75	42	33	53	74	73
	5	75	40	31	50	73	72
	7	69	37	29	45	55	38
0.10 (0.25)	1	72	42	34	51	72	71
	3	68	37	30	44	67	66
	5	64	33	26	39	62	60
	7	54	28	17	33	39	26
0.05 (0.13)	1	53	29	24	38	53	52
	3	46	23	18	30	46	45
	5	37	18	14	23	38	37
	7	32	14	11	19	19	13

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TABLE 4.46 Projected Average Annual Pollutant Removal Rates for Infiltration Practices: Townhouse/Garden Apartments (88)

Storage capacity, in (cm)	Drawdown time, days	Annual sediment removal, %	Annual total P removal, %	Annual total N removal, %	Annual BOD removal, %	Annual lead removal, %	Annual zinc removal, %
0.46 (1.17)	1	96	61	43	83	91	91
	3	89	56	40	73	86	86
	5	87	53	38	69	83	83
	7	85	51	37	66	81	81
0.33 (0.84)	1	89	56	41	73	86	86
	3	81	49	36	60	78	78
	5	80	48	35	58	77	77
0.21 (0.53)	7	77	46	34	55	74	74
	1	80	49	36	60	78	78
	3	75	44	33	51	72	72
0.14 (0.56)	5	74	42	32	49	71	71
	7	69	39	30	45	66	66
	1	70	42	32	49	70	70
0.07 (0.18)	3	67	38	29	44	65	65
	5	63	35	26	40	60	60
	7	57	32	24	36	53	53
	1	51	28	22	36	50	50
	3	46	24	18	30	45	45
	5	42	21	16	26	40	40
	7	37	17	13	22	34	34

TABLE 7.47 Projected Average Annual Pollutant Removal Rates for Infiltration Practices: High-Rise Residential (88)

Storage capacity, in (cm)	Drawdown time, days	Annual sediment removal, %	Annual total P removal, %	Annual total N removal, %	Annual BOD removal, %	Annual lead removal, %	Annual zinc removal, %
0.82 (2.08)	1	99	65	40	89	94	94
	3	97	64	39	87	92	92
	5	96	63	38	86	91	91
	7	94	61	37	84	90	90
0.54 (1.37)	1	96	64	40	88	93	93
	3	92	61	38	84	89	90
	5	92	60	38	83	89	89
	7	90	59	37	82	87	87
0.34 (0.86)	1	92	62	39	85	90	91
	3	89	58	37	82	86	87
	5	88	57	36	80	85	85
	7	80	51	32	72	77	77
0.23 (0.58)	1	84	55	35	78	83	83
	3	79	51	32	73	78	78
	5	76	48	30	68	73	73
	7	68	42	27	61	65	65
0.11 (0.28)	1	61	38	23	57	60	61
	3	57	34	21	52	56	56
	5	54	31	18	48	51	51
	7	47	26	16	42	45	45

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TABLE 7.48 Projected Average Annual Pollutant Removal Rates for Infiltration Practices: Industrial Land Use (88)

Storage capacity, in (cm)	Drawdown time, days	Annual sediment removal, %	Annual total P removal, %	Annual total N removal, %	Annual BOD removal, %	Annual lead removal, %	Annual zinc removal, %
0.82 (2.08)	1	99	65	40	89	94	94
	2	97	64	39	87	92	92
	3	96	63	38	86	91	91
	4	94	61	37	84	90	90
0.54 (1.37)	1	96	64	40	88	93	93
	3	92	61	38	84	89	90
	5	92	60	38	83	89	89
0.34 (0.86)	7	90	59	37	82	87	87
	1	92	62	39	85	90	91
	3	89	58	37	82	86	87
0.23 (0.58)	5	88	57	36	80	85	85
	7	80	51	32	72	77	77
	1	84	55	35	78	83	83
0.11 (0.28)	3	79	51	32	73	78	78
	5	76	48	30	68	73	73
	7	68	42	27	61	65	65
0.11 (0.28)	1	61	38	23	57	60	61
	3	57	34	21	52	56	56
	5	54	31	18	48	51	51
	7	47	26	16	42	45	45

TABLE 7.49 Projected Average Annual Pollutant Removal Rates for Infiltration Practices: Commercial Land Use (88)

Storage capacity, in (cm)	Drawdown time, days	Annual sediment removal, %	Annual total P removal, %	Annual total N removal, %	Annual BOD removal, %	Annual lead removal, %	Annual zinc removal, %
1.10 (2.79)	1	99	65	42	90	95	95
	3	98	65	42	89	93	93
	5	98	65	41	87	92	92
	7	96	64	40	85	91	91
0.78 (1.98)	1	99	65	42	89	94	94
	3	97	64	40	86	91	91
	5	96	64	39	85	90	90
	7	95	62	39	83	88	88
0.50 (1.27)	1	97	65	40	87	92	92
	3	95	63	39	84	89	89
	5	94	62	38	83	88	88
	7	88	58	36	78	82	82
0.33 (0.84)	1	91	61	37	82	87	87
	3	86	56	34	76	82	82
	5	85	54	34	74	78	78
	7	75	48	30	65	69	69
0.16 (0.41)	1	68	43	26	62	66	66
	3	65	40	24	57	61	61
	5	61	36	22	52	56	56
	7	56	32	19	47	51	51

Five-day drawdown: 0.7 in/h (1.8 cm/h)
 Seven-day drawdown: 0.5 in/h (1.3 cm/h)

Porous Asphalt Paving. The use of porous asphalt pavement in place of conventional paving materials is relatively new and represents a significant departure from conventional asphalt pavement design. Many questions have been posed as to the long-term operation and maintenance of porous pavements.

Extensive laboratory and some limited field testing have been performed on porous pavement (89). Results indicate that porous pavement has potential for a variety of applications. The following is a discussion of some of the major factors which should be considered in determining the applicability of porous pavement. In general, porous pavement is composed of four layers (89):

1. Minimally compacted subbase consisting of undisturbed existing soil.
2. Reservoir base course consisting of 1 to 2 in (2.54 to 5.08 cm) diameter crushed stone aggregate. The thickness of this layer is determined from runoff storage needs.
3. Two inches (5 cm) of 0.5-in (1.3-cm) crushed stone aggregate to stabilize the reservoir base course surface.
4. Porous asphalt concrete surface course whose thickness is based on bearing strength and pavement design requirements. In most applications, 2.5 in (6.35 cm) has been found to be sufficient.

A typical porous asphalt pavement cross section is presented in Figure 7.43.

The design of porous asphalt concrete roadways equivalent to conventionally constructed roads depends primarily on the load-bearing capacity of the subgrade, the expected traffic volume, and the reservoir capacity of surface and base. Specifications of the Asphalt Institute were used to design the porous pavement roadways listed in Table 7.50 (90).

Because of the low load-bearing capacity of a wet subgrade, a very poor subgrade (California Bearing Ratio = 2) was assumed in establishing these designs. Traffic volumes are indicated by the design traffic number (DTN), which is the average number of equivalent 18,000-lb (8165-kg) single-axle loads per day expected in the heaviest traffic lane during the design period (normally 20 years). The method of calculating the DTN is given in detail in the Asphalt Institute Manual Series No. 1, "Thickness Design." The classifications are as follows:

DTN	Traffic classification
1–10	Light (parking lots, residential streets)
10–100	Medium (city business streets)
100–10,000	Heavy (highways)

The gravel base depths are minimums provided by Asphalt Institute specifications. These minimum base depths may be increased to provide additional storage capacity.

The reservoir capacity figures given in Table 7.51 are based on an assumed average void space of 15% for the surface course and 30% for the base course. These are conservative averages.

Soil permeability should be determined by field percolation tests. The minimum allowable percolation rate should be 1 in/day (2.5 cm/day). Acceptable soils to resist frost heaving have been defined by the Asphalt Institute as soils with a subgrade where the total clay and silt content is less than 40% by weight (90).

Concrete Grid Pavement. Concrete grid pavement is composed of precast reinforced modular concrete pavers. A typical paver is shown in Figure 7.44. These pavers are placed on a compacted granular base (Figure 7.45) to provide a permeable, flexible pavement.

Concrete grid pavement is most useful in areas where some type of pavement is necessary but the traffic

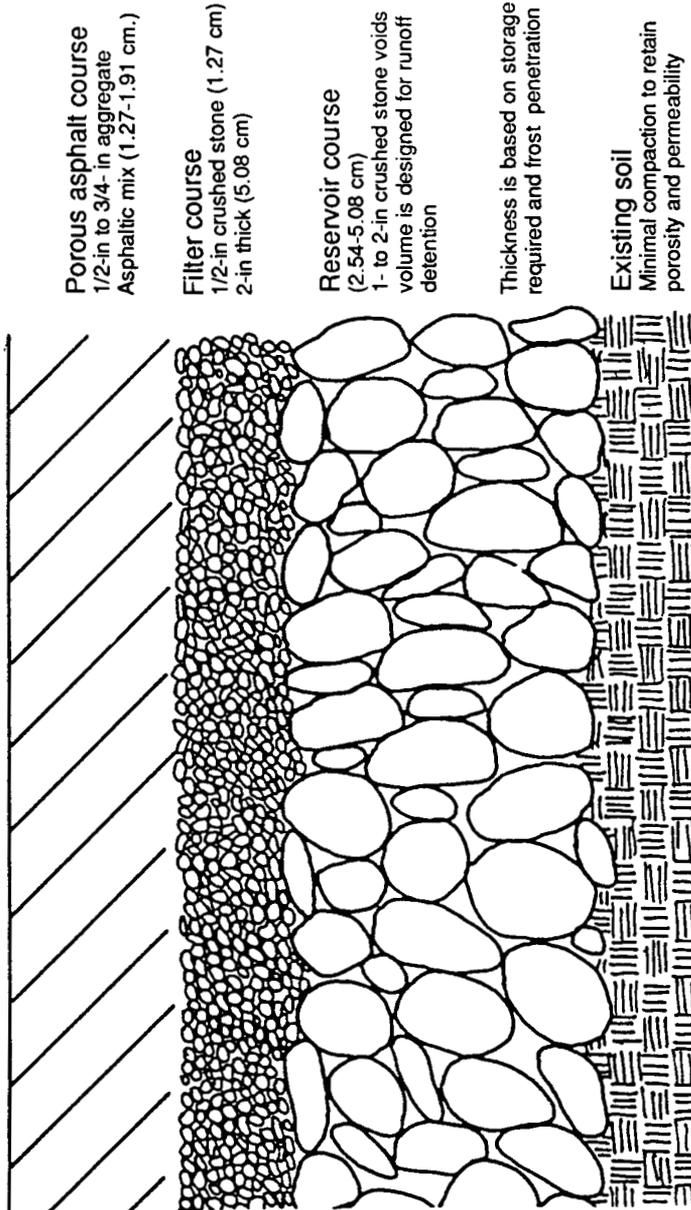


FIGURE 7.43 Porous asphalt paving, typical section (89).

TABLE 7.50 Requirements for Porous Pavement (91)

CBR*	DTN [†]	Surface thickness (in) [‡]	Base thickness (in)	Reservoir capacity, in of rainfall		
				Surface	Base	Total
2	1	4	6	0.60	1.80	2.40
2	10	4	12	0.60	3.60	4.20
2	20	4.5	13	0.66	3.90	4.56
2	50	5	14	0.75	4.20	4.95
2	100	5	16	0.75	4.80	5.55
2	1000	6	20	0.90	6.00	6.90
2	5000	7	22	1.50	6.60	7.65

*California bearing ratio, a measure of the soil support value.

[†]Design traffic number, a measure of traffic volume.

[‡]Asphalt Institute specification, assume a dense graded mix.

Note: 1 in 2.54 cm.

TABLE 7.51 Sedimentation Basin Requirements (100)

State	Required design storage capacity
Maryland	0.5 in (1.27 cm) over total drainage 0.2 in (0.51 cm) over total drainage*
Kentucky	0.2 ft (6.1 cm) over area disturbed
West Virginia	0.125 ft (3.8 cm) over area disturbed [†]
Pennsylvania	$V = (A/C) + (A^{1/3}C)^{‡}$

*To be cleaned when storage capacity drops below 2 in (0.5 cm)

[†]To be cleaned when sediment accumulation approaches 60% of design capacity.

[‡] V = volume; A = area drained; I = 24-h rainfall; C = runoff coefficient.

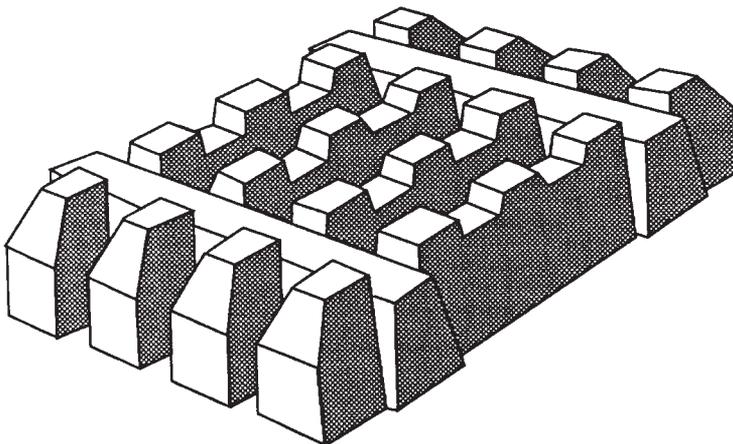


FIGURE 7.44 Typical castellated grid paver (106).

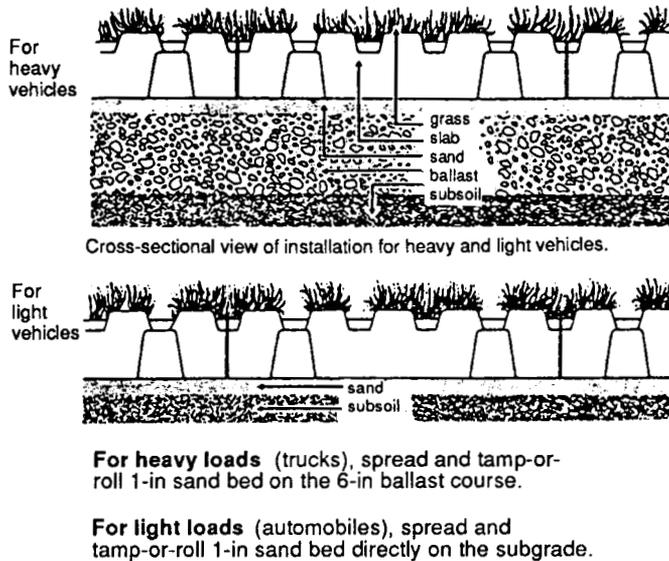


FIGURE 7.45 Bedding practice for light and heavy loads (92).

volume is low, such as auxiliary parking areas, recreational areas, fire lanes, or bikeways. Actual field data on infiltration capacity are limited. Infiltration capacity at a specific site depends on the natural permeability of the subsoil, degree of compaction of subsoil, and void spaces of gravel base course.

Infiltration Pits and Trenches. An infiltration pit or trench consists of a small excavation filled with crushed stone. The stone is commonly 1.5 to 3 in (3.8 to 7.6 cm) in diameter and graded as uniformly as possible to provide maximum void space. Typical stone of this diameter can be assumed to have approximately 40% voids. In order to reduce maintenance and safety problems, a number of modifications to this basic design have been developed.

1. The crushed stone is wrapped in a layer of heavy-duty filter fabric to prevent clogging with sediment. An extra layer of filter fabric is placed on top of the gravel to allow easy replacement.
2. A small sediment trap is provided upstream of the infiltration pit to preclean the stormwater.
3. The ratio of side area to bottom area should not exceed 4:1.
4. Traffic should be kept out of the pit for safety and to prevent excessive compaction of the crushed stone.

Detention. Stormwater detention practices are incorporated into the drainage system to reduce the peak rate of discharge and to possibly reduce the capital cost of the total drainage system. Detention facilities operate by storing excess flow during the most intense portion of the storm and then releasing this flow as conveyance capacity in the drainage system becomes available. Figure 7.46 shows typical stormwater runoff hydrographs for an uncontrolled site and a controlled site with a detention pond.

Detention Basin Design Criteria. In recent years, questions have arisen as to the effectiveness of detention basins in reducing downstream flooding and the possibility that they may actually aggravate flooding. Early modeling studies (93) indicated that detention practices placed in the lower portion of a large watershed could delay the hydrograph peak from a subbasin long enough to coincide with the hydrograph peak of

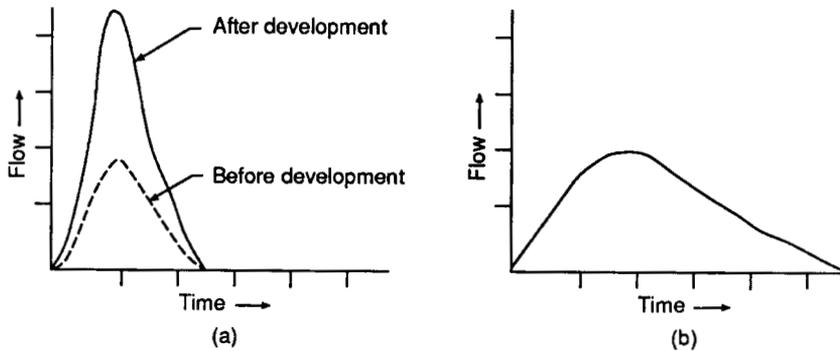


FIGURE 7.46 Typical stormwater hydrographs. (a) Without detention basin; (b) with detention basin.

the large watershed and actually increase the maximum discharge. More recent studies (94, 95) have confirmed that this situation is possible but can be easily avoided if detention practices are placed only on the main channel or within subbasins in the upper portion of the large watershed.

A more complex problem is to determine the optimum combination of controls to reduce flood damages. When a municipality is considering constructing detention storage as a flood control measure, modeling of the watershed will normally be required to determine the combination of detention location, storage volume, and release rate to reduce flooding at the minimum cost. Modeling will be necessary since the timing of a subbasin hydrograph (location within watershed) greatly affects the significance of that basin in downstream flooding.

When stormwater detention is required for new developments, it is usually not possible to analyze the effects of individual storage facilities on the entire basin, and therefore, standard design criteria are necessary. A common practice is to require that the postdevelopment peak runoff rate not exceed the predevelopment peak runoff rate at the development property line. This criterion is intended to maintain peak flow rates in downstream channels at existing levels; however, in practice it does not. The reason for this failure is described in Ref. 96 and is illustrated in Figures 7.47 to 7.49. Assume that subbasin 3 in Figure 7.47 is developed so that the uncontrolled peak outflow rate increases from 500 to 600 ft³/s (14 to 17 m³/s) (Figure 7.48). In order to meet typical regulations, a detention basin is constructed to limit the peak outflow to 500 ft³/s (14 m³/s). As shown in Figure 7.49, the downstream peak flow is still increased. The release rate of a detention pond must be less than the predevelopment peak flow to prevent an increase in the downstream peak flow, and to account for the longer duration of high discharge.

There are two approaches in use which reduce the potential for increased downstream flooding while at the same time providing a standard criterion for the design of detention storage. The first approach was developed for Allegheny County, Pennsylvania (96, 97), and requires that the maximum discharge from a detention structure be a fixed percentage of the predevelopment peak runoff rate. This percentage is determined by first modeling the watershed to determine the size and timing of subbasin hydrographs as they reach critical points on the downstream channel. Ratio of the flow rate contribution of each subbasin to the peak flow rate at a point on the downstream channel is used to calculate the allowable release rate percentage by the following equation:

$$\frac{\text{Subbasin contribution to downstream peak flow}}{\text{Subbasin maximum outflow rate}} = \frac{\text{release rate}}{\text{percentage}} \times 100 \quad (7.54)$$

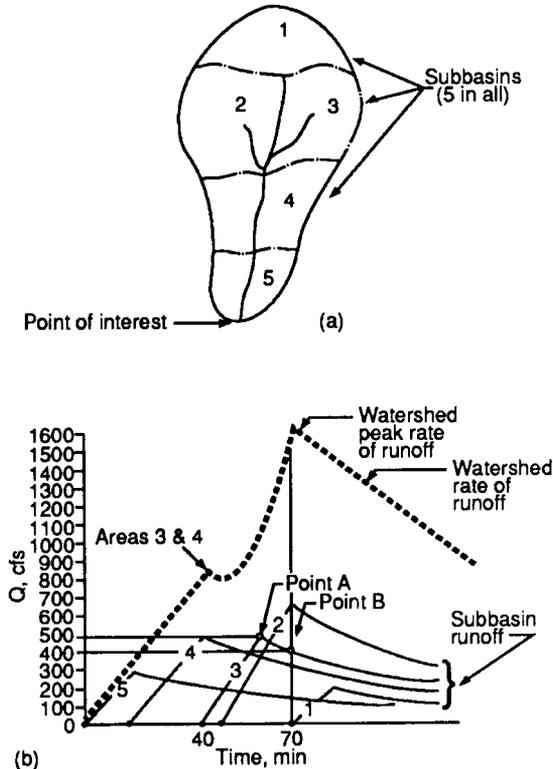


FIGURE 7.47 Watershed hydrograph, before development
(a) Watershed plan; (b) watershed hydrograph (96).

Release rate percentages are determined for various points on the downstream channel and used to determine the critical release rate for a given subbasin. These critical release rates are then specified as part of a stormwater detention ordinance using an overlay map for the watershed.

The release rate percentage concept is a definite advance over previous detention criteria, but it does retain some weaknesses. When applying this concept, individual developers calculate the predevelopment peak runoff rate. This runoff rate depends on the time of concentration T_c and consequently on the size of a development's subbasin. An example can best illustrate the problems in this approach. Two medium-density housing developments are constructed in an area with an allowable release rate percentage of 85%. Each development increases the design storm runoff from 0.5 in (1.3 cm) (pasture) to 1.0 in (2.54 cm) (developed). Development *T* covers approximately 10 acres (4 ha), has a T_c of 0.1 h and a peak undeveloped outflow of 0.77 ft³/s/acre (0.054 m³/s/ha). Development *TT* covers 500 acres (200 ha), has a T_c of 0.3 h, and a peak undeveloped outflow of 0.5 ft³/s/acre (0.0357 m³/s/ha). Development *T* will be required to reduce outflow to 0.66 ft³/s/acre (0.0462 m³/s/ha) while development *TT* will be required to reduce outflow to 0.44 ft³/s/acre (0.0308 m³/s/ha). At the critical point downstream ($T_c = 2.0$ h), development *T* will be contributing 16% more flow per area yet there will be no significant difference in the timing of the two hydrographs. One solution to this inequity would be to specify the design T_c along with the release rate percentage on the watershed overlay map.

A second possible criterion for detention storage design was developed in Independence, Missouri (95). The maximum discharge rate from a detention basin is controlled to less than or equal to the predevelopment

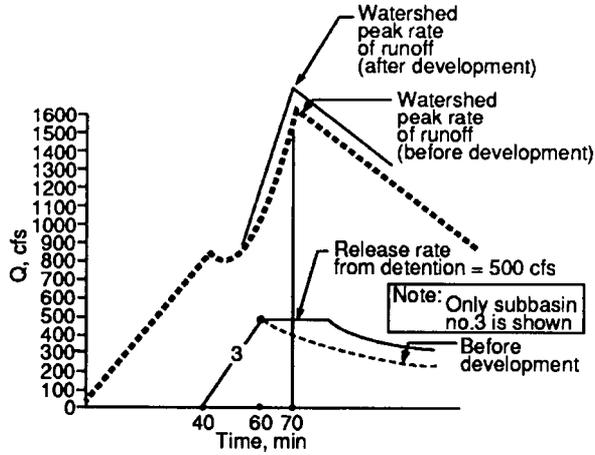


FIGURE 7.49 Subbasin hydrograph, after development with detention (96).

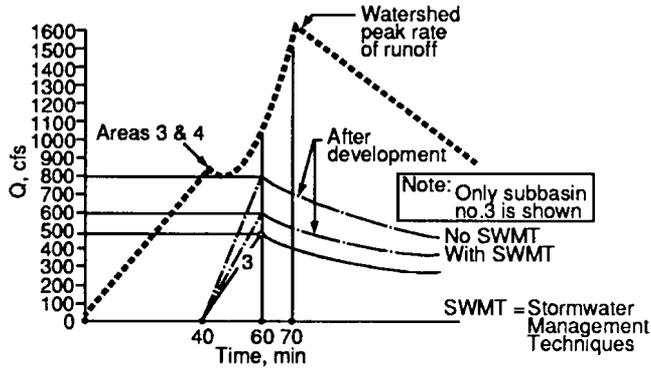


FIGURE 7.48 Subbasin hydrograph, after development (96).

peak outflow rate of the major watershed on a cubic foot per second per square mile (cubic meter per second per hectare) basis. The actual design criterion requires that the outflow rate from a detention practiced be determined using the time of concentration from the major watershed.

The Independence detention criterion will not allow any significant increases in downstream flooding but may also be overly conservative. This criterion assumes that the hydrograph peak from an individual subbasin arrives at the critical point at the same moment as the hydrograph peaks from all other subbasins. This assumption is obviously somewhat conservative and, as such, requires additional detention storage volume, but at the same time provides a higher level of flooding protection.

Rooftop Storage. Rooftop storage involves the use of small perforated weirs or similar structures placed around the inlets of roof downdrain pipes. These fixtures are designed to cause water to pond behind them and

be released at a reduced rate until a maximum design depth is reached. At this design depth, these weir structures overflow and the roof downdrains are allowed to function at their peak capacity. During most storms, no water will be stored because it will run off as quickly as it falls. Only during the larger storms will water accumulate. Care must be taken, however, in the design, construction, and maintenance of such facilities.

Parking Lot Storage. Parking lot storage may be designed to reduce peak rates of runoff to the predevelopment level by storing excess runoff and releasing it slowly. The storage area should have at least a 1% slope toward the outlet to assure complete drainage following a storm. The maximum depth of water within the ponded area should not exceed 7 in (17.8 cm).

Detention Ponds. Small ponds can be constructed to detain stormwater by building small earth dams across natural drainage courses or by regrading a site to provide storage capacity. These ponds may be located in areas where other site development is unsuitable or expensive. Ponds may also provide visual and recreational assets.

Detention ponds are most commonly constructed by impounding water behind an existing or proposed road fill, constructing an embankment to impound stormwater in a “dry pond,” or by adapting an existing or proposed “wet pond” to capture and slowly release stormwater. Wet ponds are impoundments that hold water during dry periods for recreation, fire protection, irrigation, and fish or wildlife use.

When road fills are used to detain stormwater, special precautions should be taken. These precautions normally include:

1. Check storage depth to ensure that stormwater does not encroach on roadway.
2. Design inlet to reduce potential for clogging.
3. Provide an emergency spillway.
4. To prevent hazard to life, check flows through emergency spillway to ensure that the quantity depth times velocity does not exceed $9 \text{ ft}^3/\text{s}$ ($0.84 \text{ m}^2/\text{s}$).

When dry ponds are constructed, the pond bottom should be graded to a minimum 1.5% slope to provide adequate drainage. Where soils are particularly impermeable, underdrains should be considered.

Figure 7.50 shows a typical underdrain layout. Criteria for the design of subsurface drainage can be found in Ref. 84. Sufficient sediment storage capacity should be provided to allow for accumulation of sediment without adversely affecting the performance of the basin. Procedures for calculating required sediment storage capacity are described under sedimentation practices. The banks of dry ponds should normally be graded to a maximum slope of 25% to facilitate rescue of small children. Where slopes are greater than this, fencing of the pond should be considered. Consideration should also be given to the construction of barricades to prevent entrance of motor vehicles.

Wet ponds can also be designed for stormwater detention. These structures are often designed with multiple uses in mind, such as recreation, aesthetics, and fire control. The pond should be designed with adequate inflow to allow for evaporation and seepage losses. From the waterline to 3 ft (1 m) below the water surface, the cross slope should be as high as practical to minimize the growth of aquatic plants. From this point outward, to a point where the water depth is 5 ft (1.5 m), the pond bottom should be graded as flat as possible to facilitate the rescue of small children. Where slopes around the perimeter of the pond are greater than 4:1, fencing should be considered to prevent the access of small children.

Ponds used in a stormwater detention are typically created by constructing a dam or embankment across a small water course. Primary considerations in the design of any “embankment” pond are: (1) seepage through the fill, (2) slope stability and erosion control, (3) integrity of any discharge structures, and (4) prevention of overtopping by large floods. References 12, 13, 84, and 98 should be consulted for guidance on the design of earthen embankments.

Regional Detention Basins. Regional detention basins modify the runoff characteristics of an upstream catchment by reducing the peak flow and releasing detained stormwater over a relatively long period. Re-

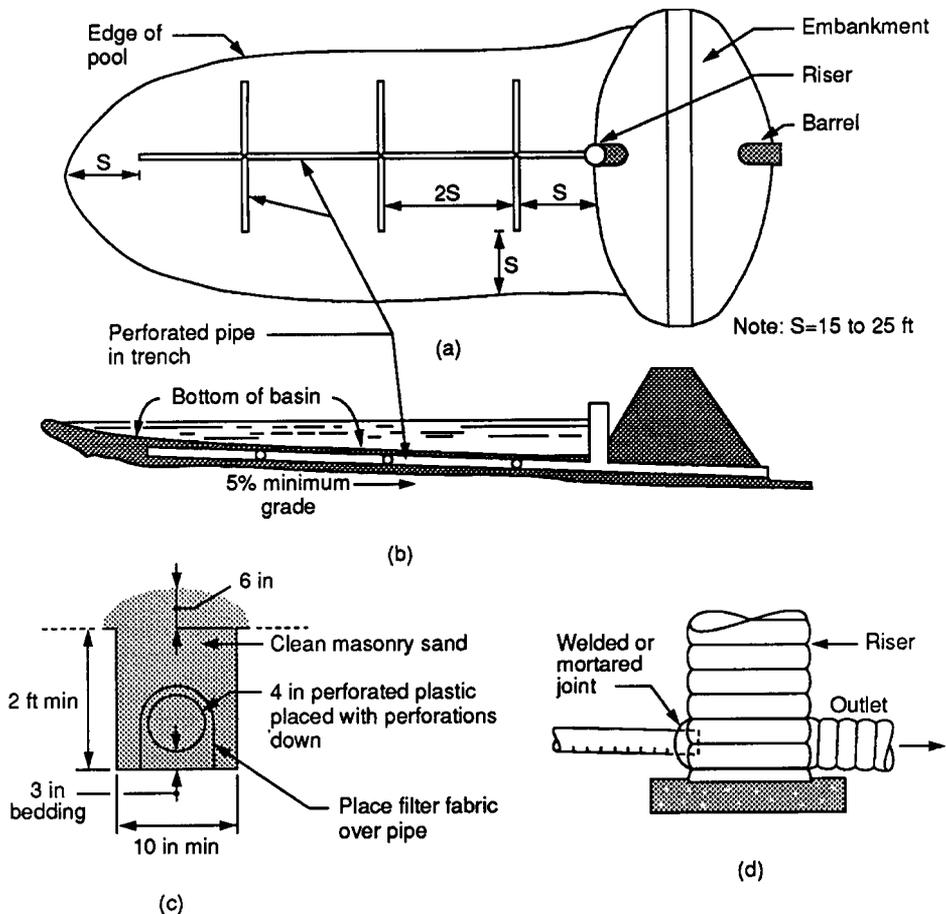


FIGURE 7.50 Dewatering sediment basin with subsurface drain. (a) Plan view; (b) profile; (c) cross section of drain pipe in trench; (d) riser connection (100).

gional detention basins differ from on-site detention in that they are frequently designed to control specific problems. Their larger size reduces the cost per unit volume of storage and these basins become attractive as permanent ponds or recreational areas.

The distinction between small upstream controls and main channel controls was made by Leopold and Maddock (99). This work documents that small detention facilities, or those associated with on-site systems, produce the greatest benefit immediately downstream from the outlet of the site, but have little impact on the flows in main channels which occur during large storm events. In contrast, large detention facilities or regional systems can have a substantial impact on main channel flows downstream, but do nothing for flood control in upstream areas. When the majority of flooding costs are associated with the major channels, large detention facilities will be more cost effective. These findings were originally documented in the context of major river basins, but the same concepts of hydrograph timing and economies of scale in construction also apply to smaller basins.

Sedimentation. Sedimentation may be used as a pretreatment for other permanent stormwater practices such as infiltration measures, as a temporary control to remove sediment from construction site runoff, or as a permanent practice to remove particulate-related pollutants. Particulates such as soil particles are removed since they are heavier than water and will settle out given sufficient time. The rate of settling is described by Eq. (7.55) for quiescent conditions.

$$V_s = \frac{gD^2}{18\mu}(S - 1) \quad (7.55)$$

where V = particle settling velocity, ft/s (cm/s)

g = acceleration due to gravity, 32.2 ft/s² (980 cm/s²)

S = specific gravity of solid, typically 62.2 lb/ft³ (2.65 g/cm³)

D = diameter of particle, in (cm)

μ = kinematic viscosity of water [0.01 g per centimeter per second at 20°C]

Examination of this equation reveals that the settling velocity is lowest for small particles and increases as the square of the particle diameter. Because of this, sedimentation ponds will be most effective in removing the coarser particles such as sand and least effective in removing clay-size particles.

The poor settleability of small particles does not preclude the use of sedimentation as an effective control in areas with a large fraction of clay in the parent soil. Field studies have shown that soils containing an appreciable clay fraction commonly erode as aggregates of primary particles. The median diameter of this clump may be sand size or larger.

Pollutants trapped in sedimentation ponds may be particles themselves or adsorbed onto particles. Many pollutants, most notably phosphorus, adsorb onto a particle in proportion to the particle surface area. Because smaller particles have a higher surface area per unit weight, smaller particles may contain a disproportionately high percentage of the total pollutant load. Because of the low settling velocity of small particles, this fraction of the pollutant load can be very difficult to remove.

Sedimentation Pond Design. Sedimentation ponds may be designed to achieve a certain trap efficiency or based on a simple rule-of-thumb storage requirement. Typical storage volumes required for surface mining are shown in Table 7.51.

When designing a pond to achieve a certain trap efficiency, the design event and the incoming sediment grain size distribution pond designed to trap 75% of the incoming sediment during the 2-year storm may only achieve a 50% removal during the 10-year storm. There is currently no straightforward technique for estimating the average annual trap efficiency. Various investigators have modeled sedimentation ponds over a long period of time and developed recommendations for improving the long-term trap efficiency. Some of these recommendations are described in the following section on detention pond modifications.

An incoming sediment grain size distribution is required for the design of any sedimentation practice. This distribution can be obtained from past studies, field measurements, or extrapolated from published soil surveys. Actual field data are by far the most desirable but may not always be feasible. When no other data are available than soil surveys and basin topography, the grain size distribution may be estimated by the following procedure.

1. Determine the average grain size distribution for soil within the pond watershed.
2. Estimate the sediment delivery ratio(s).
3. Assume that all soil particles coarser than a certain size d_s settle out in upstream areas where the percent finer subscript s is equal to the sediment delivery ratio in percent.
4. Estimate the grain size distribution of sediment actually delivered to the pond by dividing the fraction of soil within each size class less than d_s by the sediment delivery ratio as a fraction of s .

This method is illustrated in Table 7.52. Incoming grain size distributions obtained by this method can be used as a first cut but will usually be overly conservative since a significant portion of the eroded sediment may be present as clumps, not as the primary particle grain size distributions found in soil surveys.

Once the influent grain size distribution, required trap efficiency, and design storm are specified, the sedimentation pond may be designed. A design grain size should be selected so that the portion of incoming sediment larger than this grain size is equal to the required trap efficiency. The design settling velocity should then be calculated using Eq. (7.55). This settling velocity is commonly reduced by 25 to 90% to account for short circuiting and turbulence. Probable pond dimensions should be estimated including width, length, and mean depth. Average discharge during the design storm should also be estimated. Pond dimensions should then be adjusted so that

$$L = \frac{Q}{WV_s} \quad (7.56)$$

where L = pond length, ft (m)

Q = mean flow, ft³/s (m³/s)

W = mean width, ft (m)

V_s = design fall velocity, ft/s (m/s)

The pond length-to-width ratio should be greater than 2:1 in all cases. When the length-to-width ratio is less than 3.5:1, baffles should be considered to reduce short circuiting. A permanent pool with a minimum depth of 3 ft (1 m) should normally be included to provide sediment storage and reduce turbulence. When a permanent pool is not used, the pond should be drained by a perforated riser or siphon. Bottom drains typical in conventional detention ponds greatly reduce trap efficiency.

Trap Efficiency Regression Equations. Sedimentation pond trap efficiency may also be calculated using regression equations developed at the University of Kentucky (101, 102). These equations were developed from a database generated by a computer model developed to route different-size sediment particles through the detention basin for varying flow rates. The computer model was calibrated for a number of monitored storms and showed good agreement. Trap efficiency equations are given for various combinations of outlet structure, permanent pool, and base flow.

Peak inflow, peak outflow, and the weighted average detention time are obtained by routing a design storm through the pond. The average detention time is the difference in hours between the centroids of the inflow and outflow hydrographs.

TABLE 7.52 Estimating Influent Grain Size Distribution

Particle diameter, mm	Original soil: percentage finer than	Influent sediment: * percentage finer than
4	100	0
1	99	0
0.25	89	0
0.0625	65	65/0.65 = 100
0.160	42	42/0.65 = 65
0.004	36	36/0.65 = 55
0.001	14	14/0.65 = 22

*Sediment delivery ratio = 0.65.

When a minimum trap efficiency is required for a specified design storm and all lesser storms, perforated risers, drop inlet risers with siphons, or gravel filters as a bottom drain will normally be necessary. Ponds designed to discharge at the bed layer as in a typical detention pond will achieve the maximum trap efficiency at the design storm and a lower trap efficiency for lesser storms.

Drop Inlet with Permanent Pool

$$E = 89.2 + 25.4(S/Q) + 1.77(P_{20} - P_5)(t_d/t_{st}) - 1.23(q_{out}/q_{in})^{0.3}(P_{20}) \quad (7.57)$$

Drop Inlet with Permanent Pool and Base Flow

$$E = 93.1 + 27.6(S/Q) + 0.046(P_{20} - P_5)(t_d/t_{st} - 1.4P_{20} (q_{out}/q_{in})^{0.3} \quad (7.58)$$

Drop Inlet with Dry Basin at Beginning of Storm

$$E = 92.5 + 13.2(S/Q) + 1.9(P_{20} - P_5)(t_d/t_{st}) - 1.4(q_{out}/q_{in})^{0.3}P_{20} \quad (7.59)$$

Note: This equation assumes 100% withdrawal from the surface and should not be used unless the basin is designed with a drop inlet riser and a gravel drain for dewatering after the storm has past. When the peak outflow is controlled to less than 10% of the peak inflow, this equation will yield unreasonably high trap efficiencies.

where E = trap efficiency, %

S = storage capacity up to riser crest, acres/ft

Q = total inflow volume, ft³

t_d = volume weighted average detention time, h

t_{st} = storm duration, h

q_{out} = peak outflow rate, ft³/s

q_{in} = peak inflow rate, ft³/s

P_{20} = percent finer than 20 μm

P_5 = percent finer than 5 μm

Drop Inlet with Perforations Uniformly Distributed on Riser

$$E = 100.56 + 0.92Q - 0.89q_{out} - 0.31P_{20} - 0.58P_5 - 1.0D + 0.13 + out \quad (7.60)$$

100% Bottom Withdrawal

$$E = 100.28 - 1.12Q - 1.05q_{out} - 0.53P_{20} - 0.67P_5 - 1.05D + 0.14 + out \quad (7.61)$$

where E = trap efficiency, %

Q = volume inflow, acre/ft

D = depth at outlet, ft

q_{out} = peak outflow rate, ft³/s

t_{out} = time for 95% of possible withdrawal, hr

Detention Pond Modifications. Conventional detention ponds are designed to control large storms with the minimum storage volume. This results in a structure that has little impact on small storms, which can produce a significant portion of the sediment and pollutant load. Many municipalities are now considering the use of best management practices (BMP) to manage water quality as well as to control peak rates of runoff.

The most common method of modifying detention basins to improve pollutant trap efficiency is to use as

two-stage outlet structure. The first stage is constructed to capture the most common storms and hold them for an extended period so that the finer particles can settle out. The second stage is sized as a conventional detention pond. The increase in storage required for sediment trapping typically results in only a minor increase in the total cost of the basin.

New Jersey recommends that surface runoff generated by a 1-year 24-h storm be captured and detained for 40 h to remove sediment (103). Fairfax County, Virginia, requires nonpoint source (NPS) management storage be provided in all detention ponds draining into their drinking water reservoir (104). The required storage for Fairfax County is listed in Table 7.53. The first-stage outlet is also designed to drain the pollutant trapping pool in approximately 40 h.

The addition of NPS management storage to conventional detention basins can greatly increase the trap efficiency for pollutants associated with fine particles. Modeling studies by the Northern Virginia Planning District Commission indicate that the particulate phosphorus trap efficiency can be increased from 20% for conventional peak shaving basins to 90% when NPS management storage is provided (88). Estimated annual trap efficiencies for a variety of pollutants are shown in Table 7.54.

The NPS management storage may be drained using either a perforated riser or subsurface drains. Subsurface drains will probably increase the pollutant removal by removing very fine particles and possibly a portion of the soluble fraction.

Permanent Ponds and Wetlands. Permanent pools have commonly been constructed in conjunction with detention storage for improved recreation and aesthetics. Natural wetlands have also been used for detention storage since these areas cannot be conveniently used for any other development activity. In recent years, many investigators have noted substantial soluble as well as particular pollutant removal in wetlands as well as permanent ponds. At the present time, the processes leading to pollutant removal in wetlands and ponds are poorly understood. Field studies conducted as part of the Nationwide Urban Runoff Program (NURP) greatly improved the level of understanding in these two areas.

Permanent Ponds. Permanent ponds have been observed to be effective in removing phosphorus and suspended solids and in oxidizing organics. Substantial removal of nitrogen is not common since organic nitrogen is effectively converted to soluble ammonia nitrogen, nitrate, and nitrite. Removal of phosphorus is frequently in excess of that expected for plain sedimentation and has been attributed to uptake by algae and settling. Permanent ponds are substantially more effective in removing phosphorus during the summer growing season than during the winter.

Monitoring of a small recreational lake in Missouri from April to October indicated substantial pollutant removal (105). Reductions in average pollutant concentration and total load are listed in Table 7.54. In another study of permanent ponds in northern Virginia, two permanent ponds and one detention pond were monitored (106). The detention pond was modified to provide an NPS management storage pool for improved trap efficiency. Results of this study are shown in Table 7.55. In both the Missouri and northern Virginia studies, permanent ponds were very effective in removing phosphorus but less effective at nitrogen removal.

Wetlands. Wetlands are known to remove substantial quantities of pollutants including suspended solids, BOD, pathogens, hydrocarbons, and heavy metals. Wetlands may act as both sources and sinks of nitrogen and phosphorus. These nutrients may be absorbed into plant tissue during the growing season but released in the fall when plants die off. Substantial denitrification has also been observed in many wetlands.

Removal mechanisms occurring in wetlands may be physical, chemical, and/or vegetative. The most common are sedimentation, coagulation, adsorption, filtration, and biological uptake. When managing a wetland to improve pollutant removal, the hydrology of the area is very important. Critical factors to consider are

- Velocity and flow rate
- Water depth and fluctuation

TABLE 7.53 Annual Pollutant Removal Rates for On-Line Detention Control BMPs (88)

Land use	NPS mgt. storage, in	NPS release rate at brim full condition, in/day	Annual removal, %									
			sediment	inorganic P	total P	TKN	total N	BOD	lead	zinc		
Large-lot single family (12% impervious)	0.10	0.10	88	0	33	25	21	31	82	44		
Medium-density single family	0.21	0.21	89	0	40	42	35	39	83	50		
Townhouse/garden apartment (25% impervious)	0.33	0.33	88	0	40	46	36	47	82	48		
High-rise residential (70% impervious)	0.54	0.54	93	0	43	31	23	46	87	52		
Industrial (70% impervious)	0.54	0.54	93	0	43	31	23	46	87	52		
Shopping center (90% impervious)	0.78	0.78	94	0	47	32	24	47	88	53		

Note: 1 in = 2.54 cm.

TABLE 7.54 Pollutant Removal in Frisco Lake (105)

Parameter	Percent reduction	
	Average concentration	Total load
Chemical oxygen demand	52	54
Total phosphorus	65	65
Total suspended solids	89	88
Organic nitrogen	31	22
Ammonia nitrogen	-13	-30
Watershed: 111 acres (44.9 ha) mixed residential		
Lake: Surface area: 5.7 acres (2.3 ha)		
Lake: Detention time: 28 days		
Lake mean depth: 3.3 ft (1.0 m)		

- Detention time
- Circulation and distribution patterns
- Soil permeability and groundwater movement
- Turbulence and wave action

These factors should be manipulated with care since altering any one may shift the makeup of the community. A typical example might be to raise the water level in a wetland to increase detention time and sedimentation. An unforeseen effect of this could be to cause the soil to become anaerobic, releasing nutrients to the water column.

Field data on pollutant removal in wetlands is extremely variable. Some wetlands are known to be nitro-

TABLE 7.55 Projected Annual Pond Performance (106)

Parameter	Burke pond, permanent pond			Westleigh pond, permanent pond			Stedwick, temporary detention		
	Inflow, kg/ha/yr	Outflow, kg/ha/yr	Removal, %	Inflow, kg/ha/yr	Outflow, kg/ha/yr	Removal, %	Inflow, kg/ha/yr	Outflow, kg/ha/yr	Removal, %
Total suspended solids	51.3	32.4	36.8	72.0	9.5	86.8	99.0	22.5	77.3
Total Kjeldahl nitrogen	5.58	3.51	37.1	2.16	1.17	45.8	3.15	4.32	-37.1
Soluble Kjeldahl nitrogen	3.96	2.38	39.8	1.48	0.76	48.6	2.38	3.15	-32.1
Nitrite and nitrate nitrogen	2.38	0.40	83.6	0.81	0.23	71.1	1.17	1.98	-69.2
Total phosphorus	0.79	0.32	59.2	0.40	0.12	69.8	0.79	0.58	26.2
Total soluble phosphorus	0.40	0.17	56.2	0.23	0.12	50.9	0.62	0.45	27.4
Outlet drainage area	11.0 ha			19.4 ha			13.9 ha		
Storage volume	33,000 m ³			36,000 m ³			3520 m ³		
Mean depth	2.6 m			2.3 m			1.7 m		

Note: 1 lb/ac/yr = 1.12 kg/ha/yr.

gen sources while others remove up to 1150 lb/acre (1290 kg/ha) of nitrogen. Maximum phosphorus removal potentials range from 25 to 70 lb/acre (30 to 80 kg/ha) of phosphorus. Wetland vegetation has also been shown to remove substantial quantities of cadmium, copper, iron, lead, and zinc. References 107 and 108 provide good summaries of knowledge concerning pollutant removal in wetlands.

LEGAL, INSTITUTIONAL, AND FINANCIAL IMPLICATIONS

Legal problems in stormwater management result from action taken by a land developer, builder, public agency, or individual. Complaints and litigation focus on changes in natural drainage flows, flood stages, topography, vegetative cover, and environmental quality. Institutional aspects include financing and management of drainage programs.

Legal Concepts

Diffused surface water is runoff from precipitation prior to entry into a natural watercourse or lake. The overriding legal concern of diffused surface water is the right to dispose of it, or to otherwise protect property from its adverse effects. The right to use diffused surface water, which is the primary concern of natural watercourse law, is considered to belong to the owner of the property where the water is found. Three major doctrines governing the use and disposal of stormwater runoff have evolved: the *common enemy doctrine*, the *civil law doctrine*, and the *reasonable use doctrine*.

Common Enemy Doctrine. The common enemy doctrine, which views runoff as an undesirable element, grew from the nineteenth century desire to promote the development of land. Runoff water is considered to be a common enemy to all landowners, and each may defend against it to the best of his or her ability without liability for injury which the landowner's defensive actions impose on other parties. Within this legal framework, a landowner is free to both obstruct flow from higher ground and to improve the natural drainage of his or her own property, thereby increasing the burden on lower property owners.

The common enemy doctrine has been modified in many states to prohibit the concentration of runoff and discharge en masse onto lower property. The right of a landowner to make drainage modifications without liability to lower property owners is now generally restricted to situations where modifications are necessary to the development of the property and are carried out with proper care. With regard to the freedom to obstruct flow from higher land, culverts and other controls must be included where feasible to prevent flooding of the upper property, especially where the water concentrates in a surface depression.

Few courts follow the doctrine strictly, and it has been tempered over the years to include the requirement of reasonableness in one's actions to protect oneself from the common enemy. Willful, wanton, and malicious conduct concerning the disposition of surface water can now result in liability of the landowner. Nevertheless, instances of increase in runoff and acceleration of flow due to property development continues to persist (109).

Civil Law Doctrine. The civil law or natural flow doctrine assumes a natural easement or servitude over lower lands within a drainage basin. Nothing can be done to change the flow of runoff in any way that is harmful to other property or that is unnatural to normal flow. The landowner is liable for any injury resulting from interference with the natural drainage process. A landowner must accept runoff from higher ground and must not obstruct its flow to the detriment of the upstream (or upslope) landowner. A landowner may not lawfully injure a downstream landowner by increasing, concentrating, or accelerating the flow leaving his or her property. Strict adherence to this doctrine does not require a lower landowner to accommodate development-induced changes in flow, only natural flows.

In practice, however, some degree of necessary drainage modification without liability is allowed; and owners of lower ground may not modify existing drainage patterns as long as such modifications are reasonable. Upper landowners may increase runoff above natural flows as long as the increase is not ruled to be unreasonable (110).

Reasonable Use Doctrine. The reasonable use doctrine is a compromise theory and is concerned with the rights of both upstream and downstream landowners. This doctrine holds that some modification of runoff is a necessary and proper consequence of property development but that allowable modifications must be limited by the impact on others. Three questions that recognize the similarity and equality of the rights of each landowner are basic to an analysis under this doctrine.

1. Was there a reasonable necessity for the landowner to alter the drainage in order to make use of the land?
2. Was the alteration done in a reasonable manner with all due care to prevent injury to another's land?
3. Does the utility of the landowner's conduct reasonably outweigh the gravity of harm to others?

Factors to be considered for analysis of damage-causing activities include (110)

1. The social value of the activity
2. The suitability of the location of the activity
3. The difficulty in preventing damage
4. The impact on the activity if compensation is required for the resulting injury

Factors to be considered for the gravity of the resulting harm are (110)

1. The extent and character of the injury
2. The social value of the injured activity
3. The suitability of the location in question
4. The difficulty for the injured party to avoid harm

The reasonable use doctrine requires a case by case balancing of the utility of the drainage activity versus the resulting harm.

Federal Role in Stormwater Management

Federal law generally has a greater role with respect to water quality protection than with respect to the disposal and allocation of surface runoff. Concern for protecting the quality of runoff water was late in developing because the water does not exist in permanent form. However, national water policy in recent years has focused on the utilization of stormwater as a resource for urban design, recreation, alternative water supply, and other purposes in contrast to earlier attention to disposal. As first addressed in the Federal Water Pollution Control Act Amendments of 1972 and continued in the Clean Water Act, concern with the quality of diffused surface water is termed nonpoint pollution control. In the 1987 amendments to the Clean Air Act, Congress mandated that EPA develop a permit system for certain point sources of stormwater discharge.

In 1987, the U.S. Congress amended the Clean Water Act to require implementation of a comprehensive approach for addressing stormwater discharges under the National Pollutant Discharge Elimination System (NPDES) program. General permits are used in lieu of individual permits and are most focused when issued on a watershed basis. Also, general permits may be used instead of individual permits to reduce the administrative burden on permitting authorities. However, individual permits may be needed in some cases to address specific concerns, including permit noncompliance.

Stormwater discharges can produce a number of environmental effects as a result of land development, illicit discharges, construction site runoff, and improper disposal of materials. Problems also can occur from agricultural stormwater discharges and return flows from irrigated agriculture; however, this source is exempted from regulation by the Clean Water Act. Other sources may be of concern in certain areas and can be addressed on a category-by-category or case-by-case basis.

The NPDES Phase I stormwater program resulted in significant improvement of surface water quality by reducing polluted runoff from a large number of priority sources, including major industrial facilities, large and medium city storm sewers (municipal separate storm sewer systems or MS4s), as well as construction sites that disturb five or more acres. The Phase II stormwater regulations expanded the program to smaller municipalities and construction sites that disturb one to five acres.

Initial efforts to improve water quality under the NPDES program primarily focused on reducing pollutants in industrial process wastewater and municipal sewage. This focus developed because many sources of industrial process wastewater and municipal sewage were not adequately controlled and represented immediate and pressing environmental problems. Furthermore, these discharges were easily identified as responsible for poor, often drastically degraded, water quality conditions.

Stormwater discharges associated with industrial activity must meet technology-based requirements and any more stringent requirements necessary to meet water quality standards. NPDES permits for discharges from municipal separate storm sewer systems may be issued on a system- or jurisdiction-wide basis, must include a requirement to effectively prohibit illicit (nonstormwater) discharges into the storm sewers, and must require controls to reduce pollutant discharges to the maximum extent practicable, including best management practices. As with all point source discharges, stormwater discharges are subject to more stringent limitations when necessary to meet applicable water quality based standards.

The major role of the federal government in the regulation of water quantity is in floodplain management in the form of the National Flood Insurance Program. As with nonpoint source control, participation by local government is through state-enabling legislation.

State Role in Stormwater Management

The basic power to regulate land uses within a state, exclusive of federal lands, resides with the state. The basic constraint on water law modification consists of constitutionally protected property rights. The courts have generally upheld the right of the states to change the water law provided that adequate protection is given to vested rights. Thus, a municipality or other local unit must derive the power to regulate from a state legislative or constitutional provision.

In the past, state legislatures have regarded stormwater management as a relatively low priority item with only a few states adopting pertinent, enabling legislation. Floodplain management, with its ties to land use zoning and regulation, is an area where the state government has served as an intermediary and facilitator. The principal activity of the state has been the enactment of enabling legislation and facilitation of local government participation in the National Flood Insurance Program. Traditionally, the states have taken the strongest leadership in the areas of stormwater pollution control, erosion, and sedimentation (*111*).

Formation of local benefit districts is authorized by state legislation and variable interpretations have resulted. In Colorado, the state legislature passed a statute defining benefits from drainage programs, making it possible to assess benefits to property that does not directly suffer flood damage. This allows the costs of public flood control and drainage work to be assessed to upperlying lands, expediting the financing of stormwater improvement projects and making special assessment financing more practical. In contrast, the General Assembly of South Carolina undertook legislation to adopt a "home rule" law allowing municipalities to establish benefit districts only against property receiving direct benefit in the form of reduced damage (*110*).

State-enabling legislation includes eminent domain; police power; taxation by ad valorem; issuance of general obligation bonds, permits, and licenses; and the maintenance of navigable streams.

Legislation requiring local governments to adopt a comprehensive ordinance establishing procedures governing land disturbance activities has been passed by some states. Others provide for a comprehensive state program including state soil erosion and sedimentation control regulation to be administered and enforced under a state commission. Local governments may administer and enforce their own local ordinances if those ordinances meet minimum state standards. Others simply permit local governments to enforce drainage ordinances.

The major role of the states in stormwater management has been to provide for the involvement of local agencies in stormwater management through enabling legislation. Since most of the specific development legislation is done at the municipal and county levels of government, drainage programs with the highest chance of success are those that are administered and enforced by these units of government.

Municipal Role in Stormwater Management

Drainage and detention are traditionally a function of local government. Public agencies have identified the lack of areawide criteria, laws, and guidelines for development of public stormwater management facilities as the most pressing problem facing municipalities. Land development in a single watershed that covers several jurisdictions, each having separate drainage regulations, is the source of administrative, physical, and legal problems. The adoption of county- or watershed-wide stormwater control ordinances with the use of uniform criteria and standards is helping to alleviate this problem (*112*).

County involvement in stormwater management ranges from Denver's Urban Drainage and Flood Control District, with virtual police power to regulate floodplains in six counties in both incorporated or nonincorporated areas, to the Metropolitan Sanitary District of Chicago, with jurisdiction in a single county and no power to regulate floodplains in unincorporated areas. The principal responsibility and authority for drainage facilities lies with the local government having jurisdiction, usually a municipality (*111*).

Legal controls available to municipalities include

- Erosion and sedimentation ordinances
- Floodplain ordinances
- Stormwater drainage ordinances
- Zoning ordinances
- Subdivision regulations
- Building codes
- Grading ordinances

In general, municipalities are not required by the state to provide drainage and are not liable for the failure to do so, nor are they liable for negligence in the initial planning of stormwater disposal, or for damages resulting from the failure to provide drainage. Municipalities are liable for negligence in the maintenance of existing storm sewers and for the continued maintenance of a system which constitutes a private nuisance. A private nuisance results where a city takes action that interferes with natural drainage and liability results for damage to private property as a result of this action.

Many municipalities now require developers to consider future development of the watershed when designing stormwater drainage systems for a new development. Detention storage by developers is frequently required and is implemented through subdivision laws, zoning ordinances, building codes, and water pollution regulations. Requirements for maintenance of these facilities vary (*112*). Because they are constructed for the public good, some attorneys and municipalities feel that the maintenance of detention facilities should be public responsibility. Other municipalities require the private owners to assume maintenance responsibility. The requirements for maintenance of stormwater detention facilities constructed on rooftops and parking lots are not well documented. While public maintenance for reason of protecting the public

good is advocated by some, others equate the situation with the requirement of landowners to shovel snow on public rights of way for reasons of protecting the public good. While many municipalities have adopted ordinances and regulations for stormwater detention, many are not effective. Often maintenance responsibility and liability for failure to maintain facilities is not clearly defined.

Financing of Stormwater Management Programs

Financial issues in stormwater management include the construction, operation, and maintenance of physical facilities. Legal problems occur in stormwater management programs when cost-benefit calculations are challenged. Financing methods and the apportionment of costs vary considerably and are discussed in this section.

Authority to Establish Drainage Control Programs. Modern drainage-related cost recovery programs originated in the early storm drainage and reclamation districts. These programs are authorized by state legislatures concerning police power and the power to tax. Police power authority is a right vested with a sovereign to require owners of property to use their property only to the extent that such use does not preclude a neighbor's reasonable use of his or her land. Regulation, management, and control of private property to promote the general health, safety, and welfare of the community are allowed. The power to tax authority is a right vested with a sovereign to make exactions from the general public to pay for services in the interest of the general health, safety, and welfare of the community. Municipal drainage ordinances regulate and tax private property and have required the expansion of police power authority. This has been upheld in court as a necessity to regulate increasing development and conversion of land from its natural state (113).

The necessary requirements of cost recovery programs are not specified in police power or power to tax authorities, and litigation questioning the equitability of some programs has resulted. Pertinent points of contention concerning a specific cost recovery program include

1. The calculated benefits from a drainage project exceed the calculated costs
2. The benefits from a drainage project accrue to the property being assessed
3. The assessment costs have been properly and fairly apportioned among the landowners in the assessment area
4. The benefits from and the costs of the project have been properly calculated

In order to withstand judicial review, each of these questions requires criteria to be developed based on the following considerations:

1. *Benefits greater than costs.* The legislatively implied definition of benefits has resulted in a broad range of interpretations by the courts. A standard interpretation of the term is needed in order to ensure consistent identification of special and general benefit, and the relative weights of each, that can be included in a computation of project benefits.
2. *Benefits accruing to assessed area:* The courts have held that a property cannot be taxed for benefits that do not exist, and any attempt by taxing authorities to impose a tax burden without a compensating advantage is a power arbitrarily exerted. This amounts to confiscation, and violates the 14th Amendment.
3. *Proper apportionment of costs:* Responsibility for defining the boundaries of the drainage district and specifying the benefit to each landowner within an assessment of cost recovery program lies with the legislature and is not normally subject to judicial review. Formation of a district is a legislative act carried out in the exercise of the police or taxing power of the state.
4. *Proper computation of benefits and costs:* Virtually no action has been taken to question the technique used to calculate the hydrologic information necessary to compute project benefits and costs. All

drainage programs with cost recovery provisions must be based on a master drainage plan which requires high initial costs. The major question is whether or not a municipality can collect drainage fees based on a simplified drainage plan. This is important for newly urbanizing areas that do not have funds to support the high cost of a master drainage plan with detailed computation of costs and benefits, yet wish to assess fees for drainage programs on a less specific master drainage plan. Without any sort of drainage plan, the courts have a tendency to invalidate a fee collection program, but the question of the reasonableness of a simplified plan has not been addressed (110).

Cost Apportionment Methods. Municipalities vary widely in the method used to apportion costs of stormwater management programs among landowners. Frequently used methods include assessed value of the property, front footage, land area fee, and land use and contribution to runoff. Other methods for financing and maintaining the program include allocations by special dedication of a portion of water and sewer user charges or ad valorem taxes, development-impact fees, fee-in-lieu of detention, special stormwater assessment areas, and state or federal grants.

Assessed Value. The costs of drainage improvements are apportioned based upon the assessed value of the property. Because this is the basis for general property taxation, a system already exists that is readily adopted to stormwater, and it is easy to apply to stormwater management programs. Payments made by property owners are state and federal tax deductible. The method relates cost with presumed ability to pay because property owners with high assessed value pay more than those with low assessed value. The method does not relate benefit of the improvements to the amount of payment, nor does it relate payment to the contribution to the problem.

Front Footage. In this method, property adjacent to a stream is assessed according to the amount of linear footage along the stream. This relatively simple to administer method tries to relate benefits to whomever pays. However, the costs are apportioned to relatively few properties. This burden on an individual property can be high because other properties in the drainage basin that are located away from the stream are excluded from payment.

Land Area Fee. Costs are apportioned on the basis of the land area of each property. Equal benefit throughout the basin is assumed. The calculation is made by dividing the total cost of the proposed basin improvement by the total land area within the basin. Information on lot size is readily available in tax records, making the method fairly easy to administer. Payment is somewhat related to the degree to which a property contributes to a drainage problem. However, lot size alone is not an accurate measure of the contribution to runoff because a large vacant lot will have considerably less runoff than a smaller lot which is mostly paved. Also, this method does nothing to promote effective on-site runoff controls (95).

Land Use and Contribution to Runoff. This method is computed in the same way as the land area fee except that land use is considered, in recognition that different variations in hydrologic response are caused by different land uses. In this method, a shopping center parking lot with nearly 100% runoff is charged more than the same area of farmland. The method can be used to discourage excessive pavement and encourage the use of on-site infiltration. Graduated fees are instituted based on general estimates of runoff associated with typical land uses. In order to arrive at the total impervious area in a basin, the areas of each land use are measured. The annual costs of drainage improvements are then divided by the total area to calculate the cost per unit area. This cost is usually termed an *equivalent residential unit* and is the amount of runoff from a typical single-family house and lot.

Allocating Costs between Present and Future Residents. It is impractical to make incremental drainage improvements as each new home or building is added in the basin. When a culvert is constructed, the pipe must be oversized to carry the future runoff from a completely developed basin. A large part of culvert cost is attributed to construction. In effect, present basin homeowners are paying the high initial cost so that future owners do not have to fund the work to replace an undersized culvert with a larger one. The three basic

approaches to allocating costs between present and future owners are: (1) delay in making improvements, (2) bonding, and (3) system development charges.

Delay in Making Improvements. If a drainage basin is only partially developed and existing runoff is not creating significant problems, improvements can be delayed until the basin is more developed and runoff problems occur. This approach does not work well unless projected future runoff from new development can be handled by the existing system with only minor inconvenience.

Bonding. This method allows drainage improvements to be made at the present time with the costs spread out over time by raising the capital needed through the sale of bonds. As the basin population increases, new residents share in paying off the bond, and individual payments decrease. This approach assumes that growth will occur, and new residents will share in retiring the bonds. Present residents will have to continue bond payments at the initial rate if this does not occur.

System Development Charge. With this method, the governing agency assesses a one-time-only fee at the time of development for “buying into” the stormwater drainage system. Similar assessments have been customarily used for sewer, water, and other utilities. Revenue from a system development charge may be accumulated in a fund for capital expenses or be used to pay off bonds.

Stormwater Drainage Utilities. A drainage utility is a financial structure developed to provide financing for stormwater management activities, such as the installation, maintenance, replacement, and improvement of stormwater management facilities.

Funds from the utility can also be used to retire bonds. Most drainage utilities consist of a new financial structure rather than a new technical or organizational structure, and are operated by the Public Works or Wastewater Management Division. These departments are already staffed and managed by professionals with a background in civil or environmental engineering and have existing means to regularly invoice homeowners and industry.

The two most popular means of apportioning costs are area fees and land use fees, as discussed in the previous section. The average 1983 user fee for a quarter-acre lot ranged from \$1.00 to \$3.00 per month. In addition to user fees, many drainage utilities charge a one-time connection fee for all landowners. This is charged initially to all existing developments and to new developments when they are built. Rather than being a structural connection to a storm sewer, the connection is to the whole stormwater management system including streams, culverts, bridges, and regional detention basins. The major functions of a storm drainage utility are

1. Review and evaluation of development plans to ensure compliance with a master drainage plan
2. Performance of detailed watershed studies
3. Design of regional detention basins, channel improvements, and other stormwater control facilities
4. Land acquisition for installment of regional drainage facilities
5. Installation, operation, and maintenance of regional detention basins
6. Installation, maintenance, replacement, and improvement of channel structures, bridges, and culverts
7. Stream channel improvement
8. Retirement of principal and interest of revenue or general obligation bonds
9. Administration of stormwater management activities and utility operations

Stormwater Drainage Ordinances

Many municipalities have adopted stormwater management ordinances to establish a uniform basis for municipal involvement. The power to do so is derived from police power and the power to tax. Ordinances in current use range from those dealing only with new development, to those permitting drainage control and management over all phases of development from undivided land to established development.

New Development. Requirements vary considerably within this category. Some ordinances require satisfactory drainage within the new development only, with no mention of where drainage waters collected within the development should be discharged. More specific ordinances require the developer to ensure that all the storm runoff waters and those draining into his or her property be conveyed to a designated outfall with costs born by the developer. While some ordinances confined to on-site drainage attempt to prevent drastic alteration in hydrologic response, more advanced ordinances provide for off-site collection drainage utility fees.

Existing Development. Three forms of cost assessment for drainage facilities are in common use for existing development: bond issues, assessment districts, and utility fees. Bond issues require a voter referendum; they have not been very successful in recent years as a result of the trend away from higher taxes. Assessment districts also have not been very successful since they impose an extra tax and require tremendous support to institutionalize and implement. In addition, this approach tends to be piecemeal without a basinwide plan. The newer approach of the utility fee provides citywide storm drainage services. A monthly fee, collected based on land area or land use, affords a comprehensive citywide approach to drainage, but relies heavily on general rather than special benefits created by control facilities.

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CHAPTER 8

SOLID WASTE

Eugene A. Glysson, Ph.D., P.E.*

Solid waste management continues to progress from conventional collection techniques and disposal methods to an integrated approach focused on source reduction and recycling. As land becomes more limited and regulations increase, the environmental engineer also directs attention to development and application of advanced disposal technologies.

Solid wastes are those materials, other than liquids or gases, that are deemed by their owner to no longer possess value and are discarded. They are generated by almost every activity, and the amount varies by source, season, geography, and time.

Historically, solid waste disposal consisted of open dumping but now is carried out in double-lined landfills with collection of and controls for gases and/or leachate. Other disposal means include composting and various incineration processes, which also may be used for codisposal of wastewater treatment sludges. These disposal means typically require controls for created pollutants, such as leachate and odor from compost operations and chemical and particulate emission from incinerator combustion.

Recovery and reuse are practiced widely. Source or central facility separation is used for a variety of products including paper, glass, plastics, ferrous metals, and nonferrous metals. Also, refuse-derived fuels may be used for energy production, and yard wastes may be composted to produce a humus soil conditioner.

SOLID WASTE—SOURCE AND EFFECT

The individual or organization discarding solid waste becomes the waste generator. The concept of waste having no value is defined by the generator, since the waste may represent some value to others through recycling or reclamation.

The amount of solid waste generated varies by season, geography, and time. The amount of solid waste generated from various sources under average conditions is discussed in this section. Waste characteristics are discussed in another section.

Source

Solid waste generation can be subdivided into residential and nonresidential, depending on its source. Residential wastes are generally considered to be household-type wastes, whereas nonresidential includes commercial, light industrial, and other wastes.

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Residential Waste Generation. Wastes generated by residential households are usually calculated in pounds (kilograms) per capita per day. This measurement is suitable for gross estimates for use in sizing disposal facilities and resource recovery operations, but is not appropriate for the design of collection systems (1).

Collection systems are more appropriately designed utilizing the annual average weight (pounds) per household (or stop) per week (PPHW). Households are an easily observed unit along a collection route. The U.S. Environmental Protection Agency (2) reports a range of from 46.2 to 71.0 lb per household per week from nine cities with curbside pickup in the 1970s. The average was 57.3 lb per household per week, which might be considered a typical value. In 1981 two studies (1) showed generation rates to be between 48.7 to 52.0 lb per household per week. One state department of natural resources recommends 52.0 lb per household per week for residential refuse collection (3). One pound per household per week (PPHW) equals 0.454 kg per household per week.

Estimates for household waste generation should be based on actual measurement. This means actually counting the residences on residential routes and weighing the refuse collected. Care must be taken to include the entire spectrum of residential premises and the various seasons of the year. Attempts have been made to correlate residential solid waste generation to several measurable factors including population served, households served, value of property, size of living area, and household income. Statistical analysis has led to the conclusion that population served is the most significant variable.

Measurement of residential refuse picked up at the curbside indicates that weekly refuse generation can be expressed by the following equation:

$$G = a + bP$$

where G = generation of household refuse in mass per week per household

a, b = constants determined by waste measurement survey

P = average persons per household in sample area (see Figure 8.1.)

Typical values of a and b as derived by field measurement are as follows:

Field Study I $a = 45.0, b = 3.3$

Field Study II $a = 44.4, b = 2.8$

Residential waste generation is not uniform throughout the year. An EPA report (2) shows average weekly rates per household for each month in 11 cities scattered throughout the United States. These data have been reduced to a monthly multiplier (Table 8.1) for use with the annual average weekly generation rate. Table 8.1 gives monthly multiplier and the maximum and minimum multipliers based on the data from these 11 cities. These data are also shown graphically in Figure 8.2.

Nonresidential Waste Generation. Refuse generation from various other sources has been evaluated by various agencies. These waste-generation rates are shown in Table 8.2.

Effect

Solid waste has a prevailing characteristic that sets it aside from the liquid and gaseous wastes produced by society. The characteristic is that it remains highly visible in the environment in which we live. Liquid wastes are quickly relegated to a sewer and are out of sight, and gases disperse into the atmosphere. Solid wastes, however, are stored and transported in and through societies' living space and have great potential for adversely affecting the quality of the environment.

The environmental effect of solid waste management begins with on-site storage. This aspect of management has a profound impact on the local environment, since improperly stored refuse may attract insects and

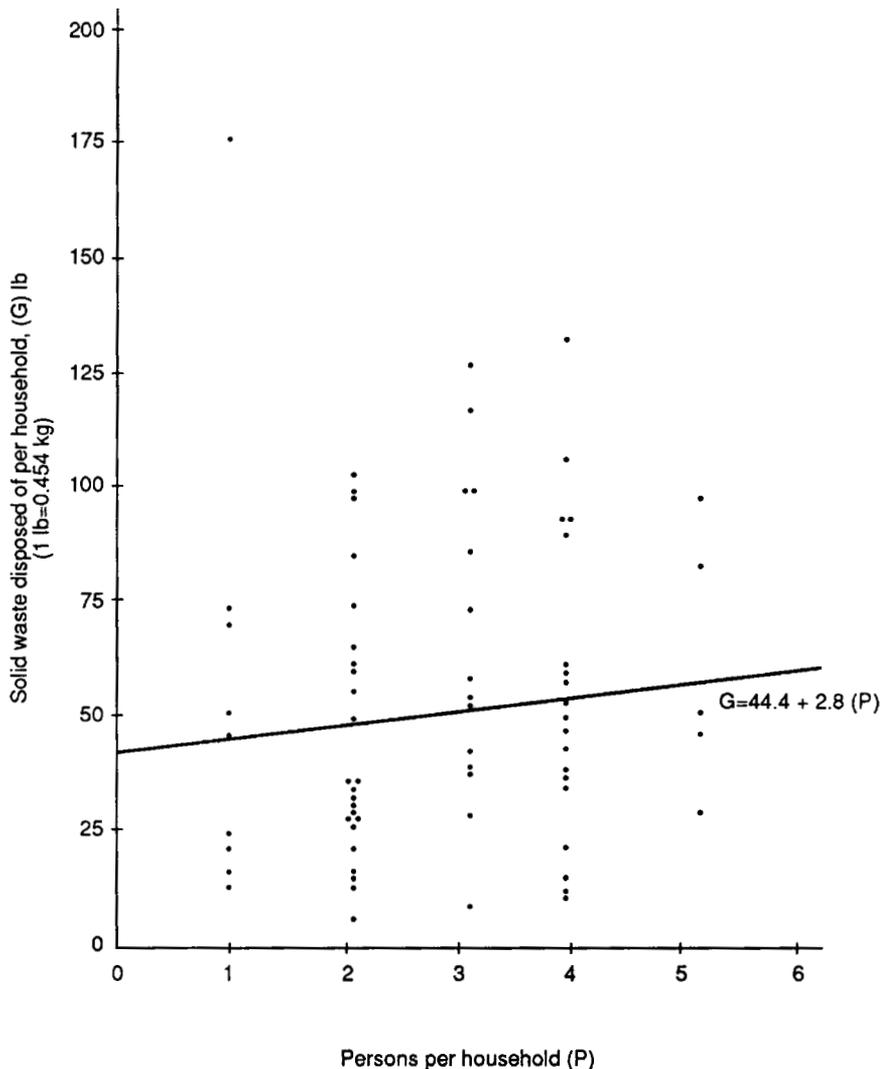


FIGURE 8.1 Household refuse collection (1).

rodents, present a fire hazard, be an unattractive nuisance, and produce odors, litter, and other unsightly conditions. On-site storage involves proper containerization in order to minimize these possible adverse effects. Various containers are available: galvanized steel and cans, plastic cans, plastic and paper bags for residential use, and steel bulk containers for commercial and other wastes. For residential use, single-use plastic and paper bags have, in general, been shown to be most suitable from all aspects, while plastic cans with good covers are next. Metal cans have the least capability to cope with all the conditions of proper on-site residential refuse storage. The frequency of refuse collection should include consideration for reducing or eliminating as many of the adverse effects of on-site storage as possible.

SOLID WASTE

TABLE 8.1 Multiplier for Annual Average Solid Waste Generation (2)

Month	Average	Maximum	Minimum
January	0.876	0.983	0.786
February	0.871	1.028	0.726
March	0.972	1.123	0.872
April	1.050	1.162	0.956
May	1.125	1.256	0.986
June	1.107	1.268	0.979
July	1.085	1.163	0.991
August	1.024	1.195	0.931
September	1.010	1.083	0.922
October	0.994	1.105	0.890
November	0.985	1.049	0.886
December	0.901	1.098	0.769

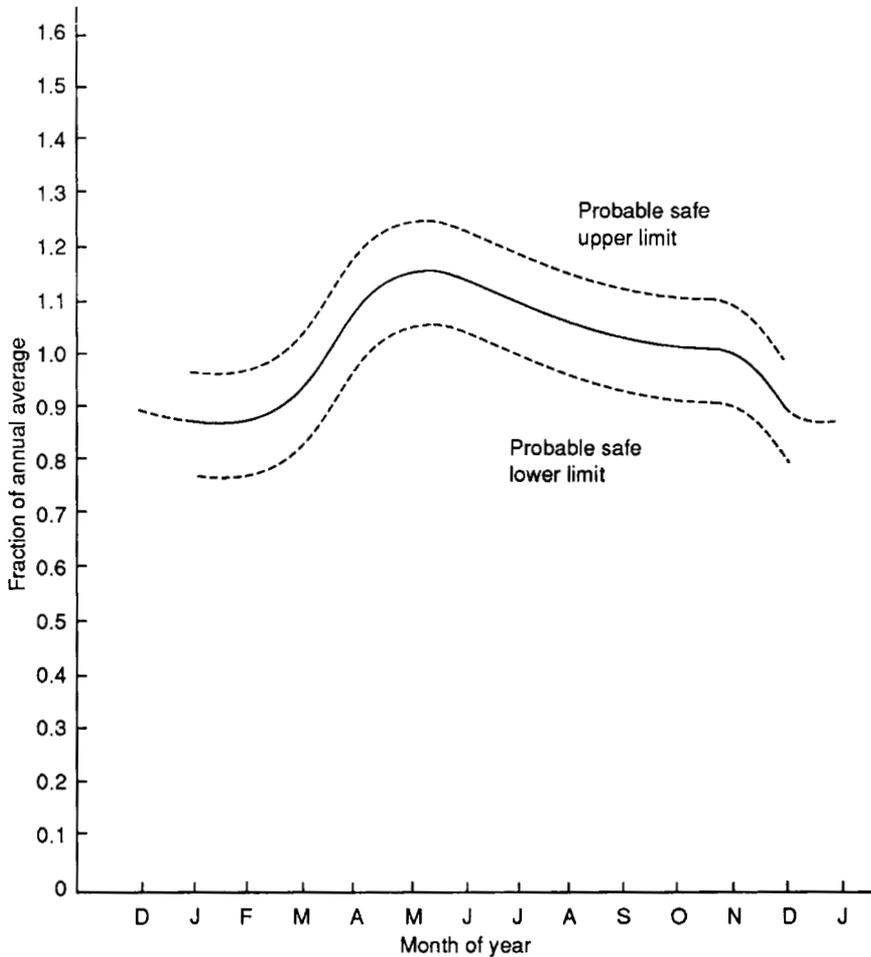


FIGURE 8.2 Monthly variations in solid waste generation (2).

TABLE 8.2 Unit Waste Factors for Various Generators

Category or generator	Unit waste factor
Commercial (SIC 50–99)	5.75 lb/employee/day (4)
Industrial (SIC 19–49)	10.6 lb/employee/day (4)
Transportation equipment (SIC 37)	20.5 lb/employee/day (5)
Nonelectrical machinery (SIC 35)	25.5 lb/employee/day (5)
Electrical machinery (SIC 36)	23.5 lb/employee/day (5)
Hospitals	2 to 4.5 lb/staff/day (6)
Patient care	8.6 lb/bed/day (6)
Food service	2.7 lb/bed/day (6)
Rehabilitation care	6.4 lb/bed/day (6)
Prisons	4.5 lb/inmate/day (7)
Universities with student housing	1.0 lb/student/day (7)
Colleges without student housing	0.6 lb/student/day (7)
Office buildings	1.5 lb/employee/day (7)
Multiple housing units	2.7 lb/resident/day (7)
Wood industry	151.0 lb/employee/day (4)
Demolition/construction debris	1.2 lb/capita/day (4)
Street sweepings	0.3 lb/capita/day (4)
Agricultural	13.0 lb/capita/day (4)
Campgrounds	1.3 lb/camper/day (5)
Family picnicground	1.0 lb/picnicker (5)
Group picnicground	1.16 lb/picnicker (5)
Organization camp	1.81 lb/occupant/day (5)
Resort area	
Rented cabin (with kitchen)	1.46 lb/occupant/day (5)
Lodge room (no kitchen)	0.59 lb/occupant/day (5)
Restaurant	0.71 lb/meal served (5)
Residence	2.31 lb/occupant/day (5)
Ski area	
Overnight lodge (all facilities)	1.87 lb/visitor-day (5)
Day lodge (all facilities)	2.92 lb/visitor-day (5)
Observation site	0.05 lb/incoming axle (5)
Visitor center	0.02 lb/visitor (5)
Swimming beach	0.04 lb/visitor (5)
Concession stand	0.14 lb/patron (5)
Administrative residence	1.37 lb/occupant/day (5)

Note: 1 lb = 0.454 kg.

Refuse collection involves the selection of vehicles and their routing through the community to most efficiently collect the solid wastes generated. Administrative decisions must be made as to requiring the public to set their containerized refuse at the curbside for pickup or to require the collectors to pick up the wastes from the backyard or other points of storage.

The principal environmental effects of collection relate to the care exercised by the collector in avoiding spilling refuse from the containers and picking up loose material. Empty containers left at the curbside after collection can be unsightly and present a potential hazard to traffic if they roll out into the street.

The environmental effect of various processing and disposal methods will be addressed in later sections.

SOLID WASTE CHARACTERIZATION

Any system employed to process waste must have the inherent flexibility to cope with its varying character. There are myriad influences of varying intensity that continually alter the quantity, composition, and physical and chemical character of the material. The intensity of the influences can vary within the community, from community to community, from year to year, from season to season, and even from day to day.

There are three primary purposes for waste characterization. First, the data become the basis for planning economic analysis, design, and subsequent management and operation of a disposal system or materials–energy resource recovery facilities. These data must consider the varying nature of the material to be processed. Second, waste characterization for rehabilitation or retrofit of a facility redefines the quantity and type of waste for disposal. For this purpose, waste recharacterization is concerned with marked changes in legislation or in the economy in general that may have some effect on the waste generated. Third, plant optimization, emissions monitoring, or malfunction analysis of a waste-to-energy facility can be expedited by the characterization of the wastes being processed. Therefore, a sampling characterization program can delineate the major constituents of the waste—such parameters as moisture and ash content—which have consequential impact on the energy value of the materials, and other parameters that influence combustion and the character of gas emissions.

Although these precepts have been widely recognized and much effort has been expended in waste characterization at various locations in the United States and abroad, there have been no standard methods, procedures, or programs established. Each investigator has resorted to his or her own devices, ingenuity, resourcefulness, and expediency to satisfy the current need for information and data. Consequently, correlations of the data obtained by many investigators (federal and state agencies, municipal administrations, involved industries, trade associations, plant operators, consulting engineers, academic researchers, and even citizen groups) using an array of techniques and procedures for differing assortments of constituents still prompt misgivings regarding the confidence level in the information reported.

The primary concern in waste characterization is the selection of the sample, its size, and the number of samples necessary to provide confidence that the sample and data are representative of the large mass of material. The secondary concern is in the actual analysis technique(s).

These concerns surface repeatedly when attempting to finalize mutually beneficial commercial agreements between producers and users of secondary materials and fuels derived from municipal waste. The American Society of Testing and Materials (ASTM) E-38 Committee on Resource Recovery addressed these concerns with consensus guidelines and standards that can be the basis of agreements minimizing many of the uncertainties.

Some investigators endeavored to characterize “their own household discards” with the expectation that they would be typical of the community. Others sorted and characterized crane bucket loads of material randomly drawn from an incinerator waste-receiving bin, and still others grabbed samples from newly dumped loads of material on the floor of a processing facility or landfill. In establishing the credibility of the data generated (reproducibility of the data being the objective), the size of each grab sample, the number of samples, and the location taken from the mass of material were areas of uncertainty.

Statistical analyses have been made and reported by several researchers regarding the efficacy of drawing many small size samples [200 to 300 lb (90 to 140 kg)], to determine waste composition and subsequent chemical analysis (8–13). Although there is apparent recognition that as-discarded, heterogeneous materials are coarse and fine in size, dense, compressible, loose, bagged, boxed, do not have granular characteristics, do not flow, do not blend but will segregate, the “cone-and-quartering” technique is frequently employed to obtain “the representative sample” for analysis. This sample selection technique is highly dependent on crew judgment; therefore, it can easily and inadvertently become biased. Although the cone-and-quartering technique is an expedient procedure to obtain a sample of reasonable size for closer analysis, it is best employed when the materials to be sampled are reasonably uniform in size and density.

This section is confined to describing two procedures for sampling and characterizing as-discarded municipal solid waste. These procedures are

- Truckload sampling
- Spot sampling

Sampling and chemical analysis procedures and standards for prepared refuse-derived fuels (RDF) and the separated materials for recycling is available in the American Standards and Testing Materials (ASTM) Standards, Water and Environmental Technology, Sec. 11, Vol. 11.04.

Many of the standards and procedures described can also be applied to commercial and industrial office and shipping waste as differentiated from production-line waste.

The ASTM publication STP 832, "Thesaurus on Resource Recovery Terminology," can also be a useful document.

Solid Waste Constituents

Municipal solid waste (MSW) consists of both materials and products. Materials in MSW include paper and paperboard, yard trimmings, glass, metal, plastic, wood, and food wastes. With exception of food wastes and yard trimmings, each material category is made up of many products. In 1995, MSW generation in the United States totaled 208 million tons; Table 8.3 provides a breakdown by material categories and associated weight (92, 93).

A portion of each material category was recycled, including being composted. Recovery rates for some products within a material category are higher than the overall recovery rate for the material category, because some products are not recovered at all. For example, aluminum cans are recovered at rates above 60%,

TABLE 8.3 Generation and Recovery of Materials in MSW (93)

Material	Weight generated, million tons	Percent of total weight generated	Weight recovered, million tons	Recovery as percent of generation
Paper and paperboard	81.5	39.2	32.6	40.0
Glass	12.8	6.2	3.1	24.5
Metals				
Ferrous metals	11.6	5.6	4.2	36.5
Aluminum	3.0	1.4	1.0	34.5
Other nonferrous metals	1.3	0.6	0.9	69.4
Plastics	19.0	9.1	1.0	5.2
Rubber and leather	6.0	2.9	0.5	8.9
Textiles	7.4	3.6	0.9	12.2
Wood	14.9	7.2	1.4	9.6
Other materials	3.6	1.7	0.8	23.1
Other wastes				
Food wastes	14.0	6.7	0.6	4.1
Yard trimmings	29.8	14.3	9.0	30.3
Miscellaneous inorganics	3.2	1.5	Negligible	Negligible
Total other wastes	46.9	22.5	9.6	20.4
Total municipal solid waste	208.0	100.0	56.2	27.0

but the overall recovery rate for aluminum is only 35%. Likewise, even though corrugated containers are recovered at rates above 64%, the overall recovery rates for paper and paperboard is 40%.

Products in MSW are grouped into three main categories: (1) durable goods, such as appliances; (2) non-durable goods, such as newspapers; and (3) containers and packaging. Other wastes include food wastes and yard trimmings. These product categories generally contain each type of MSW material. Table 8.4 presents a summary by product categories and associated weight from the 1995 characterization study.

TABLE 8.4 Generation and Recovery of Products in MSW by Material (93)

Material	Weight generated, million tons	Percent of total weight generated	Weight recovered, million tons	Recovery as percent of generation
Durable goods				
Ferrous metals	8.7	4.2	2.7	30.7
Aluminum	0.8	0.4	Negligible	Negligible
Other nonferrous metals	1.3	0.6	0.9	69.4
Total metals	10.8	5.2	3.6	33.1
Glass	1.3	0.6	Negligible	Negligible
Plastics	6.2	3.0	0.2	3.8
Rubber and leather	5.2	2.5	0.5	10.3
Wood	4.2	2.0	Negligible	Negligible
Textiles	2.3	1.1	0.1	50
Other materials	1.1	0.5	0.8	77.8
Total durable goods	41.9	20.1	5.3	17.0
Nondurable goods				
Paper and paperboard	43.5	20.9	12.7	29.3
Plastics	5.1	2.5	Negligible	<1
Rubber and leather	0.8	0.4	Negligible	Negligible
Textiles	5.0	2.4	0.8	15.8
Other materials	2.7	1.3	Negligible	Negligible
Total nondurable goods	57.0	27.4	13.5	23.7
Containers and packaging				
Steel	2.8	1.3	1.6	54.6
Aluminum	2.0	1.0	1.0	51.6
Total metals	4.8	2.3	2.6	53.4
Glass	11.5	5.5	3.1	27.3
Paper and paperboard	38.1	18.3	19.9	52.3
Plastics	7.7	3.7	0.7	9.7
Wood	10.6	5.1	1.4	13.5
Other material	0.1	>0.1	Negligible	Negligible
Total containers and packaging	72.9	35.0	27.8	38.1
Other wastes				
Food wastes	14.0	6.7	0.6	4.1
Yard trimming	29.8	14.3	9.0	30.3
Miscellaneous inorganics	3.2	1.5	Negligible	Negligible
Total other wastes	46.9	22.5	9.6	20.4
Total municipal solid waste	208.0	100.0	56.2	27.0

For planning and design projects, characterization of solid waste constituents must be tailored to the area or generator served. Two important reasons for sorting and sampling MSW are:

- To determine the constituent mix from the standpoint of recovered secondary materials for reuse or recycling
- To determine the character of the waste for use as a fuel or just incineration

To address these objectives, the following are deemed to be a practical array of constituents of interest usually found in municipal solid wastes.

Combustible	Noncombustible
Newsprint	Ferrous
Other paper	Aluminum
Diapers	Nonferrous
Textiles and garments	Glass
Plastics, film	Brick
Plastics, rigid	OBW (oversize bulky wastes)
Food wastes	
Wood	
Yard wastes (grass clippings)	
Sweepings (floor of sorting area)	

While sorting the gross sample or increment of waste into these constituents and determining the weight percentage, random fractions of the combustible constituents are accumulated for subsequent laboratory analysis to determine basic fuel characteristics such as moisture, calorific (heating) value, and ash content.

Sampling Methodology

In endeavoring to define the character of municipal solid waste, it is necessary to obtain representative samples. The number and size of samples depends upon the variability in actual physical size and other properties of the constituents, as well as the confidence level desired.

Municipal solid waste is typically made up of a broad spectrum of materials, some abundant and some sparse, as well as a cross section of physical size from mattresses to dirt. Since the bulk of waste is quite coarse, a relatively large sample increment may be necessary for high-confidence-level characterization. In addition to the bulk size of some wastes, there are other constituents that are quite small and sparse in the mix of waste. In order to not lose the concentration of sparse constituents, a large sample size is usually necessary to reasonably assure accountability of these constituents. If all constituents were relatively uniform in size, such as found in a bushel of mixed nuts, a modest sample size could more readily represent the total and conventional statistical sampling theory would apply.

The objective of a sampling program is to determine the character of the waste by sorting it into representative constituents of interest and comparing the analysis with data obtained elsewhere by other investigators.

Recognizing the variabilities of municipal solid waste, particular attention must be directed to avoiding bias when obtaining the samples. The program timing should avoid possible extraordinary external influ-

ences on the character of the wastes such as pre- and post-holidays, or prolonged adverse weather conditions. The following sampling procedures are based on obtaining sufficient repetitive, truly random samples, thereby obtaining replicable high-confidence-level data.

Regardless of the sampling method, it is necessary to establish areas of the community from which the sample will be drawn, as well as when and how this shall be done. The community planner, in conjunction with the individual(s) responsible for waste collection and disposal, should delineate the geographic areas from which the waste character might tend to be different. These areas may include tenements, condominiums, townhouses, single-family lots, campus areas, permanent residential, transient resort, and other socioeconomic factors.

If not already a local collection practice, arrange for once-per-week collection for the selected area(s) during the sampling period, thereby avoiding the day of the week variability or bias that might occur in the waste character.

Truckload Sampling

Each day, randomly select a loaded collection truck from one of the designated areas for waste sorting and characterization. An attempt should be made to limit the weight of the truckloads to 3 or 4 tons (2.7 or 3.6 metric tons)—a manageable quantity for sorting in one day. The procedure should be followed each day until all of the designated geographic (planning) areas have been sampled and characterized. The community should be divided into at least five geographic areas. A reasonable time interval should elapse, perhaps two or three days, following heavy weather to minimize the bias that is bound to occur (14).

Planning the Sorting Program. The sorting area should be indoors and sufficiently large to permit a collection vehicle to deposit its collected load and still have available adequate space for personnel to maneuver easily with the sorting drums, laboratory sampling drums, and access to a 500-lb (225-kg) platform scale. A typical sorting facility arrangement is shown in Figure 8.3. The area should be about 30 ft (9 m) wide by 60 ft (18 m) long, well-lit, and ventilated. The floor should be of a smooth, easily cleanable surface. Provisions should be made for a standby container or collection truck to dispose of the materials after they have been sorted, weighed, and sampled.

Prior to the start of the program, the waste collectors should be contacted and advised that one of their trucks will be randomly selected each day to deposit their load in the sorting room after having been weighed. The waste collectors may also be requested to have their drivers fill out a refuse collection vehicle survey data sheet (Figure 8.4), describing in detail the route of the truck, truck number, fuel tank capacity, and truck tare weight. Drivers of municipal collection vehicles are usually the most cooperative. The gross and net weights of each truck delivering to the sorting area should be documented. (The actual weight of the refuse unloaded from the truck is subsequently compared with the total weight of the constituents sorted. The net truck load weight should turn out to be somewhat higher—the difference being the weight of moisture that may have evaporated during the sorting activity.)

The information on the survey sheet will provide the weight of the waste to be characterized and also the opportunity to develop a waste profile correlating the quantity and waste character with the residential community served, based on population and the socioeconomic and age base. This correlation can be valuable in planning for other activities and facilities serving the community.

Another method of determining the per capita waste generation rate that may be influenced by the character of the community is a curbside weighing program. This program is relatively simple and can be initiated and terminated quickly. Further discussion of this method is found at the end of this section.

The following is a list of equipment recommended to conduct this sorting program:

- A portable platform (dial) scale with a maximum capacity of 500 lb (225 kg) with ¼-lb (0.1-kg) graduations

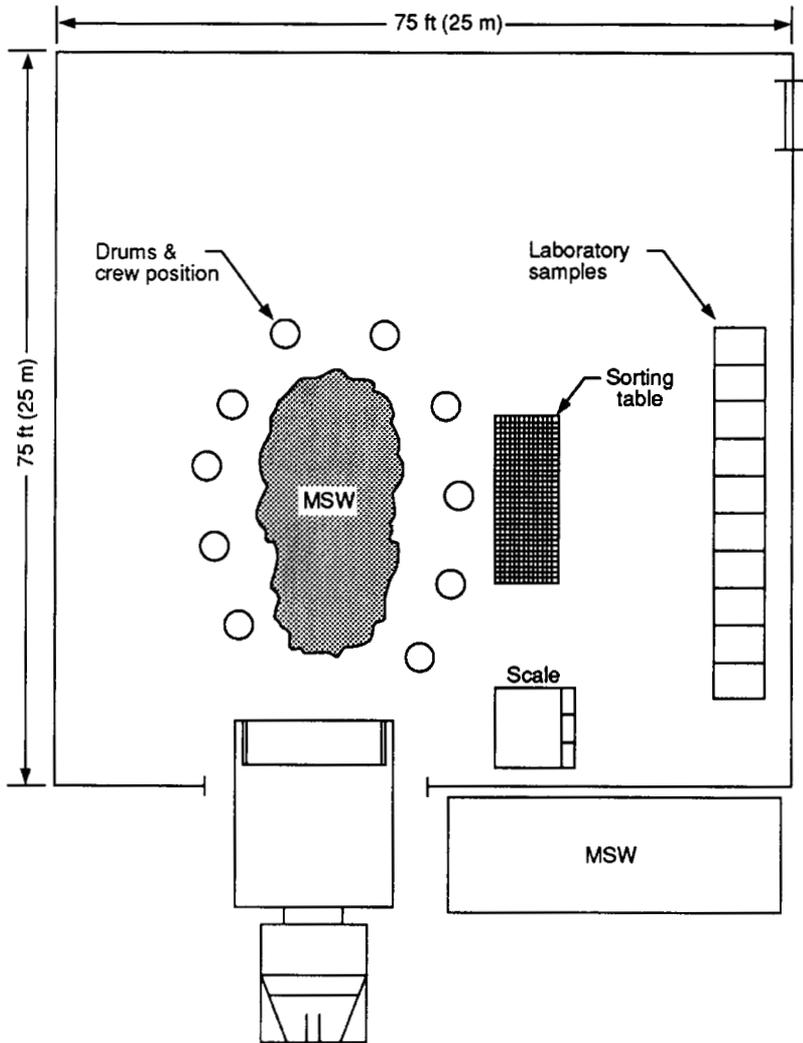


FIGURE 8.3 Truckload sorting facility arrangement.

- Twenty 32-gal (120-L) heavy-duty plastic trash containers equipped with (detachable) casters for easy mobility
- A pair of long-sleeved coveralls for each crew member
- Four snow shovels
- Two rakes
- Two heavy-duty pushbrooms
- Twelve pairs of heavy-duty puncture-resistant gloves
- Heavy work boots for each crew member

Refuse Collection Vehicle Survey Data

Date _____ Time 8:10 Weather Conditions Cloudy

- What Community? Inkster
- What Area within Community? Southwest
- Is it all Residential? Yes How many Commercial? few
- Residential area (indicate first and last street collected)

From <u>Beech Daley Rd</u>	To <u>John Daly Rd</u>
<u>Inkster Rd</u>	<u>Harrison Ave</u>
_____	_____
_____	_____
_____	_____
- Anything unusual about trash pickup? Many moving cartons

Is there much grass? more trash than grass

Bulky Waste? sofa, table, 3 chairs
- Collection Frequency per Week: one two three (circle one)
- Collection Company Municipal

Gross Weight 49,800

Net Weight 22,900

Truck Number IN 49

Fuel Capacity 50

Gallon Level: 1/4 1/2 Full (circle one)
- Other Comments Single family homes, some garden apartments

Recorded By: J. Barr

Original signed by: _____

Note: 1 lb = 0.45 kg
1 gal = 3.8 L

FIGURE 8.4 Refuse collection vehicle survey data.

- One hundred, 4-mil-thick 32-gal (120-L) plastic bags for constituent samples
- A 12 ft × 4 ft × 3 ft (3.5 m × 1.2 m × 1 m) sorting table with 1-in (2.5-cm) square screening
- Fifty heavy cardboard boxes for sample storage
- Tape, identification tags, and marker for laboratory samples
- Two handheld quick-release magnets for metal sorting

The laborers necessary for this sorting and sampling program should consist of 10 workers, including the crew chief.

The facility layout for the program should include a portable scale placed as close as possible to a collection truck hopper, container or 30 yd³ (20 m³) roll-off receptacle for the deposition of materials that have been sorted and weighed. The sorting area should easily accommodate the largest load to be characterized. Laboratory sample boxes should be placed adjacent to the scale for ease of sample deposition and packaging.

Sorting Methodology. After being weighed, the collection truck would deposit its load in the center of the sorting floor. The sorting crew members circle the deposited load while pulling one or two of the plastic trash drums mounted on casters. Each member of the team is assigned a specific item on the list of constituents to remove from the pile.

The most effective technique first addresses those constituents in the greatest abundance. When the drums have been filled with sorted material, the crew member wheels them to the platform scale and returns with empty drums to repeat the procedure.

The crew chief and scale attendant weigh each filled sorting drum and record its net weight on a special data sheet. A sample sorting weight chart is shown in Figure 8.5. (Tare weights of the empty drums should have been recorded previously.) The scale attendant then selects pieces from the contents in the sorting drums considered most representative of that particular constituent and deposits these pieces into the designated laboratory sample bag that has been assigned for that constituent. (Laboratory analysis sampling is discussed further in the following section.) The sorting drum, after having been weighed and sampled, is emptied into the receptacle or rear loading packer for discard. The emptied drum is then returned to the scale area for reuse in sorting. The materials considered oversized and bulky, such as tires, mattresses, toilets, and rugs, are pulled out for separate classification as oversized bulky wastes (OBW) and photographed.

Initially, all metals are placed into the same sorting drum. Subsequent differentiation of ferrous and non-ferrous is accomplished by emptying the contents of the drums onto a clean section of the floor and passing a handheld quick-release magnet over it to draw off the ferrous fraction. This weight should be recorded as described above.

After the mound of trash is reduced to approximately one-fourth of its original size, a filtration of fairly small top-size trash particles [less than 6 in (15 cm)] will have become apparent. Sorting trash with diminishing top sizes is especially laborious and greatly adds to sorting time. Therefore, a sorting table can be used for the remaining portion (Figure 8.6). Two or three of the crew members and at least four drums are assembled around the table. One of the members, using a snow shovel, scoops some of the remaining waste to be sorted onto the table. The remaining crew members then sort from the table. This “assembly line” technique is less taxing and improves sorting time and accuracy. All particles sifting through the 1-in (2.5 cm) square screening of the table are weighed and labeled as sweepings for further analysis.

The daily total of net weights from the sorting drums should be compared with the net MSW truckload delivered. The possible loss in weight can be attributed to the moisture loss during exposure and handling from the day-long sorting activity. Although this difference in weight may be small (0.3 to 3.0%) it can be distributed among the constituents based on the assumed tendency for that particular constituent to pickup or lose moisture, thereby providing a moisture loss “adjusted” value.

RAW REFUSE CHARACTERIZATION SAMPLING (GAI)																	
SORTING WEIGHTS																	
RECORDER R. McGee	TRUCK IDENT. IN-49	WASTE SOURCE Inkster -- (Inkster-Beech Daily to John Daly Street)															
DATE	TRUCK GROSS WT. 49,800#	TRUCK NET WT. 22,900#	PAGE 1 OF 8														
TIME 8:10 am																	
CATEGORIES	PER-CENT 100	TOTAL WEIGHT 22739	BARREL NO. (NET WEIGHT)														
1. NEWSPRINT	9	1961	63	47	43	60	40	34	58	51	25	75	69	66	55	70	
2. OTHER PAPER	25	5787	37	28	27	25	27	24	28	60	75	35	88	80	24	30	
3. DIAPERS	1	275	50	40	30	34	45	19	15	10	32						
4. TEXTILES/GARMENTS	5	1091	53	51	30	30	22	20	58	46	48	63	47	45	60	34	
5. PLASTIC (FILM)	4	866	20	17	24	44	25	12	13	12	19	12	22	20	24	27	
6. PLASTIC (RIGID)	2	528	23	14	15	14	15	13	20	10	19	26	25	20	33	15	
7. FOOD WASTE	4	787	60	80	60	60	47	50	60	35	55	99	48	49	52	32	
8. WOOD	2	340	13	40	55	65	25	70	42	40							
9. YARD WASTE	36	8203	48	65	48	98	64	60	107	75	99	88	40	82	104	68	
10. SWEEPINGS	1	257	105	90	62												
11. FERROUS	3	690	25	22	30	35	25	20	20	17	20	36	35	40	40	40	
12. ALUMINIUM	0	37	17	20													
13. NON FERROUS	0	108	70	38													
14. GLASS	3	673	60	60	45	61	20	90	44	40	30	28	62	55	38	40	
15. BRICK	1	152	37	115													
16. ODW	4	984	85	77	88	118	85	76	100	78	88	89	18	60	22		
REMARKS: *ADD 16# TO CATEGORIES 1 - 10 TO COMPENSATE FOR MOISTURE LOSS																	
TRUCK NET WEIGHT: 22900#																	
SORTED TOTAL WT: 22739#																	
DIFFERENCE: 161#																	
TOTAL DAYS TONNAGE: 811.0																	
ASSUME WEIGHT DIFFERENCE IS MOISTURE LOSS DURING SORTING/SAMPLING OPERATIONS																	

FIGURE 8.5 Sorting weight chart (14).

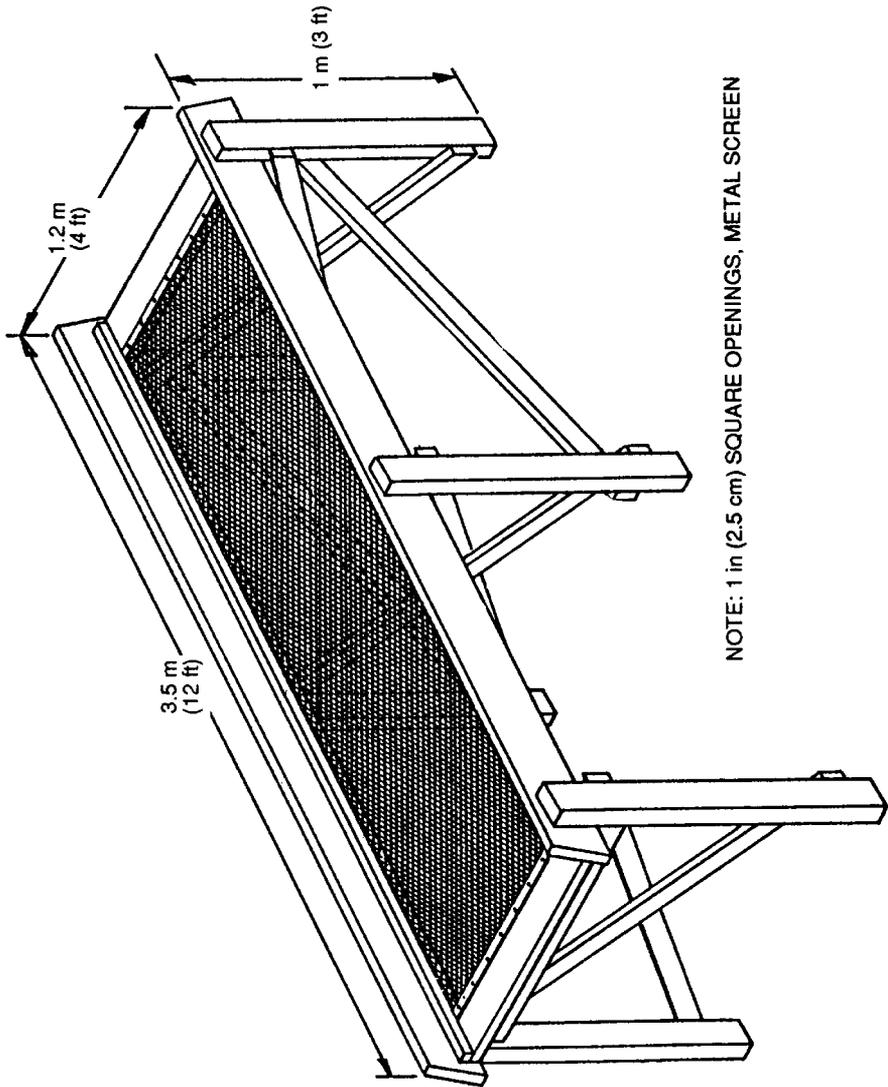


FIGURE 8.6 Sorting table.

All jugs, jars, and bottles that appear to contain material are arbitrarily deemed to be “hazardous” and should be accumulated unopened for disposal. Typically, there are so few “hazardous” containers that an accounting of them has been considered of no consequence.

Compositing Samples. The procedure to obtain representative constituent samples for laboratory analysis is to randomly select approximately 3 ft³ (0.08 m³) of each specific material from each truck load. By accumulating small samples from each sorting drum, a daily composite sample of each of the 10 combustible categories is obtained for laboratory analysis for a total of 50 samples for a five-day sampling program. Potential bias is minimized since constituent materials of varying sizes are selected randomly during the day from each sorting drum.

At the end of each day, the 10 bags of laboratory samples should be securely sealed. Each sealed bag is then resealed in another 4-mil plastic bag, labeled, weighed, and dated. The weight is logged onto a laboratory specimens chart (Figure 8.7). This double-sealed sample is then placed into a corrugated box, sealed with plastic tape, labeled, and stored. The boxed laboratory samples should be logged into the laboratory within one week after completion of the sampling program.

The program should address each waste constituent separately. Subsequently, the laboratory parametric analyses for each constituent are combined to form composite analyses to generally characterize the waste as a whole. For the data to have credibility, the elements of prime concern include the number, size, and representativeness of the laboratory samples.

It is only really necessary to have laboratory analyses conducted on constituents numbered 1 through 10, the designated combustible categories, to yield the data considered to be of practical significance. Constituent categories numbered 11 through 16 are considered to be essentially noncombustible, contributing little other than ash (residue) when consumed in a furnace. Nevertheless, it should be recognized that some surface moisture and some combustible material are present in these items such as, labels, decals, and coatings (paints). Containers may contain some organic residues and “some” oxidation of the container material itself would take place. However, the weight percent of the lot is usually small relative to the whole. The weight fraction of the combustible portion would be so small that the complexities of specific laboratory analyses may not be justified.

Data Summaries

The field and laboratory data obtained and the correlations prepared can result in many charts and tables. These can be synthesized into summary tables and charts, such as those illustrated herein.

Constituent Makeup. The constituent makeup of the waste for each day of the five-day program and their composite averages are shown in Tables 8.5 and 8.6. These data reflect the “as-sorted,” as well as the “adjusted” (moisture-distributed), weights discussed previously.

So that correlations and extrapolations might readily be made, the data can also be presented to reflect the constituent mix on a “yard-waste-free” basis. The principal yard waste usually encountered is grass clippings. Presenting the data in this manner also provides an indication of the waste constituent makeup that might be encountered during the months of little yard activity. This information is particularly significant in view of the high percentage of yard waste that can be encountered in the waste and its very high moisture content (especially in townhouse and suburban areas of the community). The impact of yard waste on the character of the refuse is dramatic. Table 8.5 reveals that it can be 40% of the total weight or 48% of the combustible portion, and can average 70% moisture.

Similarly, data summaries can be structured with and without the noncombustibles (constituents 11

**RAW REFUSE CHARACTERIZATION SAMPLING (GAI)
LAB SPECIMENS**

RECORDER R. McGee DATE 9/28/89 TRUCK IDENT. IN-49 WASTE SOURCE Inkster - (Inkster-Beech Daly to John Daly Street)

TRUCK GROSS WT. 49,800# TRUCK NET WT. 22,900# PAGE 1 OF 1

CATEGORIES	CATE- GORY WT%	BAGGED WT 182	DATE LAB RECD	H ₂ O ANALYSIS		ASH ANALYSIS		HEAT CONTENT											
				IDENT	TEST DATE	IDENT	TEST DATE	IDENT	TEST DATE	IDENT	TEST DATE								
1. NEWSPRINT		15																	
2. OTHER PAPER		13																	
3. DIAPERS		16																	
4. TEXTILES/GARMENTS		26																	
5. PLASTIC (FILM)		0																	
6. PLASTIC (RIGID)		8																	
7. FOOD WASTE		14																	
8. WOOD		18																	
9. YARD WASTE		28																	
10. SWEEPINGS		38																	
OTHER																			

NOTES:

1. ASH ANALYSIS: % ASH IS FOR MOISTURE FREE SPECIMENS. % ASH OF AS-RECEIVED SAMPLE = % ASH_{MF} $\left(1 - \frac{\% H_2O}{100} \right)$

2. HHV ANALYSIS IS ON A MOISTURE FREE BASIS. HHV FOR AS-RECEIVED SAMPLE = HHV_{MF} $\left(1 - \frac{\% H_2O}{100} \right)$

FIGURE 8.7 Lab spectrum chart (14).

through 16), so as to readily reflect the makeup of the organic and combustible fraction of waste—as if subjected to a highly selective and efficient form of front-end or source-separation program. A data display as given in Table 8.4, can be especially informative. This illustration indicates that for the five-day composite, the noncombustible constituents were only 11.3% of the total waste (as discarded), and 18.7% when reported on a yard-waste-free basis.

The weight percentages illustrated for ferrous, aluminum, and glass appear to be lower than the values usually reported elsewhere, even on a yard-waste-free basis (19). Perhaps the values are closer to reality or reflect the impact of the recent passage of a regional “bottle bill.”

Several other items draw focus; the very low percentages of food waste (less than one-third the values usually reported), the low percentages of aluminum, nonferrous, and particularly glass (less than half what is usually reported), and the high percentage of plastics (more than one-fourth greater than usually reported). So much of the textiles, garments, and footwear are composed (at least in part) of synthetic materials that categorizing them expediently can be taxing. The weight percent for this constituent was generally higher (twice) than expected.

This sampling procedure reduces the size of the catch-all usually described as “miscellaneous.” The constituent designated as sweepings is the catch-all for the described procedure. The quantity encountered is comfortably small.

As mentioned previously, by programmed intent, collection vehicles should be randomly selected for waste load characterization based on knowledge of the general residential areas they serve. In this manner, the data can reflect the variation in waste character based on socioeconomic and age considerations. Interestingly, the tabulations in Table 8.5 did not reveal marked differences in constituent concentrations. There was only modest variation from locale to locale in the weight percent of the constituent mix during this characterization program.

Laboratory Analysis. An analysis should be made only for the parameters of fundamental interest. For this program, the parameters selected were moisture, ash (inerts), calorific value, sulfur, total chlorine, and water-soluble chlorides. The analytical procedures and methods should closely adhere to the consensus standards developed for municipal waste constituents as described in the ASTM Standards, Water and Environmental Technology, Sec. 11, Vol. 11.04.

Moisture. The daily and composite moisture data for the 10 “combustible” constituents are displayed in Table 8.7. The high moisture value(s) for the waste can be attributed principally to the relatively high weight percent of the (high-moisture-content) yard waste constituent. The high moisture values for diaperlike materials and food wastes were as expected. However, the high moisture (22%) for plastic film was not anticipated and may be attributed to the extensive use of plastic bags as the containment for the large quantity of very wet grass clippings. Therefore, this high moisture for plastic film should be considered as essentially surface (free) moisture rather than inherent moisture.

These values could be recast and tabulated on the basis of the total weight of all 16 constituents, which would indicate 42.1% moisture versus 47.4%. Recasting the data on a yard-waste-free basis, the five-day composite values would be 28% moisture for the combustible portion and 22.8% moisture based on the remaining 15 constituents. This wide range in moisture values illustrates the care that must be taken in the manner of reporting moisture data to avoid creating an erroneous impression regarding the character of the waste.

Ash. A similar tabulation for the inherent ash (dry) in the combustible materials is displayed in Table 8.8. The tabulated high ash content for sweepings should be of little concern since the quantity of sweepings is typically small. The ash content in plastic film and in yard waste is higher than expected. However, the data reported should not be considered as absolute values for each of the constituents; e.g., for plastic film there must have been moisture adhering to the film surface, as well as dirt, grass clippings, and other small particles whose weight is quite high relative to the very light weight of the plastic film itself. Overall, the five-

TABLE 8.5 Daily Refuse Constituent Weight Distribution (14)

Constituent	RAW REFUSE CHARACTERIZATION SAMPLING AND ANALYSES											
	REFUSE SORTING PROGRAM				AUGUST 27 THROUGH 31				AUGUST 27 THROUGH 31			
	TOTAL NET WEIGHT: 65,800 LB				TRUCKS: DH 452, IN-49, NA-20, WE-6S, B-234				TRUCKS: DH 452, IN-49, NA-20, WE-6S, B-234			
	August 27		August 28		August 29		August 30		August 31		August 31	
	Weight, lb as sorted	Weight, %	Weight, lb as sorted	Weight, %	Weight, lb as sorted	Weight, %	Weight, lb as sorted	Weight, %	Weight, lb as sorted	Weight, %	Weight, lb as sorted	Weight, %
1. Newsprint	822	6.84	1,961	8.62	1,250	8.64	853	7.94	402	7.96	402	7.96
2. Other paper	2,967	24.7	5,787	25.5	3,373	23.3	2,098	19.6	1,447	28.6	1,447	28.6
3. Diapers	156	1.30	275	1.20	152	1.05	108	1.01	4	0.09	4	0.09
4. Textiles/garments	219	1.82	1,091	4.80	797	5.51	425	3.95	80	1.59	80	1.59
5. Plastics, film	277	2.31	866	3.80	545	3.77	259	2.40	85	1.69	85	1.69
6. Plastics, rigid	1%	1.63	528	2.31	368	2.54	256	2.37	86	1.70	86	1.70
7. Food waste	496	4.13	787	3.45	149	1.03	554	5.15	17	0.35	17	0.35
8. Wood	66	0.56	340	1.50	143	0.99	143	1.32	32	0.64	32	0.64
9. Yard waste	5,444	45.3	8,203	36.1	5,232	36.2	4,866	45.4	2,034	40.2	2,034	40.2
10. Sweepings	420	3.50	257	1.12	341	2.36	190	1.76	115	2.29	115	2.29
11. Ferrous	240	2.00	690	3.02	579	4.00	364	3.39	140	2.78	140	2.78
12. Aluminum	16	0.14	37	0.16	89	0.63	27	0.25	—	—	—	—
13. Nonferrous	35	0.29	108	0.47	97	0.68	69	0.64	—	—	—	—
14. Glass	455	3.79	673	2.95	494	3.42	276	2.56	285	5.64	285	5.64
15. Brick	19	0.16	152	0.67	724	5.02	30	0.28	—	—	—	—
16. OBW	182	1.53	984	4.33	125	0.86	212	1.98	327	6.47	327	6.47
Total	12,010	100.0	22,739	100.0	14,458	100.0	10,730	100.0	5,054	100.0	5,054	100.0
Truck net weight	12,200		22,900		14,500		11,000		5,200		5,200	
Difference	190	1.56	161	0.70	42	0.29	270	2.46	146	2.18	146	2.18

Note: 1 lb = 0.45 kg.

TABLE 8.6 Constituent Weight Distribution—5-Day Composite (14)

REFUSE SORTING PROGRAM COMPOSITE OF 5 DAYS COLLECTION		RAW REFUSE CHARACTERIZATION SAMPLING AND ANALYSES									
		Adjusted for moisture loss during sampling					Excluding yard waste				
Constituent	Weight, lb sorted	Weight, %	Weight, lb ad- justed	Weight, %	Weight, lb ad- justed	Weight, %	Weight, lb ad- justed	Weight, %	Weight, lb ad- justed	Weight, %	
1. Newsprint	5,288	8.14	5,322	8.09	5,322	9.12	5,322	13.4	5,322	16.4	
2. Other paper	15,672	24.1	15,726	23.9	15,726	26.9	15,726	39.5	15,726	48.5	
3. Diapers	695	1.07	835	1.27	835	1.43	835	2.09	835	2.58	
4. Textiles/garments	2,612	4.02	2,644	4.01	2,640	4.52	2,640	6.62	2,640	8.14	
5. Plastics, film	2,032	3.13	2,089	3.17	2,089	3.58	2,089	5.24	2,089	6.44	
6. Plastics, rigid	1,434	2.21	1,449	2.20	1,449	2.48	1,449	3.64	1,449	4.47	
7. Food waste	2,003	3.08	2,194	3.33	2,194	3.76	2,194	5.50	2,194	6.77	
8. Wood	724	1.11	755	1.15	755	1.29	755	1.89	755	2.33	
9. Yard waste	25,779	39.7	25,940	39.4	25,940	44.4	—	—	—	—	
10. Sweepings	1,323	2.04	1,421	2.16	1,421	2.43	1,421	3.56	1,421	4.38	
11. Ferrous	2,013	3.10	2,013	3.06	2,013	—	2,013	5.05	—	—	
12. Aluminum	169	0.26	169	0.26	169	—	169	0.42	—	—	
13. Nonferrous	309	0.48	309	0.47	309	0.78	—	—	—	—	
14. Glass	2,183	3.36	2,183	3.32	2,183	5.48	—	—	—	—	
15. Brick	925	1.42	925	1.41	925	2.32	—	—	—	—	
16. OBW	1,830	2.82	1,830	2.78	1,830	4.59	—	—	—	—	
Total	64,991	100.0	65,800 ^a	100.0	58,372 ^b	100.0	39,860	100.00	32,431 ^c	100.0	
Difference	809	1.2	—	—	7,428	11.3 ^d	25,940	39.4	7428	—	

^aGross weight.^b% of gross weight.^c49% of gross weight.^dNoncombustible constituents—11 through 16.

Note: 1 lb = 0.45 kg.

TABLE 8.7 Combustible Constituents Moisture Distribution—Daily and Composite (14)

RAW REFUSE CHARACTERIZATION SAMPLING AND ANALYSES									
AUGUST 27 through 31									
Constituent	Adjusted total weight, lb	Moisture contribution, lb					Day S	5-Day total moisture, total lb	Composite average moisture, %
		Day 1	Day 2	Day 3	Day 4	Day 5			
1. Newsprint	5,322	108	307	287	133	49.8	885	16.6	
2. Other paper	15,726	784	1,851	736	399	296	4,066	25.9	
3. Diapers	836	125	180	92.5	103	13.8	514	61.6	
4. Textiles/garments	2,640	12.3	258	126	57.4	6.29	460	17.4	
5. Plastics, film	2,089	75.7	170	127	70.3	27.5	470	22.5	
6. Plastics, rigid	1,449	22.2	20.4	9.9	18.7	3.97	75	5.18	
7. Food waste	2,194	441	695	118	532	46.9	1,833	83.6	
8. Wood	755	5.19	59.3	18.4	25.7	5.73	114	15.1	
9. Yard waste	25,940	3,803	6,143	3,600	3,702	1,339	18,587	71.6	
10. Sweepings	1,421	230	127	179	85.4	47.4	669	47.1	
Total	58,372	5,606	9,811	5,294	5,126	1,836	27,673	47.4	
Weight collected, lb		11,253	20,256	12,392	10,022	4,448	58,372		
Moisture (yard-waste-free basis), %	49.8	42.7	51.1	41.3	47.4		28		

Note: 1 lb = 0.454 kg.

TABLE 8.8 Ash in Combustible Constituents—Daily and Composite (14)

Constituent	Adjusted total weight, lb	Ash contribution, lb (Dry basis)					5-Day total ash, total lb	Composite average ash, %
		AUGUST 27 through 31						
		Day 1	Day 2	Day 3	Day 4	Day 5		
1. Newsprint	4437	13.6	25.8	11.8	8.55	2.36	62.1	1.40
2. Other paper	11660	135	156	138	111	61.40	601	5.15
3. Diapers	322	1.51	3.78	1.02	0.99	0.34	7.64	2.38
4. Textiles/garments	2180	4.85	33.8	23.5	7.82	4.80	74.8	3.43
5. Plastics, film	1619	20.5	104	54.7	10.6	9.66	200	12.4
6. Plastics, rigid	1374	10.9	56.6	10.8	7.79	1.51	87.6	6.38
7. Food waste	361	6.77	10.0	3.98	5.60	0.70	27.1	7.51
8. Wood	641	2.25	18.4	1.51	3.61	0.40	26.2	4.09
9. Yard waste	7353	312	378	299	210	149	1348	18.3
10. Sweepings	752	55.6	51.3	45.7	55.2	44.1	252	33.5
Total	30699	563	838	590	421	274	2686	8.75
Dry weight collected, lb	5648	10447	7099	4896	2612	30699		
Ash (dry basis), % (wt. ash/wt. refuse collected)	9.97	8.02	8.31	8.60	10.5	8.75		

Note: 1 lb = 0.454 kg.

day composite ash content of the combustible constituents, 8.75% (dry basis) can be considered low. The ash content of the composite including the noncombustibles is 15.4% on an as-received basis, and 22% when reported as yard-waste-free.

Calorific Value. The higher heating value (HHV) on a moisture- and ash-free (MAF) basis is reported in Table 8.9 for each constituent. The five-day composite MAF heating value for the combustible constituents is 9673 Btu/lb (22,500 kJ/kg) and on a yard-waste-free basis (9826 Btu/lb) (22,850 kJ/kg). This value is somewhat higher than what is generally reported. The MAF heating value is a convenient base from which calculated transformations can readily be made to provide values reflecting specific entrained and inherent moisture and ash values encountered (14).

Summary tabulations of the basic thermochemical parametric data based on an as-received and yard-waste-free basis can be displayed as in Table 8.10.

Sulfur and Chlorine. In view of the intense interest and concern regarding potential acid gas formation during combustion, replicate analyses should be made for sulfur and chlorides on samples obtained of synthetic materials in addition to the normal determinations for these parameters on the other constituents. Tables 8.11 and 8.12 illustrate summaries of such data obtained during the reference characterization program.

In an effort to determine the chlorine derivatives that may have a harmful effect in a combustion system, it is generally postulated that the chlorine atoms that are insoluble in water are those of particular concern (15). The premise for this assumption is that the temperatures usually encountered in a furnace will vaporize the insoluble forms of chlorine and will therefore readily combine with other constituents and become chemically aggressive.

Laboratory investigation of chlorine-bearing constituents usually focuses on identifying the organic or insoluble chlorine(s). This can be accomplished by deducting the values for the water-soluble chlorides (H_2O -soluble Cl^-) from the determined total chlorine values. The resulting arithmetic difference will be the organic chlorine. These values are illustrated in Table 8.11. However, a qualification is necessary when assessing these data. Not all organic chlorines are insoluble in water and not all inorganic chlorides are soluble in water. Therefore, the values in Table 8.11 can only be considered indicative, not absolute. The data usually reported in the literature are the total chlorine values rather than the arithmetic differences just discussed.

The full significance and impact of the above is apparent when examining the data in Table 8.11 reported for textiles and garments, which were sampled only on day 3. The total chlorine is reported to be 3.78% and the water-soluble chloride is 3.73%. The calculated difference of 0.05% is the organic chlorine. Although some of the as-determined values reported may seem high, it must be recognized that it is the difference in these values that should be used in assessing the degree of possible chemical aggressiveness. Inspection of the low as-determined values in Table 8.11 reveals that the arithmetic differences (organic chlorine) are much greater than what appear at first glance. This is reflected in the last column of tabulated data reported on an MAF basis.

The sulfur and chlorine values for all 10 combustible constituents were determined only for the laboratory samples obtained on day 1. These are reported in Table 8.12 on an as-received basis and reflect the composite weight averages, all of which are considered to be relatively low.

Parametric Data. For design, confirmation, or operational purposes, specific laboratory analyses are generally necessary, especially for unusual types and/or quantities of materials encountered. However, for preliminary planning purposes, overview laboratory parametric data may be adequate in lieu of conducting analyses of specific constituents. In addition to the specific characterization data available in the aforementioned tables, typical densities of waste components are found on Table 8.13, and fuel proximate analysis of typical components in discarded solid wastes is found on Table 8.13. This information may provide useful data for the investigator or planner.

TABLE 8.9 Btu in Combustible Constituents (MAF)—Daily and Composite (1/4)

Constituent	Total MAF Adjusted weight, lb	Btu contribution (MAF), Btu × 5 ³					5-Day total Btu, Btu × 10 ⁵	Composite average MAF, Btu/lb
		AUGUST 27 through 31						
		Day 1	Day 2	Day 3	Day 4	Day 5		
1. Newsprint	4375	61,749	142,212	81,958	63,046	30,869	379,834	8682
2. Other paper	11059	174,280	311,721	232,761	136,250	91,800	946,812	8562
3. Diapers	314	5,836	9,648	5,278	4,503	1,093	26,358	8394
4. Textiles/garments	2105	18,940	76,345	59,683	32,231	6,859	194,058	9219
5. Plastics, fiber	1419	29,293	118,158	59,398	34,956	9,432	251,237	17705
6. Plastics, rigid	1286	30,383	78,360	64,140	42,272	15,950	231,105	17971
7. Food waste	334	9,029	10,248	3,032	6,930	0,674	29,913	8956
8. Wood	615	5,471	24,959	13,041	10,958	2,844	57,273	9313
9. Yard waste	6005	122,031	156,517	120,006	96,844	51,959	546,993	9109
10. Sweepings	500	14,575	10,152	11,394	5,925	3,869	45,915	9183
Total	28012	471,587	938,320	650,691	433,915	214,985	2709,498	9673
MAF or combustibles weight, lb		5084	9609	6509	4475	2338	28012	
HHV (yard-waste-free basis)—Btu/lb		9276	9765	9997	9696	9195	9673	9826

Note: 1 lb 0.454 kg; 11 Btu/lb 0.43 kJ/kg.

TABLE 8.10 Comparative Summary—Moisture, Ash, HHV (14)

	RAW WASTE CHARACTERIZATION SAMPLING AND ANALYSIS			
	AUGUST 27 through 31		5-Day composite (Yard-waste-free)	
	August 27 as-received, all constituents	As-received, all constituents	All constituents	Combustible constituents
Moisture, %	45.9	42.1	22.8	28.0
Ash (as received), %	12.4	15.4	22.0	4.13
Ash (dry basis), %	22.9	26.5	28.5	5.73
HHV (as received), Btu/lb	3866	4118	5425	6668
HHV (dry basis) Btu/lb	7146	7112	7027	9263
HHV (MAF), Btu/lb	9276	9689	9826	9826
Total chlorine*	0.171			
H ₂ O chloride	0.082			
Δ Chlorine (organic)	0.089			
Sulfur*	0.058			

*See Tables 8.11 and 8.12 for more specific data on chlorine and sulfur.

Note: 1 Btu/lb = 0.43 kJ/kg.

TABLE 8.11 Sulfur and Chlorine* in Textile and Plastic Constituents (14)
RAW REFUSE CHARACTERIZATION SAMPLING AND ANALYSES

AUGUST 27 through 31

Constituent	As received			Dry basis			Moisture and ash free			
	S, %	H ₂ O-Soluble Cl ⁻	Total Cl, %	S, %	H ₂ O-Soluble Cl ⁻ , %	Total Cl, %	S, %	H ₂ O-Soluble Cl ⁻ , %	Total Cl, %	Total Cl minus H ₂ O-Soluble, %
Date 8-27										
Textiles/garments	0.007	0.026	2.33	<0.007	0.028	2.47	<0.007	0.029	2.53	2.50
Plastics, film	0.006	0.122	0.201	<0.008	0.165	0.272	<0.009	0.182	0.301	0.119
Plastics, rigid	0.38	0.143	1.15	0.43	0.161	1.29	0.46	0.171	1.37	1.20
Date 8-28										
Textiles/garments	0.030	0.146	0.295	0.039	0.190	0.385	0.041	0.198	0.401	0.203
Plastics, film	0.01	0.197	0.282	<0.01	0.244	0.350	<0.01	0.286	0.410	0.124
Plastics, rigid	0.01	0.099	0.290	<0.01	0.103	0.303	<0.01	0.116	0.341	0.225
Date 8-29										
Textiles/garments	<0.01	3.73	3.78	<0.01	4.43	4.49	<0.01	4.59	4.65	0.06
Plastics, film	<0.01	0.265	0.277	<0.01	0.345	0.360	<0.01	0.397	0.414	0.017
Plastics, rigid	<0.01	0.299	0.374	<0.01	0.307	0.384	<0.01	0.317	0.396	0.079
Date 8-30										
Textiles/garments	0.061	0.095	0.098	0.070	0.109	0.113	0.071	0.111	0.115	0.004
Plastics, film	<0.01	0.101	0.3%	<0.01	0.135	0.530	<0.01	0.142	0.558	0.416
Plastics, rigid	<0.01	0.154	0.477	<0.01	0.166	0.514	<0.01	0.172	0.531	0.359
Date 8-31										
Textiles/garments	0.022	0.058	0.322	0.024	0.063	0.348	0.026	0.067	0.371	0.304
Plastics, film	0.060	0.329	0.364	0.083	0.457	0.506	0.096	0.530	0.586	0.056
Plastics, rigid	0.074	0.032	0.086	0.077	0.034	0.090	0.078	0.035	0.092	0.057

*Total chlorine minus water-soluble chlorides (H₂O-soluble Cl⁻) = organic chlorine.

TABLE 8.12 Sulfur and Chlorine* in Combustible Constituents (14)
RAW WASTE CHARACTERIZATION SAMPLING AND ANALYSIS

Constituent†	Adjusted total weight, lb	Sulfur contribution		Total Cl, lb	H ₂ O-soluble Cl, lb	Total Cl, %	H ₂ O-soluble Cl, lb	Total Cl, %	Total Cl, minus H ₂ O-soluble Cl, %
		lb	%						
As received									
1. Newsprint	829	0.829	0.010	0.307	0.298	0.037	0.036	0.001	
2. Other paper	2,981	2.981	0.10	5.753	2.683	0.193	0.090	0.103	
3. Diapers	191	0.038	0.02	0.193	0.187	0.101	0.098	0.003	
4. Textile/garments	222	—	<0.007	5.173	0.058	2.33	0.026	2.304	
5. Plastic, film	292	—	<0.006	0.587	0.356	0.201	0.122	0.079	
6. Plastic, rigid	202	0.768	0.38	2.323	0.289	1.15	0.143	1.007	
7. Food waste	540	0.054	0.01	0.956	0.853	0.177	0.158	0.019	
8. Wood	70	0.042	0.06	0.0413	0.027	0.059	0.039	0.02	
9. Yard waste	5,480	2.192	0.04	4.173	4.603	0.086	0.084	0.002	
10. Sweepings	447	0.1341	0.03	0.796	0.648	0.178	0.145	0.033	
Total weights	11,253	7,0381		20,823	10,002				
Weighted average, %	0.063	0.185	0.096%						
11. Ferrous	240								
12. Aluminum	16								
13. Nonferrous	35								
14. Glass	455								
15. Brick	19								
16. OBW	182								
Total	12,200	7,0381		20,8423	10,002				
Weighted average, %	0.058	0.171	0.082						

*Total chlorine minus water-soluble chlorides (H₂O-soluble Cl) = organic chlorine.

†Sulfur and chlorine determinations for "all" combustible constituents (1 through 10) conducted *only* on laboratory samples obtained on August 27.

Note: 1 lb = 0.454 kg.

TABLE 8.13 Densities of Waste Components

Component	Density
Waste densities, lb/yd ³	
Loose waste	100–200
After dumping from compactor truck	350–400
In compactor truck	500–700
In landfill	500–900
Shredded waste	600–900
Baled in paper baler	800–1200
Bulk densities, lb/ft ³	
Cardboard	1.87
Aluminum	2.36
Plastics	2.37
Miscellaneous paper	3.81
Garden waste	4.45
Newspaper	6.19
Rubber	14.90
Glass	18.45
Food	23.04
True densities, lb/ft ³	
Wood	37
Cardboard	43
Paper	44–72
Glass	156
Aluminum	168
Steel	480
Polypropylene	56
Polyethylene	59
Polystyrene	65
ABS	64
Acrylic	74
Polyvinylchloride (PVC)	78
Resource recovery plant products, lb/ft ³	
dRDF	39
Aluminum scrap	15
Ferrous scrap	25
Crushed glass	85

Source: Prepared by Cal Recovery Systems, Richmond, Calif., for ASTM E-38.

Note: 1 lb/yd³ = 1.685 kg/m³ 1 lb/ft³ = 0.0624 kg/m³.

SOLID WASTE

TABLE 8.14 Fuel Proximate Analysis for Typical Components (21)

	Proximate analysis (as-received), weight %					Higher heating value, Btu/lb	
	Moisture	Volatile matter	Fixed carbon	Non-combustible	S	As-received	Moisture and ash free
Paper, mixed	10.24	75.94	8.44	5.38	0.20	6,800	8,055
Newsprint	5.97	81.12	11.48	1.43	0.16	7,974	8,600
Brown paper	5.83	83.92	9.24	1.01	0.11	7,256	7,800
Trade magazines	4.11	66.39	7.03	22.47	0.09	5,254	7,150
Corrugated boxes	5.20	77.47	12.27	5.06	0.21	7,043	7,850
Plastic-coated paper	4.71	84.20	8.45	2.64	0.08	7,341	7,940
Waxed milk cartons	3.45	90.92	4.46	1.17	0.10	11,327	11,890
Paper food cartons	6.11	75.59	11.80	6.50	0.16	7,258	8,250
Junk mail	4.56	73.32	9.03	13.09	0.09	6,088	7,400
Office trash	4.10	79.80	7.90	8.20	<.1	6,950	7,860
Vegetable food waste	78.29	17.10	3.55	1.06	0.20	1,795	8,700
Citrus rinds and seeds	78.70	16.55	4.01	0.74	0.12	1,707	8,300
Meat scraps (cooked)	38.74	56.34	1.81	3.11	0.19	7,623	13,110
Fried fats	0.00	97.64	2.36	0.00	0.07	16,466	16,466
Mixed food waste	72.00	20.26	3.26	4.48	0.52	2,370	10,100
Hardwood (pallets, crates)	11.60	74.80	14.00	0.60	<.1	7,500	8,520
Green logs	50.00	42.25	7.25	0.50	0.08	2,102	4,250
Rotten timbers	26.80	55.01	16.13	2.06	1.2	4,710	6,560
Demolition softwood	7.70	77.62	13.93	0.75	<.1	7,300	7,995
Waste hardwood	12.00	75.05	12.41	0.54	<.1	6,430	7,340
Furniture wood	6.00	80.92	11.74	1.34	<.1	7,350	7,940
Evergreen shrubs	69.00	25.18	5.01	0.81	0.19	2,708	8,960
Balsam spruce	74.35	20.70	4.13	0.82	0.20	2,447	9,850
Flowering plants	53.94	35.64	8.08	2.34	0.26	3,697	8,460
Lawn grass	75.24	18.64	4.50	1.62	0.42	2,058	8,900
Ripe leaves	9.97	66.92	19.29	3.82	0.16	7,984	9,270
Wood and bark	20.00	67.89	11.31	0.80	0.05	6,900	8,700
Brush	40.00	—	—	5.00	0.05	4,745	8,600
Mixed greens	62.00	26.74	6.32	4.94	0.05	2,690	8,135
Upholstery	6.9	75.96	14.52	2.62	<.1	6,960	7,690
Tires, whole	1.02	64.92	27.51	6.55	1.5	13,800	14,900
Leather	10.00	68.46	12.49	9.10	0.40	7,960	9,850
Leather shoe	7.46	57.12	14.26	21.16	1.00	7,243	10,150
Shoe heel and sole	1.15	67.03	2.08	29.74	1.34	10,899	15,790
Rubber	1.20	83.98	4.94	9.88	2.00	11,200	12,600
Mixed plastics	2.0	—	—	10.00	—	14,100	16,000
Plastic film	3.20	—	—	—	0.07	—	14,870
Polyethylene	0.20	98.54	0.07	1.19	0.03	18,687	20,000
Polystyrene	0.20	98.67	0.68	0.45	0.02	16,419	16,510
Polyurethane	0.20	87.12	8.30	4.38	0.02	11,203	11,730
Polyvinyl chloride	0.20	86.89	10.85	2.06	0.14	9,754	10,000
Linoleum	2.10	64.50	6.60	26.80	0.40	8,150	11,450
Rags	10.00	84.34	3.46	2.20	0.13	6,900	7,844
Textiles	15.31	—	—	—	0.20	—	8,300
Oils, paints	0	—	—	16.30	—	13,400	16,000
Vacuum-cleaner dirt	5.47	55.68	8.51	30.34	1.15	6,386	9,960
Household dirt	3.20	20.54	6.26	70.00	0.01	3,670	13,650
Street sweepings	20.00	54.00	6.00	20.00	0.20	4,800	8,000

Note: 1 Btu/lb = 0.43 kJ/kg.

Spot Sampling

The second method of sampling is randomly withdrawing many small-weight increments of waste from the mass of waste material collected daily. Since municipal waste is subject to many influences and is highly variable in character, many increments are necessary so that average values for quantity and character are representative of the whole. As a practical matter, a program withdrawing many 200- to 300-lb (90- to 135-kg) increments of waste for sorting can be reasonably indicative of the character of the waste.

A procedure that has been found to be expedient employs the use of a crane with grapple bucket, such as those found at an incinerator, resource recovery plant, or a mobile articulated unit at a transfer station or landfill (16, 17). The crane operator dumps a small [approximately 1 yd³ (0.75 m³)] randomly selected bucket load onto a clean surface area preferably covered with a plastic tarpaulin to better manage the finer particles in the waste sample. [In an incinerator plant, the sampling and sorting would take place at the furnace charging level (parapet) at one end of the receiving pit.] An attempt should be made by the crane operator to mix the mass of waste by bucket action prior to extracting the sample for sorting. Although this procedure may improve the mix of materials, it does tend to sift out the granular material and fines. There is also the tendency to avoid the larger objects encountered.

Another spot-sampling procedure involves obtaining waste increments using a front-end-loader vehicle from a series of waste collection trucks (16×18). A truck, selected at random for sampling, is unloaded on the designated floor area. Determining where from the dumped load to select the sample for sorting is the challenge. Although the material was loaded in the truck at random and some mixing does take place when the load is dumped, additional mixing should be attempted by the front-end-loader operator prior to extracting a bucket load for deposit onto the cleaned floor area for sorting and sampling. However, every effort should be made to avoid segregation of the fines and oversize materials. The procedure performed on 10 to 15 randomly selected vehicles each day for two or more weeks can provide more than 120 sets of data from sorting 12 to 20 tons (10 to 18 metric tons) of waste. This repetitive procedure tends to level the variances and temper bias. Although the percentages for each may be relatively small, the analysis data should be qualified in regard to the probable losses of granular material and fines, oversized objects, and loss of surface moisture to avoid creating erroneous impressions regarding the character of the waste investigated. Too many times significant quantities are included in a “miscellaneous” category as a catch-all expediency to close the material balance calculation.

The procedures described in the truckload sampling methodology for obtaining samples for laboratory analysis also apply to spot sampling.

As mentioned, many investigations and attempts have been made to use statistical analysis in determining the quantity and character of the wastes discarded by the community so as to project the quantity and quality of the materials and energy that may be recovered. Depending on the need and degree of accuracy necessary, some of the procedures are quite useful especially for refuse-derived products. Several of these are included in the reference list of this chapter.

A fundamental discussion on statistical sampling analysis is covered in the book *Refuse Derived Fuel Processing* (13).

A statistical analysis was conducted of the truckload sampling method and the spot sampling method. Although the extremes in the as-received heating value of the truckload sampling and analysis program ranged from approximately 3860 to 4890 Btu/lb (9000 to 11,400 kJ/kg), the comparative statistical analysis revealed that the mean HHV was 4650 Btu/lb (10,800 kJ/kg) with a standard deviation of 242; therefore there was a 5.2% coefficient of variation and a relative standard error of 300 or 6.45%. This relates that 95% of the time the as-received heating value would be expected to range from 4350 to 4950 Btu/lb (10,100 to 11,500 kJ/kg) (13).

Similarly, the spot sampling program conducted on municipal waste revealed a mean HHV of 4900 Btu/lb (11,400 kJ/kg) with a standard deviation of 1075, and therefore, a 21.9% coefficient of variation and a relative standard error of 400 Btu/lb (930 kJ/kg) or 8.16%. This related that 95% of the time the as-received heating value would be expected to range from 4500 to 5300 Btu/lb (10,500 to 12,300 kJ/kg) (13).

Yard waste can have a major influence on the fuel character of municipal waste. As discussed previously, yard waste can be a high weight percent of the total waste and because of its typically high moisture adds little, if any, calorific value. Therefore, any analysis must account for the influence of yard waste on the data.

Since the wide range in yard waste quantity has such a great impact on the overall composition of municipal waste, displaying the comparative data on a yard-waste-free basis does level the compositions, as displayed in Table 8.15. However, each of these communities had approximately 25% more paper waste and as little as 50% of the metals reported as the national average. Although there was no New Jersey bottle bill, there were aggressive materials recycling programs in Gloucester and Cape May counties. The glass and brick fractions seem high, although within the national average.

The food waste quantities appear to be uniformly low compared to the reported national average, and the very high paper fraction encountered during the spring 1980 sorting program at Central Wayne County, Michigan, seems to be suspect and warrants further study. The influence of the higher paper fraction on the heating value appears to be far less than the lower quantities of synthetic materials.

The close agreement of the moisture and ash-free heating values for each of the entities listed illustrates that moisture and ash are the major influences on the energy character of municipal waste.

This type of comparative analysis can assist planners, designers, managers, and facility operators to understand, prepare for, and manage their disposal and materials–energy resource recovery programs more effectively.

Curbside Weighing Program

This is a direct method of obtaining basic waste weight generation data for demographic correlations, such as residential density, habitat, socioeconomic level, and other influences of interest. This weighing program is best suited when the household trash is set out at the curb for pickup by the collection truck. The time of day must be coordinated with the truck schedule. The best time of year to conduct a program of this nature is the early fall, when school is back in session and prior to leaf fall. Generation rates at this time generally approximate the annual average. Obviously, such operations should not be conducted during severe weather or during the week following disruption of regularly scheduled collection due to a major holiday, severe weather, or other conditions (20).

During the curbside weighing period, the specific collection truckload involved should also be weighed and the households contributing to these loads cataloged. There should be a close correlation between the total of curbside weights and the net weight of the truckload. Nonresidential waste collection in the same truck should be avoided during this period. If this is not possible, the generators of these wastes should be identified by location and nature of business for later canvassing and cataloging. These data can give an approximation of nonresidential waste generation for similar businesses.

A curbside program can provide basic data on number and nature of items set out, total container weights for each household, and the number of persons contributing. These data could be related to population and household makeup for the entire collection area serviced.

Careful planning and preparation for the curbside program is required in the interest of economy and quality of data. Planning for the program is based on a preliminary estimate of the waste stream, route schedules, and maps of existing collection operations. Preparation of a route overlay to a planning and zoning map indicating individual properties can be useful.

Selection of weighing locations should be based on a defined random method to preclude introduction of bias by the team members. For the selected location(s), all trash containers set out along a street or alley in a predefined area should be weighed and notations made of unusual materials or containers.

A two-person team can cover about 10 households per hour. With proper planning, the number of teams required to cover an area in the available time frame can be established. It may be necessary to leapfrog the collection crew and return later to obtain tare weight for all reusable containers and the occupancy data. At

TABLE 8.15 Comparative Compositions of Municipal Solid Waste*, †, ‡

Waste constituent	Central Wayne County, Mich.		Gloucester County, N.J.		Cape May County, N.J.		Branford, Conn.		Average U.S. EPA, Weight %
	August 1979, Weight %	April 1980, Weight %	October 1981, Weight %	September 1982, Weight %	October 1982, Weight %	Average U.S. EPA, Weight %			
Newsprint	13.5 } 40.0 }	12.5 } 58.7 }	10.4 } 40.2 }	11.8 } 42.6 }	10.8 } 43.8 }	42	10.8 } 43.8 }	54.6	
Diapers	1.8 } 6.7 }	1.0 } 4.0 }	1.2 } 4.2 }	incl } 3.8 }	incl } 3.4 }	5.2	incl } 3.4 }	3.4	
Textiles/garments	5.2 } 3.7 }	4.0 } 2.5 }	4.0 } 2.8 }	4.0 } incl }	4.8 } incl }	4.5	4.8 } incl }	4.8	
Plastic, film	5.1	2.0	3.7	5.5	10.2		10.2	17.9	
Plastic, rigid	1.8	2.6	1.2	3.1	2.2		2.2	4.5	
Food waste	3.4	3.7	9.2	7.1	6.8		6.8	1.9	
Wood	5.1	4.9	4.3	3.9	5.1		5.1	9.8	
Sweepings	0.4 } 0.8 }	0.2 } 0.1 }	0.6 } 0.1 }	0.8 } incl }	0.8 } incl }	1.1	0.8 } incl }	5.9	
Ferrous	7.8	3.1	13.0	12.4	6.4		6.4		
Aluminum	4.7	0.7	5.1	5.0	5.7		5.7	12.7	
Nonferrous	100%	100%	100%	100%	100%		100%	100%	
Glass/brick	9720	9290	9530	9350	9250		9250	9460	
OBW	39.7	27.3	15.1	6.6	9.5		9.5	16.3	
Total									
Moisture and ash free basis									
Higher heating value; Btu/lb	9720	9290	9530	9350	9250		9250	9460	
Yard waste	39.7	27.3	15.1	6.6	9.5		9.5	16.3	
As-received basis including yard waste									
Moisture	42.1	29.3	31.5	25.4	28.0		28.0	27	
Ash	15.4	14.9	21.8	24.0	21.6		21.6	25	
Higher heating value, Btu/lb, AR	4120	5140	4280	4690	4630		4630	4500	
Higher heating value, Btu/lb, MAF	9695	9210	9165	9270	9190		9190	9400	

*From truckload sampling and yard-waste-free analyses.

†Large suburban community—Central Wayne County, Michigan; Small suburban community—Town of Branford, Connecticut; Rural county—Gloucester County, New Jersey; Resort County—Cape May County, New Jersey.

‡Data grouped to simplify comparison with national averages; numbers have been rounded for simplicity.

least 30 residences should be sampled for each area. A subarea may be defined by neighborhood, physical, social or economic criteria, or may correspond to daily collection zones.

The equipment required in the curbside method consists of

- The team, identifiable through hardhats and name tags
- A portable handheld scale with hook, 100-lb (45.5-kg) capacity
- A grappling harness with hooks to pick up containers and a plastic pan $2 \times 3 \times 6$ in ($0.6 \text{ m} \times 0.9 \text{ m} \times 15 \text{ cm}$) to handle assorted small items or those that cannot be hooked
- A 6-ft-high forked rod to support the scale for heavier loads
- Gloves, sturdy shoes, and clipboard

Team members should be courteous in explaining their activity when questioned by citizens. Inquiring residents, when informed of the team's purpose, will usually volunteer occupancy information. Neighbors will frequently provide the information for houses where information is difficult to obtain.

Data on the number of households served and that on intermingled commercial establishments can be recorded by a cooperative collection truck driver or a team member riding in the truck.

At the very least, this basic method will reveal the weight character of residential waste generation. These types of data are illustrated in Figure 8.8 for a one week curbside weighing program of a small suburban community in Michigan in September, 1981. The plot indicates that the spread of data for the households with many members is much less than that with few members; that the rate of waste generation per capita is prone to be less for the larger households; e.g., periodicals and grocery bags discarded are not directly proportional to the number of individuals. This display is contrary to the expedient assumption that the waste generation rate per capita is essentially the same regardless of size of household. This information could be useful for correlations with other demographic and planning data.

Characterization Program Perspective

The following items should be considered in the planning and conduct of a solid waste characterization program:

- Waste composition investigations conducted in a similar manner facilitate comparative analysis, even though undertaken at different times and circumstances. The comparative composition data assembled in Table 8.15 are intended to display the character of wastes on the same basis (yard-waste-free) using essentially the same sampling and sorting procedure for four types of communities and obtained within a relatively short time span. The United States average data are also provided for comparative purposes.
- The significant influence on the quantity and constituent mix of municipal waste is the type and makeup of the community: urban, industrial, rural, university, resort, and socioeconomic and age level, etc.
- The variation in per capita waste generation rate for a particular community is influenced in the near term by the weather and the season; over the longer term by the economy and changes in consumer packaging.
- The per capita waste generation rate is influenced by the number of persons in the household.
- Additional characterization studies may be desirable to better define the seasonal changes in waste generation rate and waste composition.
- The character of municipal waste will continue to be variable in spite of imposed government requirements for source separation of select waste materials.
- A characterization program can be time consuming and costly. Therefore, it should be well planned and the actual sampling and sorting activity closely monitored to assure that the data obtained are of the caliber desired. Every effort should be made to avoid external influences and bias. These might be introduced by weather, holidays, collection upsets, or by resorting to makeshift facilities for sorting and expedient procedures for personnel, all of which can cause bias.

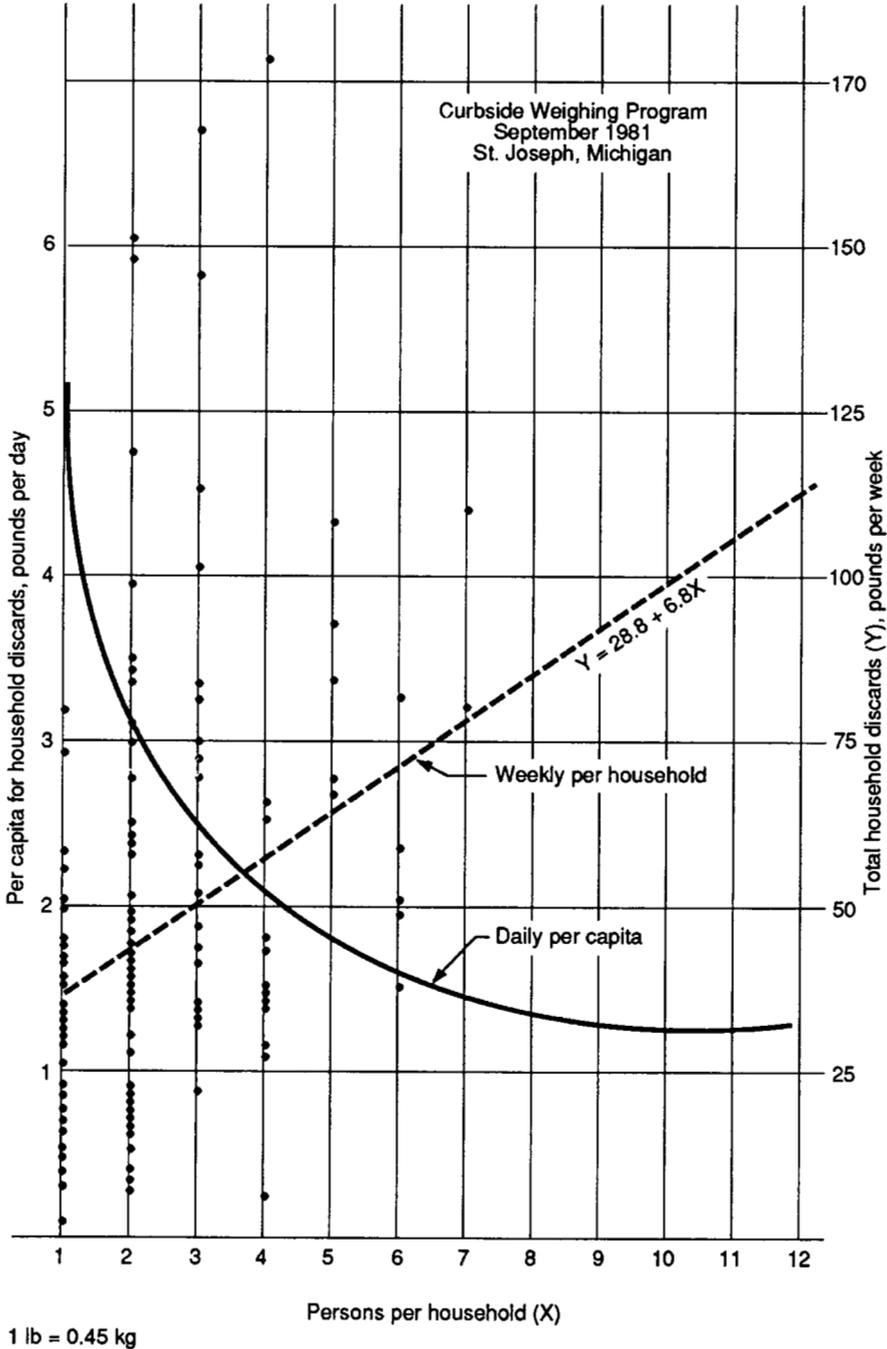


FIGURE 8.8 Residential waste generation (20).

- Once-per-week collection from residences should be arranged during the program to avoid the bias that may result from differences in midweek or weekend waste quantity and character.
- A justified materials weight balance should be made for each sample increment characterized, regardless of size. The weight of the entire sample before sorting should correspond with the sum of the weights of the constituents sorted.
- On an MAF basis, the HHV of municipal solid wastes generally varies through the relatively narrow range of 9000 to 10,000 Btu/lb (3870 to 4300 kJ/kg). However, as with all solid fuels, moisture and ash content are the major influences on the actual energy character of municipal waste. Since it would be convenient to have a “nominal” reference fuel analysis, the ASME Research Committee on Municipal and Industrial Waste is recommending the analyses shown in Table 8.16 as a basis of discussion. The physical makeup of the waste influences the material handling and combustion feeding and burning systems, as well as the practicality of materials resource recovery.
- A waste characterization program should be designed to provide the level of confidence required for the particular use of the data; for overall planning, general extrapolations of published data may be all that is necessary; for design of a new or retrofit facilities requiring contractual commitments and financing, definitive information on waste quantities and character may be required.
- The discrete sizes and size distribution of the constituent components and the concentration of the constituents within the total mass of waste will influence the size and number of sample increments necessary for high-confidence characterization. The larger the size of the components, the larger the sample increment required. With smaller particle sizes, e.g., less than tin (2.5 cm), and the greater its concentration, smaller samples and fewer sample increments would be required for the data to be representative. ASTM Standards D-2013, D-2234, and EDS-8 provide sampling principles and procedures applicable for small-particle-size materials. ASTM procedures should be used for sampling and analyses wherever possible since these should provide reproducible data.

TABLE 8.16 Municipal Solid Waste—Nominal Reference Fuel Analyses*

Ultimate analysis, %			Proximate, %	
Moisture	20–40	27	Moisture	27
Carbon	20–30	24	Volatile matter	41
Hydrogen	3–5	9	Fixed carbon	7
Oxygen	15–25	19	Ash [†]	25
Nitrogen	0.30–1.00	0.6		100%
Sulfur	0.05–0.20	0.1		
Chlorine	0.10–0.50	0.3		
Ash	20–30	25		
		100%		
Higher heating value				
As-received, Btu/lb, AR			3500–5500	4500
Moisture and ash free, Btu/lb, MAF			9100–9700	9400

*1985 ASME Research Committee on Municipal and Industrial Waste Boiler as Calorimeter Subcommittee.

[†]Ash for municipal solid waste is the noncombustible portion of the residues made up of mineral matter, metals, stones, glass, ceramics, etc.

Note: 1 Btu/lb = 0.43 kJ/kg.

COLLECTION AND TRANSFER OPERATIONS

The most costly element of refuse service is collection and transfer. Seventy-five to eighty percent of the solid waste budget is spent on collection and transfer costs. Reducing these costs can best be accomplished by understanding refuse service processes and equipment.

Collection is the part of solid waste management involving the collection route. Collection begins where the customer's waste is placed for pickup and ends when the collection vehicle leaves the collection route for the off-route haul to a disposal or processing site. A transfer station, i.e., a terminal where smaller collection vehicles empty their loads for continued haul by a larger vehicle, may be part of the collection process.

This section describes the institutional arrangements, collection practices, equipment, crew selection, routing, and transfer elements of solid waste management. The discussion incorporates aids to vehicle and route selection and transfer station economics.

Collection Practices

Solid waste collection is one of the most visible services provided to residents of a community. Whether the system is public or private, each citizen has contact with refuse collection. The high visibility of solid waste collection demands effective administration and efficient service mechanisms.

Service Arrangements. Solid waste collection and transfer can be undertaken through public, private, or mixed public-private service. Public arrangements place control in the hands of a governmental unit, while private arrangements place control under a private company or individual. Deciding whether public or private service is more appropriate within a particular collection area requires a perceptive understanding of the community and clearly articulated collection aims and priorities. Service recipient, service provider, service arranger, and service type are particularly important elements in choosing public or private service.

Public Arrangements. Public solid waste collection is conducted by municipalities that retain control of administration and/or operations. Generally, authority resides in a major department of the municipality. Public arrangements take the form of either municipal collection or contract collection. Larger municipalities may use both types of collection, using competition to control costs and improve productivity.

Under municipal collection, waste is collected by public employees using publicly owned equipment. The operations are conducted under the direct supervision of a municipal department such as the Department of Public Works. Normally, single-family residences within a community are covered by the service. Frequently, small industrial and commercial establishments, schools, hospitals, and other institutions, and small multifamily residential units are collected as well. The municipality retains full control of operations, maintenance, and fee collection.

Where contract collection is used, waste is collected by a private firm under contract to a municipality. The contractor owns the equipment, furnishes the employees, and manages operations. The public agency stipulates the service level, collection frequency, and other program elements such as container characteristics or hours of operation. The municipality retains responsibility for receiving complaints, billing customers, and controlling the activities of the private firm within the municipal jurisdiction. Generally, contracts are awarded through competitive bidding for three years or more to provide time to amortize equipment and to provide incentive to the private firm. Contracts usually provide for cost adjustments at least annually.

Private Arrangements. Private arrangements are used where collection services are not provided by a governmental unit. Primary solid waste collection responsibility and control rests with one or more private firms. The distinguishing feature of private arrangements is that collection firms are paid directly by the customer. The principal forms of private arrangement are franchise collection, private collection, and self-service.

Franchise collection consists of a governmental unit giving a private firm an exclusive license to serve a particular area, generally encompassing more than 10,000 people. Franchises are seldom awarded for less than three years and may or may not be awarded by competitive bidding. The municipality enforces the licensing arrangement and may receive customer complaints. Franchising minimally burdens a municipality but creates a monopoly with attendant potential for high prices due to lack of competition.

Private collection offers little public regulation of collection practice. Private firms do not have exclusive territories. The level of service is arranged between the customer and the collector, with customers permitted to change firms when they wish. Competition is often fierce and a single area may be served by several collection firms. The various firms seldom coordinate collection days or practices, so containers may be seen along streets several days a week. Private collection can be highly inefficient and very expensive. It is the least satisfactory and highest cost system of refuse collection.

Self-service is the practice of letting the generator take waste directly to a transfer station or disposal area. It is most often used in rural areas.

Public versus Private Arrangements. Each type of institutional arrangement offers specific advantages and disadvantages. Table 8.17 details the advantages, disadvantages, and conditions favoring particular institutional arrangements.

In a 1975 survey of over 2000 cities with more than 2500 people, Savas (23) found that about 1600 cities used either municipal, private, or self-service exclusively. Of these, 41.6% used municipal service and 57.8% used contract collection. Savas' data on service arrangements is presented in Table 8.18.

Larger cities are more likely to provide municipal service. Geographically, municipal service predominates in the South while northern and north-central cities favor private collection. Franchise collection is most commonly encountered in the West.

Savas also compares mean costs and offers the following comparison of annual cost per household using the lowest cost as unity:

Type of service arrangement	Relative cost
Municipal collection	1.15
Contract collection	1.00
Franchise collection	1.07
Private collection	1.61

Municipal service is slightly more costly than private service, but can offer somewhat higher public service (2). The reasons for higher municipal costs are, in part, due to budgeting and administrative procedures. The capacity for a municipality to mandate collection service is useful in controlling litter and maintaining public health.

Public arrangements should be encouraged because the municipality has control of refuse collection except for major commercial customers. Collection can be mandated and separate recycling can be more simply implemented. In public arrangements relying on private contractors, contract specifications must be general enough to attract bidders, but restrictive enough to discourage incompetent firms. Contracts should be for three years or more with performance bonds a requirement.

Collection System Administration. Administration is the managerial control of operation. Administrative concerns include record keeping, maintenance, standby equipment, and costs.

Record Keeping. Record keeping is a vital part of management in any solid waste collection or transfer system. Without records, productivity measurements, evaluations, cost studies, and preventive maintenance are difficult to perform. Records that should be routinely kept by the collection department are listed in Table 8.19.

TABLE 8.17 Advantages and Disadvantages of Public and Private Collection Services (22)

Alternative	Potential advantages	Potential disadvantages	Conditions that favor alternative
Municipal	<p>Tax-free</p> <p>Nonprofit</p> <p>Economies of scale</p> <p>Municipality has administrative control</p> <p>Can institute separate collection for recycling</p> <p>Can institute mandatory collection</p> <p>Management and policies continuous, resulting in experienced personnel and permitting long-range planning</p> <p>Records can be kept over a long time</p>	<p>Public</p> <p>Monopolistic</p> <p>Limited incentive to improve efficiency</p> <p>Financing and operations often influenced by political constraints</p> <p>Frequently financed from general tax fund and subject to 1-year budgeting process</p> <p>Solid waste management often low-priority item in budget</p> <p>Labor pressures may result in inefficient labor practices and strikes or inflated labor costs</p> <p>Restrictive budget policies may affect equipment replacement and maintenance</p>	<p>Past history of unsatisfactory contractual operations for public services</p> <p>Public predisposition toward government operation of public services</p> <p>Quality of service provided more important criterion than economics</p>
Contract	<p>Competitive bidding for contract(s) helps keep prices down</p> <p>Municipality retains administrative control</p> <p>Can institute separate collection for recycling</p> <p>Can institute mandatory collection</p>	<p>Danger of collusion in bidding</p> <p>Public agency must regulate contractors</p>	<p>Flexibility to make change in operations that would result in labor savings and other cost reductions</p> <p>Availability of qualified private contractors</p> <p>Public predisposed toward private sector involvement in public services</p> <p>Newly incorporated communities, or communities where population growth is outpacing ability of community to provide public services</p>

Municipal system and private firms under contract	Competition helps keep price down Alternative available if either sector cannot deliver service Municipality had administrative control can institute separate collection for recycling	Could be administratively or jurisdictionally complex	Municipality is expanding through annexation or merger with other jurisdictions Changing from separate garbage and trash collection to combined collection
Private arrangements Private collection	Competition may reduce costs Self-financing	No public administrative control Danger of collusion among haulers Cutthroat competition can result in business failures and service interruptions Overlapping routes, waste of fuel Cannot institute citywide separate collection for recycling Difficult to enforce mandatory collection ordinances	Municipality not interested in refuse collection
Franchise	Self-financing	No public administrative control Monopolistic, can lead to high prices Cannot institute separate collection for recycling Difficult to enforce mandatory collection ordinances	Municipality wants little to do with refuse collection

TABLE 8.18 Service Arrangements for the Collection of Mixed Residential Refuse, by City Size, Region, and Form of Government (23)

	Municipal		Contract		Franchise		Private		Self-service		Other	
	lb	%	lb	%	lb	%	lb	%	lb	%	lb	%
Total	2531	30.3	420	16.6	166	6.5	782	30.9	376	14.9	19	0.8
Population group												
>250,000	27	73.0	4	10.8	0	0	4	10.8	1	2.7	1	2.7
50,000-249,999	149	55.6	25	9.3	22	8.2	41	15.3	28	10.4	3	1.1
10,000-49,999	242	34.3	152	21.5	59	8.4	170	24.1	81	11.5	2	0.3
2,500-9,999	350	23.0	239	15.7	85	5.6	567	37.3	266	17.5	13	0.9
Geographic region												
2531												
Northeast	186	19.0	213	21.7	22	2.2	382	38.9	176	17.9	2	0.2
North central	143	20.0	111	15.5	16	2.2	330	46.2	107	15.0	8	1.1
South	469	72.7	28	6.0	34	7.2	33	7.0	27	5.8	6	1.3
West	366	26.8	69	18.9	93	25.4	37	10.1	66	18.0	3	0.8
Form of government												
1799												
Mayor-council	374	42.7	214	24.4	42	4.8	178	20.3	64	7.3	4	0.4
Council-manager	724	44.1	109	15.1	103	14.2	100	13.8	87	12.0	6	0.8
Other	199	27.1	32	16.1	5	2.5	60	30.2	45	22.6	3	1.5

Note: This table shows the distribution of arrangements, not the distribution of cities. There is a total of 2531 arrangements in the 2052 cities.

TABLE 8.19 Routine Collection Service Records

Route books and route maps (updated as required)
Vehicle and body records
Purchase data
Maintenance and repair record
Fuel consumption record
Accident records
Operating hours
On-route hours
Off-route hours
Time to disposal and return
Crew records
Weight or volume collected per day
Households collected per day
Other stops collected per day
Truck assignment
Time on route
Time off route
Load records
Weight
Number of trips to disposal per day
Percentage of full capacity
Number and type of units collected (periodic)

Maintenance. Refuse vehicles are complicated mechanical devices requiring periodic maintenance. A mechanic capable of repairing mechanical and hydraulic systems together with the necessary tools and equipment should be a part of any solid waste collection system. For small operations, such service may be provided by a private mechanic.

Standby Equipment. Refuse collection is hard service with frequent equipment breakdown. The prudent collection manager will provide sufficient standby equipment to allow the daily collection to be completed as scheduled. Generally, one standby vehicle should be available for every five vehicles used daily, but not less than one spare should be available.

Cost Factors. The cost of solid waste collection varies considerably from system to system. A study of 314 systems in the United States by Stevens (26) found wages to be the single largest component of cost. Direct labor cost averages 60 to 65% of the total collection cost. Other factors affecting municipal collection costs include stop density, service level, management practices, waste generation, and equipment size and type.

Crew Productivity. Crew productivity is a measure of efficiency. Several factors influence crew productivity including route, service level, collection equipment, and personal characteristics of crew members. Table 8.20 outlines factors affecting crew productivity.

Crew productivity may be measured in several ways. Among these are:

- Households collected per week per crew
- Weight per year per crew or crew member
- Volume per year per crew or crew member

Data on productivity, measured in several cities across the nation in the mid-1970s, was used to produce Table 8.21. The table provides maximum, mean, and minimum productivity for various crew and truck con-

TABLE 8.20 Factors Affecting Crew Productivity

Condition	Factors
Routing, service level	Routing design, density of stops, collection location, types of containers, traffic congestion
Vehicle	Loading location, loading height, vehicle capacity, compaction density, crew size, packing cycle
Personnel	Age, attitudes, health

figurations. The cost of providing service, although not as specific as the other approaches, is sometimes used as a measure of productivity. Table 8.22 compares costs for several crew and truck configurations.

Service Level

The level of service is set by the frequency of routine collection and the type of service offered to the customer. Collection frequency and scheduling are principally matters of health, aesthetics, and economics. Service type requires a decision on resident responsibilities versus crew productivity, specifically whether residential collection is to be in the backyard or at curbside.

Residential Collection. Residential service levels vary with each municipality, depending on citizen expectations and budgetary constraints. Once-weekly service is the predominant collection interval in the United States. Twice-weekly collection is normal throughout the southern and the southeastern United States, where warm climates create nuisances associated with refuse stored for longer periods. In inner-city areas where storage is limited, collections more frequently than twice-weekly may be needed.

The more frequently wastes are collected, the more costly is the collection service. Table 8.23 outlines the advantages and disadvantages of various frequencies of residential refuse collection.

Type of Service. Whether crews pick up waste at the curb or in the backyard is an important aspect of the level of solid waste collection. Crew and truck sizes and service cost depend on the type of service offered.

Curbside collection requires the resident to place waste containers at curbside or alleyside on collection day. Curbside service is less expensive than backyard service, and about 60% of the collection systems in the United States had curbside service in the mid-1970s.

Backyard collection requires the collection crew to pick up waste where the resident stores it. In some cases, crews enter fenced areas or garages to pick up containers. Backyard service is usually in one of four forms:

- *Set out and set back.* Crews carry containers to the curb and return empty containers to the storage place.
- *Set out.* Crews carry containers from the storage place to the curb; residents return empty containers to the storage area.
- *Tote barrel.* Crews empty containers at the storage place into a tote barrel, then empty the tote barrel into the collection vehicle.
- *Satellite vehicle.* A crew member drives a small vehicle to the storage area, empties containers into the vehicle container, then empties the smaller vehicle into the main collection truck.

Curbside versus Backyard Service. The economics of fuel consumption and service time are particularly important in comparing curbside and backyard service. Table 8.24 summarizes advantages and disadvantages of curbside and backyard service.

TABLE 8.21 Comparative Productivity Factors—Solid Waste Collection Crews (2)

	Crew size, persons	Collecting time, h/day	Stops per crew-day	Volume, m ³ (yd ³) per crew-year	Weight, metric tons (tons) per crew-year	Volume, m ³ (yd ³) per person-year	Weight, metric tons (tons) per person-year
Total of all systems							
Average	2.5	4.52	552	7,042 (9,210)	2,313 (2,550)	3,310 (4,330)	1,075 (1,185)
Maximum	4	6.8	1,229	12,891 (16,860)	5,098 (5,620)	7,355 (9,620)	2,123 (2,340)
Minimum	1	3.0	182	2,783 (3,640)	825 (910)	1,319 (1,725)	340 (430)
All municipal systems							
Average	2.9	4.3	605	6,380 (8,345)	2,645 (2,265)	2,347 (3,070)	780 (860)
Maximum	4	5.6	1,155	8,732 (11,420)	3,311 (3,650)	3,991 (5,220)	1,184 (1,305)
Minimum	2	3.0	280	5,275 (6,900)	1,306 (1,440)	1,319 (1,725)	340 (430)
All municipal rear loaders							
Average	3.2	4.0	660	6,407 (8,380)	2,059 (2,270)	2,026 (2,650)	658 (725)
Maximum	4	5.6	1,155	8,732 (11,420)	3,311 (3,650)	2,359 (3,085)	1,102 (1,215)
Minimum	3	3.0	455	5,275 (6,900)	1,306 (1,440)	1,319 (1,725)	340 (430)
Municipal side loaders							
Average	1.5	4.5	599	7,103 (9,290)	2,109 (2,325)	3,991 (5,220)	1,184 (1,305)
Maximum	2	4.5	—	—	—	—	—
Minimum	1	4.4	—	—	—	—	—

(continues)

TABLE 8.21 Comparative Productivity Factors—Solid Waste Collection Crews (2) (continued)

	Crew size, persons	Collecting time, h/day	Stops per crew-day	Volume, m ³ (yd ³) per crew-year	Weight, metric tons (tons) per crew-year	Volume, m ³ (yd ³) per person-year	Weight, metric tons (tons) per person-year
Municipal front loaders	2	5.4	329	5,528 (7,230)	1,987 (2,190)	2,764 (3,615)	933 (1,095)
Municipal backyard	4	3.0	279	5,275 (6,900)	1,565 (1,725)	1,319 (1,725)	390 (430)
Municipal curbside							
Average	2.7	4.5	660	6,568 (8,590)	2,136 (2,355)	2,519 (3,295)	844 (930)
Maximum	3	5.6	1,155	8,732 (11,420)	3,311 (3,650)	3,991 (5,220)	1,184 (1,305)
Minimum	2	3.0	330	5,497 (7,190)	1,306 (1,440)	1,831 (2,395)	435 (480)
Municipal three-person crews							
Average	3.0	4.2	615	6,691 (8,751)	2,182 (2,405)	2,095 (2,740)	722 (796)
Maximum	3	5.6	1,155	8,732 (11,420)	3,311 (3,650)	2,360 (3,086)	1,102 (1,215)
Minimum	3	3.0	280	5,490 (7,180)	1,306 (1,440)	1,835 (2,400)	435 (480)
Municipal two-person crews							
Average	2.0	5.0	464	6,316 (8,260)	2,045 (2,255)	3,375 (4,415)	1,089 (1,200)
Maximum	2	5.4	600	7,103 (9,290)	2,109 (2,325)	3,991 (5,220)	1,184 (1,305)
Minimum	2	4.5	330	5,528 (7,230)	1,987 (2,190)	2,764 (3,615)	993 (1,095)

TABLE 8.21 Comparative Productivity Factors—Solid Waste Collection Crews (2) (continued)

	Crew size, persons	Collecting time, h/day	Stops per crew-day	Volume, m ³ (yd ³) per crew-year	Weight, metric tons (tons) per crew-year	Volume, m ³ (yd ³) per person-year	Weight, metric tons (tons) per person-year
Private curbside							
Average	2.2	5.2	641	8,965 (11,725)	3,075 (3,390)	4,916 (6,430)	1,610 (1,775)
Maximum	3	6.8	1,300	12,891 (16,860)	5,098 (5,620)	7,355 (9,620)	2,123 (2,340)
Minimum	1	4.0	290	5,371 (7,025)	1,592 (1,755)	2,852 (3,730)	1,093 (1,205)
Private three-person crews							
Average	3	4.4	624	7,440 (9,730)	2,608 (2,875)	2,691 (3,520)	930 (1,025)
Maximum	3	4.8	624	8,556 (11,190)	3,415 (3,765)	2,852 (3,730)	1,139 (1,255)
Minimum	3	4.0	624	6,327 (8,275)	1,800 (1,958)	2,531 (3,310)	721 (795)
Private two-person crews							
Average	2	4.0	652	10,651 (13,930)	3,152 (3,475)	5,328 (6,965)	1,574 (1,735)
Private one-person crews							
Average	1	4.3	260	5,069 (6,630)	1,474 (1,625)	5,069 (6,630)	1,474 (1,625)
Maximum	1	4.7	338	7,355 (9,620)	2,123 (2,340)	7,355 (9,620)	2,123 (2,340)
Minimum	1	3.8	180	2,783 (3,640)	825 (910)	2,783 (3,640)	825 (910)

TABLE 8.22 Average Cost Relationships for Various Collection Practices (2)

	Average number stops/year	Cost		
		\$/household/year	\$/metric ton (ton)	\$/m ³ (yd ³)*
All systems	113,100	39.36	31.45	7.82 (5.98)
All rear-load systems	121,100	40.83	32.71	9.02 (6.90)
Rear-load curbside systems	138,000	40.19	28.63	7.72 (5.90)
Crew of 2, 1-side collection, twice/week	141,300	24.17	20.78	6.02 (4.60)
Crew of 3, 1-side collection, once/week	26,500	50.41	34.11	8.46 (6.47)
Rear load—backyard service	78,900	42.43	30.94	11.84 (9.05)
Crew of 2, once/week collection	105,000	63.39	29.26	6.21 (4.75)
All side-load systems (all curbside service)	100,200	30.74	22.95	4.92 (3.76)
Crew of 1	119,400	54.79	48.17	9.94 (7.60)
Once/week collection	86,300	26.32	19.07	4.17 (3.19)
Twice/week collection	123,700	42.27	39.43	8.12 (6.21)

*Volume calculated on nominal body capacity times trips to the disposal site, assuming vehicle is emptied at end of each working day.
 Note: Costs, U.S. dollars (C.P.I., \$271.7).

TABLE 8.23 Advantages and Disadvantages of Different Frequencies of Collection (22)

Alternative	Potential advantages	Potential disadvantages	Conditions that favor alternative
Once per week or less	Less expensive Requires less fuel	Improperly stored waste can create odor and vector problems	Adequate storage provisions Cold to moderate climate
Twice per week	Reduces litter Reduces storage requirements	More expensive Requires more fuel	Quality of service provided more important criterion than economics Warm climate
More than twice per week	Reduces litter Reduces storage requirements	More expensive Requires more fuel	Seriously restricted storage space Dense population

Curbside collection can service more homes and collect more waste per day with fewer trucks than can backyard collection. Backyard collection with a three-person crew will require about 1.6 times more fuel for a set number of customers than will curbside service.

Fuel consumption on packer vehicles is as much a function of engine operating hours as it is of distance driven. A comparative breakdown showing percent of time for various activities of a collection vehicle appears in Table 8.25.

Backyard service consumes less fuel because the vehicle idles more and has fewer compaction cycles per operating hour than is required for curbside service. Curbside service consumes slightly more fuel per mile because trucks spend a greater portion of their operating hour in driving and compacting. Fuel consumption for collection vehicles is outlined in Table 8.26.

TABLE 8.24 Advantages and Disadvantages of Curbside–Alley and Backyard Collection (22)

Alternative	Potential advantages	Potential disadvantages	Conditions that favor alternative
Curbside	More efficient Less expensive Requires less labor Facilitates use of paper or plastic bags Reduces collector injuries Requires less fuel	Cans at curb look messy Special arrangements must be made for handicapped and elderly Residents must remember day of collection	High collection costs Unwillingness on part of residents to pay higher taxes or user charge
Backyard	No effort required by residents No mess at curbs	More expensive High labor turnover Increases number of collector injuries Requires more fuel	Quality of service provided more important criterion than economics

TABLE 8.25 Comparative Operating Activities for Residential Collection Vehicles in Percent

Activity	Type of service	
	Curbside	Backyard
Off-route travel and dumping	30	21
On-route driving	21	13
On-route idling	43	61
Compaction	6	5

TABLE 8.26 Typical Fuel Consumption by Collection Type (22)

Type of service and engine	Fuel consumption	
	gal/h	(L/h)
Curbside		
Diesel	2.4	9.0
Gasoline	2.8	10.5
Backyard		
Diesel	1.4	5.5
Gasoline	1.7	6.5

Time Study. The time needed to service a collection stop is a combination of the time required to set out the containers and the time to empty the containers into the truck. Curbside collection crews apply a significant amount of work time to collection that would otherwise be used for set out. A time-and-motion study was conducted in Ann Arbor, Michigan, on three-person setout crews in 1975. Backyard collection averaged 1.10 mm per household, whereas curbside collection averaged 0.64 mm per household (7).

The time needed to collect the refuse after it has been set out is a function of the number and type of containers at the stop, and not distinctly related to the weight of the containers. Figure 8.9 is a graph comparing the time of collection with container characteristics (7).

Collection Schedules and Costs. Almost all residential collection systems operate on a four-, five-, or six-day work schedule. In communities using a four-day schedule, crews are often detailed for special pickup on the fifth workday of the week. Twice-weekly collection systems must cover the collection area in 2 or in 2½ days. If a two-day schedule is selected, crews are often used for special pickup on the remaining work day.

The type of service can vary in cost according to crew size and frequency of collection. The cost comparison in Table 8.27 is based on data from nine U.S. cities (2). The data compare the cost between once-weekly and twice-weekly collection using either one-, two-, or three-person crews. Data must be used judiciously because of the small sample size.

Commercial Collection. Commercial collection is primarily a specialty service with pickup frequency dependent on waste volumes and types. Putrescible wastes are collected more frequently than nonputrescible

8.50

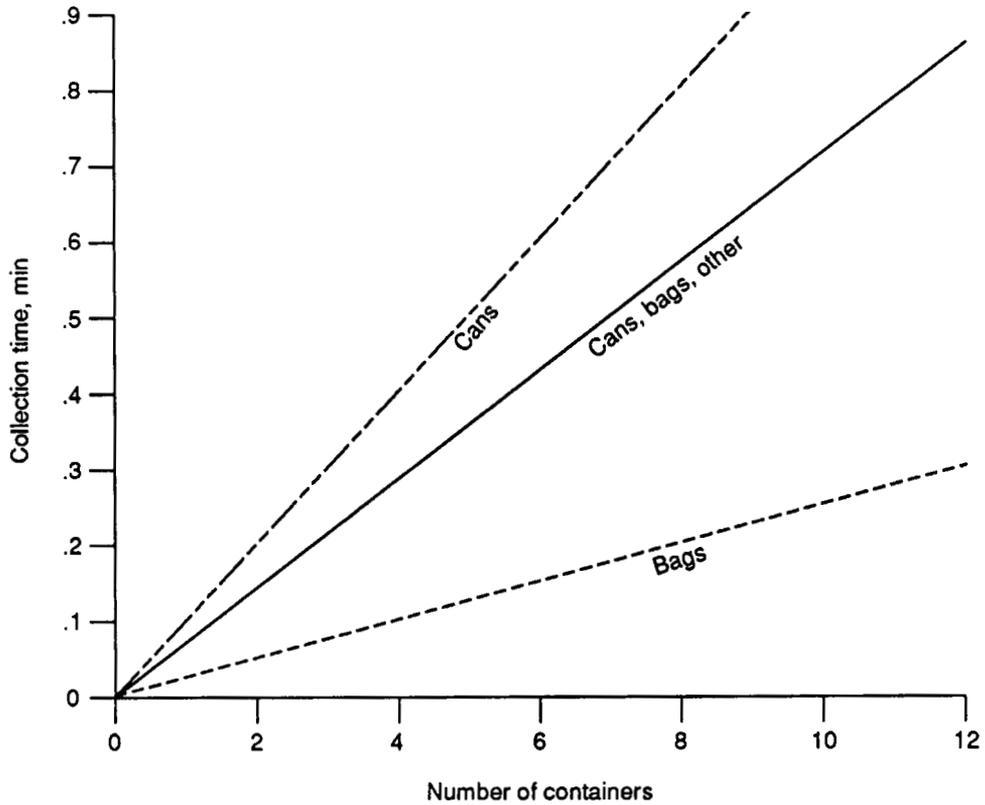


FIGURE 8.9 Collection time for refuse cans, refuse bags, and other containers.

TABLE 8.27 Comparative Cost of Curbside Collection by Frequency and Crew Size

Crew size	Frequency per week	Cost ratio
1	1	1.00
1	2	1.62
2	1	1.13
2	2	2.76
3	1	1.86
3	2	1.84

wastes. The generator and collector usually arrange service frequency at a specific point on the premises. Parks, hospitals, and other institutions may require daily if not more frequent collection.

Storage Containers

The collection agency should regulate storage container size and type, matching the container characteristics to the collection system. A suitable storage container must be easy to handle and to keep clean. The containers should contain odors, limit disease, and keep out animals and insects. Containers should be large enough to limit the number of containers at a stop, but small enough to be lifted easily and safely by one person or be appropriate to a mechanized system. In general, storage bins, 55-gal (208-L) drums, and cardboard boxes are unacceptable for storage.

Containers for Manual Collection. Two types of containers are acceptable for manual residential collection: metal or plastic cans and bags. Cans should be sized from 20 to 32 gal (75 to 120 L); bags may be paper or plastic. Table 8.28 details the advantages and disadvantages of common storage containers.

Containers for Mechanized Collection. In efforts to increase productivity, several communities are using mechanized containers for residential refuse collection. Residential mechanized collection is a new application of technology requiring more experience before it can be generally adopted. Currently, large containers for use at single- or multifamily residences are the most common mechanized storage containers. Containers for single-family residences are commonly 80-gal (300-L) heavy plastic mounted on wheels to facilitate movement to the curb. Collection vehicles are equipped to pick up the container and empty it into the vehicle. Some of these systems use special refuse bodies. Because reliability and public acceptance have not been generally established for such systems, careful study of the community is needed before such a system is selected.

Container systems to serve from two to four single-family households have been tried recently. The containers are similar to commercial containers and are handled the same way. Their principal advantage is that several homes are serviced by a single stop of the collection vehicle. Three disadvantages can be associated with such containers: (1) carelessness can lead to littering around the container; (2) reluctance to use containers that allow neighbors to observe waste; and (3) objections to having containers on private property.

Unacceptable Containers. Although residents may want to use almost anything as a refuse container, several containers, such as bins, large drums, and cardboard boxes, are unacceptable.

Some older apartment and commercial buildings have stationary concrete or cement-block storage bins. The bins are unsanitary and inefficient, must be emptied by hand, attract insects and rodents, allow blowing papers and odors, and if uncovered, can leach a foul liquid after rains.

Steel Drums and paper packing drums are also unacceptable. The drums usually do not have tight-fitting lids, allowing insects, birds, and animals access to the refuse. An empty metal drum weighs 35 to 40 lb (16 to 18 kg), and over 100 lb (45 kg) when full. Full drums can be unmanageable for one collector and unsafe due to sharp edges.

Cardboard boxes are sometimes set out on the curb for collection. Boxes filled with garbage or liquids attract pests and are a hazard to the collector. Cardboard boxes nested and open for inspection or cardboard boxes filled with newspapers may be acceptable for collection.

Collection Equipment

The system manager should choose equipment suited to the characteristics of the collection area. Anticipated service level, crew size, and route characteristics such as narrow alleys, restricted headroom, or turning radius will affect vehicle and body selection. Often more than one type or size of equipment will be needed

TABLE 8.28 Advantages and Disadvantages of Residential Waste Storage Containers (22)

Alternative	Potential advantages	Potential disadvantages	Conditions that favor alternative
Paper or plastic bags	<ul style="list-style-type: none"> Lessens lift weight Reduces spillage and blowing litter Eliminates empty cans at curb Eliminates odors and necessity to clean dirty cans Limits vector attraction Increases speed and efficiency of collection Reduces contact of collector with waste 	<ul style="list-style-type: none"> Cost per bag Bags break when too full Attract animals Not suitable for bulky, heavy, or sharp objects 	Curbside collection
Metal or plastic cans [20–32 gal (75–120 L)]	<ul style="list-style-type: none"> Reasonable size for collector to lift Economical and reusable 	<ul style="list-style-type: none"> Must be cleaned regularly when not used with liners Lids may be lost or fall to fit after use Cans may be left at curb for unreasonable periods 	<ul style="list-style-type: none"> Backyard collection Animal populations that open bags
Bulk containers for mechanized collection Drums [55 gal (208 L)]	<ul style="list-style-type: none"> More efficient than manual collection None 	<ul style="list-style-type: none"> Residents oppose storage of other people's waste on their property Lower collection efficiency Excessive weight can result in back injury and muscle strain Difficult to handle Lack of lids allows insects to breed in waste and odors to escape Rust holes at bottom of drum allow rodents to feed on waste 	<ul style="list-style-type: none"> Space for container Unacceptable alternative
Stationary storage bins	None	<ul style="list-style-type: none"> Inefficient—must be emptied manually Lack of proper cover leads to insect and rodent infestation Necessity for hand shoveling of wastes poses health hazard to collectors 	Unacceptable alternative

in a service area. Table 8.29 outlines equipment and crew characteristics and the implications for equipment operations. The average life of a collection vehicle body is from 5 to 7 years.

Residential Collection Vehicles. Residential collection vehicles have a chassis and a body. The chassis contains the drive train and operator's cab. The body includes the receiving hopper, compactor, and storage compartment. The chassis and body must be able to negotiate turns and pass under bridges along the route.

Collection equipment for residential services is classified by loading characteristics—rear-loaded or side-loaded. Consideration of loading hopper height and location is important in determining crew size and estimating productivity.

Compaction density, the measure of how much the body can compact the loose refuse picked up by the crew, is a significant consideration in loader productivity. Compaction density is measured in pounds per cubic yard (kilograms per cubic meter). Figure 8.10 illustrates the relationship between packer capacity, compacted density, and households collected.

Rear Loaders. Rear loaders are suited to densely populated areas, where stops are frequent and waste volumes are high. Both sides of the street or alley are usually collected at the same time. Where three-person crews are used, the driver helps with occasional heavy loads. On two-person crews, the driver usually doubles as a loader.

A number of manufacturers make rear-loading collection equipment (24). Units are available in 14 body sizes, ranging from 9 to 32 yd³ (6.9 to 24.5 m³) capacity (exclusive of hopper).

Compaction densities among 29 models of packers sampled range from 500 to 1100 lb/yd³ (225 to 500

TABLE 8.29 Factors to be Considered in Selecting Solid Waste Collection Equipment

Truck or crew factor	Implication
Route considerations	
Vehicle length	Garage space, turns
Vehicle height	Underpass and garage door clearance
Vehicle weight	Bridge and road weight limits
Vehicle turning radius	Turns to streets, alleys, or cul-de-sacs
Body volume	Number of collection stops and trips to disposal area
Compaction	Number of collection stops, amount of waste per stop
Number of axles	Load limits
Crew considerations	
Equipment access	Crew dismounting, mounting
Hopper height	Crew loading effort
Hopper width	Crew size and minimizing compaction cycles
Safety features	Crew safety
Cost considerations	
Vehicle fuel economy	Energy costs
Capital costs	Replacement and amortization costs
Operating and maintenance costs	Operating costs
Labor wage rates	Labor costs
Equipment reliability	Downtime
Service	Downtime
Ease of repair	Downtime

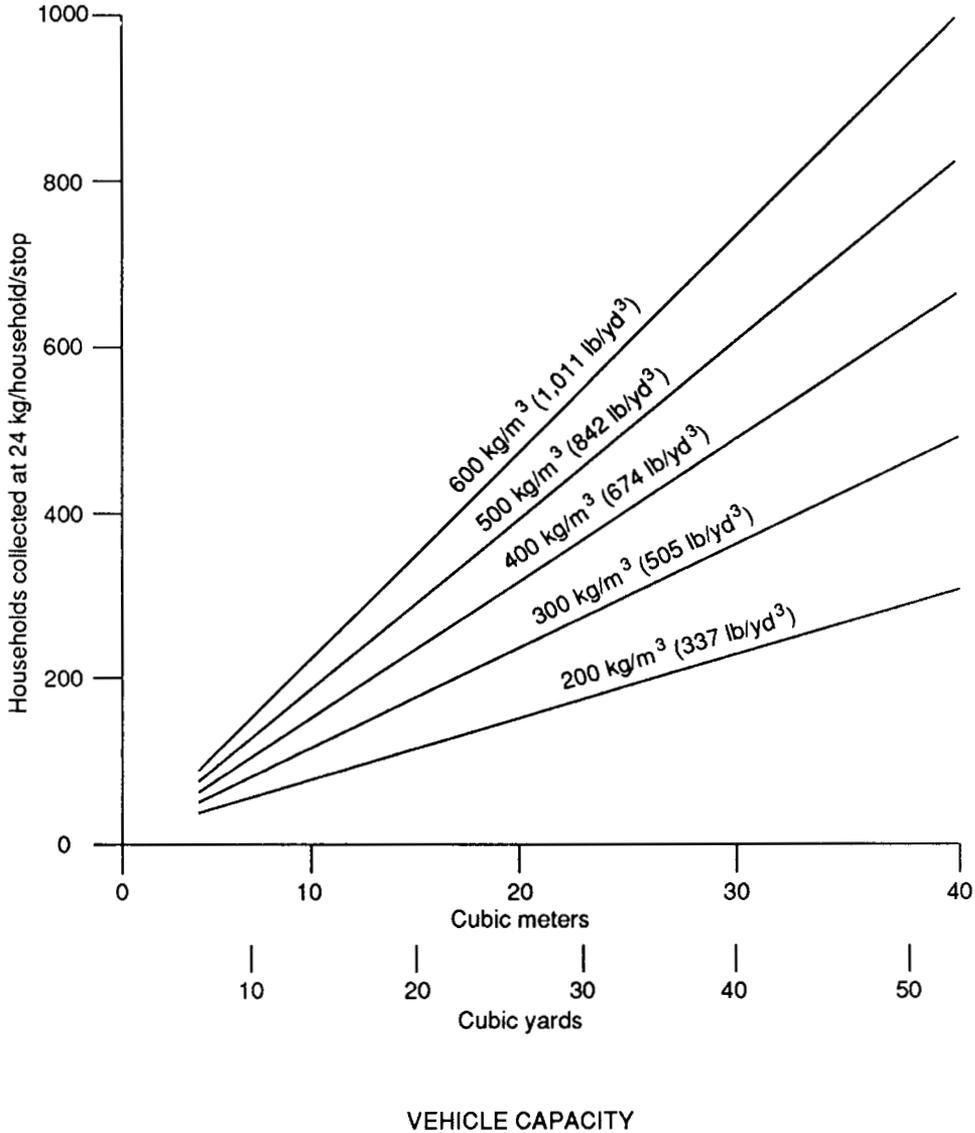


FIGURE 8.10 Number of households served by vehicle size and compacted density.

kg/m³). Compaction density on three-quarters of the rear loaders is less than 750 lb/yd³ (445 kg/m³). The average on-route efficiency that can be expected of rear-load equipment is between 80 and 85% of the manufacturer's rated maximum density.

Rear-load bodies have several advantages. The hopper height is lower than for most side-load bodies, so the crew does not have to lift containers as high. Loading hoppers are wide, usually the full width of the body, and the packer plate configuration allows large, bulky items to be accepted. The principal disadvan-

tage of rear loaders is the complexity of the packer plate design and the accompanying higher maintenance requirements.

Rear-load vehicles are emptied by gravity or by ejector. Gravity bodies are tilted so that the load slides out in a manner similar to dump trucks. If a load hangs up and cannot be shaken out, the truck must be emptied by hand. The trend has been toward ejector bodies, because dump body packers are slower to empty and more prone to overturning on the uneven surfaces of a landfill.

Side Loaders. Side loaders are best suited to densely populated areas where collection takes place on one side of the street or for rural routes. Side loaders are most often employed for residential curbside pick up, for apartments, or for small commercial establishments. Side-load vehicles are particularly useful in narrow alleys with limited maneuvering room if large containers are used. A low step-in model cab with right-hand drive can be run by a one-person crew. If two-person crews are used, two sides of the street can be collected at the same time, but the left-hand loader is exposed to traffic.

Thirteen side loader manufacturers offer 28 different body capacities ranging from 6 to 40 yd³ (4.6 to 30.6 m³). Compaction density ranges from 300 to 800 lb/yd³ (178 to 475 kg/m³), generally lower than the compaction capability of rear-load units (24). The compaction mechanism on side loaders is much simpler. The average on-route density will range from 85 to 95% of the manufacturer's statement of maximum density.

Side loaders are emptied by ejection, using the same mechanism that loads the storage compartment and compacts the load.

Specialized Collection Vehicles. Special collection vehicles are usually chosen to eliminate a cumbersome or costly collection situation or to improve productivity. Many of the vehicles are mechanized or automated, requiring fewer crew members. European cities have used specialized equipment that lifts and empties residential containers from the rear of the packer vehicle for many years. Other systems using mechanical arms or mechanical lifting devices are being tried in the United States.

Commercial Collection Vehicles. Solid waste is collected from commercial establishments as well as residences. Commercial establishments include businesses, industries, institutions, and often apartments with more than four to six units. Commercial wastes differ in quantity and type from residential wastes and are usually stored in large containers rather than small bags and cans.

Vehicle Types. Four types of vehicles are normally used in commercial collection systems: rear, side, and front loaders and specialized drop-off bodies. The least complex is a residential-type rear- or side-loading collection vehicle equipped to handle bulk containers. Bulk containers ranging in capacity from 1.0 to 8 yd³ (0.76 to 6.1 m³) are normally used with rear-loading equipment. Side-loading vehicles can accommodate bulk containers up to 6 yd³ (4.5 m³) capacity. The containers are equipped with casters to aid in maneuvering the container.

Front-loading equipment is operated by one person and is designed for use with commercial bulk containers ranging in capacity from 1 to 10 yd (0.76 to 7.6 m³). The containers are not equipped with casters, since the truck maneuvers to the container. The container is picked up by hydraulic arms, lifted over the truck cab, and discharged into a hopper in the top of the compaction body.

Front-loading vehicles are furnished by 11 manufacturers in 41 models ranging in capacity from 9 yd³ (6.88 m³) to 50 yd³ (38.2 m³). The density that can be obtained by the compaction body ranges from 400 lb/yd³ (237 kg/m³) to 850 lb/yd³ (504 kg/m³) (24).

A number of specialized body designs with drop-off boxes are on the market for commercial collection. The box is picked up, transported to a disposal or processing location, emptied, and returned to the customer's premises on a specially designed truck. Two forms are prevalent. The dumpster type provides an open or closed bin that is lifted onto the truck by arms attached near the rear of the chassis. The other offers a tilting frame chassis that moves bins on and off the truck by chain or cable. The latter form is popularly called a roll-off system.

Specialized systems usually become feasible when at least four containers per day per chassis are ser-

vided. The large containers offer storage for bulky items, as well as large volumes of material. Bin containers are available from 10 yd³ (7.65 m³) capacity to 50 yd³ (38.2 m³).

Roll-off bodies are available from 10 to 55 yd³ (7.65 to 42 m³) and may be enclosed or open. Special roll-off containers may be obtained for leaves, sludge, snow, liquid, and bulk debris. Enclosed containers can be equipped with push plates for ejecting loads.

Table 8.30 indicates the relative costs of commercial collection systems.

Maneuvering Requirements. The container must be located so that the vehicle can be driven up to it and have adequate room to lift it into the discharge position. Figure 8.11 illustrates the minimum maneuvering room required by various collection vehicles.

Routing

Routing is the process of identifying the path a vehicle is to take in serving its daily route. After administrative policy on service level, operations, and labor are clearly established, routing objectives can be set and the basic routing steps can be undertaken. Ideally, each collection vehicle should start the day empty and be packed out when the truck makes its final run to the disposal site. The path can be set by trial-and-error, computer, or heuristic methods. The steps involved in establishing a collection route are

- Define the collection area
- Assign disposal sites
- Establish daily zones
- Balance daily vehicle assignments
- Route vehicles within daily district

Collection Area. The collection area is comprised of the territory to be serviced in a single collection cycle. The limits of the collection area are usually defined by political or geographical boundaries. The limits of the collection area, together with disposal sites, transfer stations, or waste processing plants, should be marked on a map. The collection area map should show major routes to each disposal option and any restrictions, such as load or height limits.

Disposal Area. Disposal areas are those parts of the collection area assigned to a specific disposal or processing site. Where a system has only one or two disposal options, matching routes to disposal sites is relatively easy. Where several disposal sites are available, the problem becomes highly complex. Many factors must be evaluated in choosing where to route vehicles. Among these are tipping fees, site reliability and life, round-trip haul and disposal time, queuing costs, and vehicle wear and tear. In very complicated situations, a

TABLE 8.30 Relative Cost of Commercial Collection Systems

System type (crew of 1)	Cost		
	Capital	Annual	Dollars per ton
Rearload	1.00	1.00	1.00
Side load	0.70	0.040	0.98
Front load	1.67	0.99	0.52
Roll-off	1.33	0.90	0.24

Note: 1 ton = 0.9 metric tons.

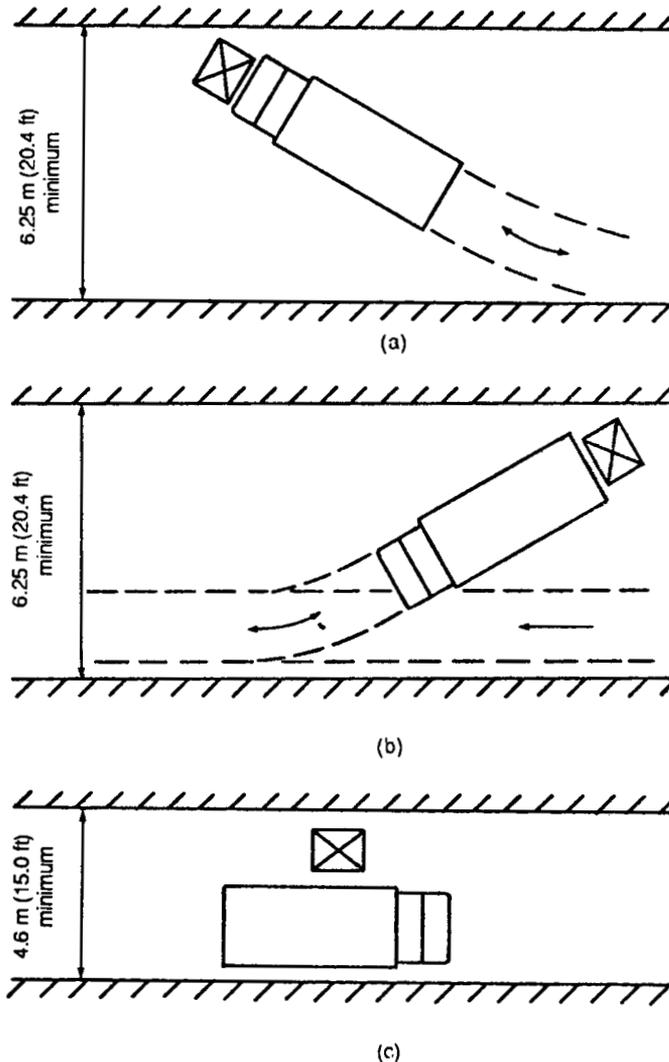


FIGURE 8.11 Maneuvering room for various types of commercial collection vehicles with 6-yd³ (4.5-m³) container (a) Front-load vehicle; (b) rear-load vehicle; (c) side-load vehicle.

computer program might be used to identify the most effective use of many different facilities. Suitable programs are available through the EPA, major universities, and private consultants.

Collection Zones. Collection zones are set up by dividing the collection area into sections or zones for daily service. All premises collected on Monday constitute one zone; those collected on Tuesday, another zone; and so on.

The average number of households assigned to each residential zone should be approximately equal. Daily zones can be slightly unbalanced to accommodate distinct geographical boundaries. When collection as-

signments are unbalanced, the heavier workload should be scheduled early in the collection cycle. Zones should be shown on a map kept at the central office.

Establishing collection zones for commercial and industrial districts is complicated by variations in the frequency of collection. The zones should be established based on compacted volume to be collected. A rule of thumb is that one compacted volume unit equals four loose volume units of residential or commercial waste, or 2.5 volume units of loose industrial waste. The following formula is useful in estimating compacted cubic meters per zone.

$$S_z = (S_1 + S_2 + \cdots + S_d)/D_c$$

where S_z = compacted volume per zone

S_1 = compacted volume at one-day collection frequency

S_2 = compacted volume at two-day collection frequency

S_d = compacted volume at collection frequency equal to the days per cycle

D_c = working days per collection cycle

Daily Vehicle Assignment. Each zone must be divided into an optimum daily workload for each collection vehicle and crew. Dividing the zone into sections reflecting the number of households to be serviced by each vehicle is called districting. Each district should be compact, consisting of streets clustered in the same geographical area. Districts should cover contiguous blocks and not be fragmented throughout the zone. Districting permits the manager to estimate the number and size of the trucks needed to collect waste, evaluate crew performance, and balance or equalize workloads.

Major considerations in balancing and districting are the productivity of the crew and the on-route time. Increases in either will lower costs.

On-route time is productive time and should be maximized to allow crews to collect as many stops as possible in a working day. In general, the major variable is the time spent in travel to and from the disposal site. The round-trip haul time can be kept low by selecting the proper number of appropriately sized vehicles for each zone. As much as 20% of the working day can be lost in travel to and from the route, delays, breaks, and personnel needs. The following formula will assist in making an estimate of on-route time.

$$T_r = 8 - [T_t + n(T_d + T_h) + T_b + T_l]$$

where T_r = time on route

T_t = time and travel between garage and route and return at day's end

T_d = time at disposal site including check-in and check-out, queuing, and emptying time

T_h = Round-trip haul time from route to disposal site gate and return

T_b = break time—coffee and lunch

T_l = time lost for delays, including personnel needs

n = number of trips to disposal site per day (must be a whole number)

The formula assumes an 8-hr work day; n can be estimated as follows:

$$n = 1.25H_r \times W_h/C_v \times C_d$$

where H_r = households per route

W_h = average weight of waste collected per household, lb (kg)

C_v = capacity of vehicle, yd³ (m³)

C_d = rated compacted density on vehicle, lb/yd³ (kg/m³)

1.25 = a constant reflecting efficiency and allowance for seasonal variation from average

n must be rounded up to the nearest whole number.

Service Stops. The number of service stops each vehicle makes per day may be estimated based on data provided in Table 8.21. Alternatively the number of loads, including fractional loads taken to the disposal site, may be multiplied by the number of services per load (N).

$$N = a(C_v \times C_d)/W_h$$

where N = the number of services per load

C_v = capacity of vehicle, yd^3 (m^3)

C_d = compacted density, lb/yd^3 (kg/m^3)

a = a constant accounting for efficiency, normally less than 1

W_h = weight per household or stop, lb (kg)

The number of trucks required to collect a zone may be calculated by dividing the number of households per zone by the number of services all working vehicles can collect per day. Each zone should have the same number of routes to maximize labor utilization. Differences in route lengths can be made to allow for variations in zone sizes.

District Routes. The route is the path the vehicle takes to make the collections within its district. The objective of routing is to direct the collection vehicle through the district so that wasted time is kept to a minimum. Routing does not include the path from the garage to the district or from the district to the disposal point.

Maps should be prepared showing the number and type (residential, apartment, commercial, institutional, industrial) of services per street segment. One-way, dead-end, and busy streets, and corner-lot residences should be marked. Each street segment should show truck direction by arrow and whether crews are to collect one or both sides of the street on a pass. A daily route map or route book should be kept on each vehicle.

A route can be set by trial and error, by computer, or by heuristic methods. Trial and error can take considerable time to arrive at an effective route, with inefficiencies persisting.

Computer routing programs can precisely optimize the route. Preparing for the computer program entails modeling stop locations, street pattern, one-way streets, and other details of the district. Collecting the data can take considerable time and it must be continually kept up to date. Computer routing has the advantages of speed and precision of analysis and ease of checking the effect of potential route changes.

Heuristic Routing. Heuristic routing is a compromise between the trial-and-error approach and the computer approach. Heuristic routing is more precise than trial and error and requires less preparation time than computer routing. The heuristic routing method was developed by the EPA in the mid-1970s (25). The method uses routing guidelines to set up the collection route. The heuristic rules are found in Table 8.31.

Routing requires a map showing street segments with number of services, garage and disposal site locations, heavily traveled streets, and one-way streets. A starting point is picked, and a continuous route is selected by applying the heuristic rules and patterns. The routing should be terminated when the number of services selected for that district is approached. Routes should be terminated at readily identifiable geographical features or artificial boundaries. Retracing route segments should be minimized, and fragmenting, or skipping uncollected areas within the route boundaries should be avoided.

Particular routings are efficient for certain block patterns. Other patterns should be considered whenever the grid has blocks arranged differently than shown in the basic routing patterns in Figure 8.12. Block patterns may be more simply identified by omitting unserved intersecting streets before establishing block patterns.

Upon completing the routing for the first district, a starting point for the next route is chosen and the procedure is repeated until all the districts within the zone have been routed. Once the initial routes have been set, each should be checked for alternate routings or modifications. Particular attention should be paid to

TABLE 8.31 Heuristic Routing Rules

-
1. Routes should not be fragmented or overlapping. Each route should be compact, consisting of street segments clustered in the same geographical area.
 2. Collection plus haul time should be reasonably constant for each route in the community.
 3. Collection routes should begin as near the garage as possible.
 4. Right-hand turns are to be preferred to left-hand turns.
 5. Heavily traveled streets should not be collected during rush hours.
 6. One-way streets are best collected by starting near the upper end of the street, working down through a looping process.
 7. Dead-end streets are to be considered as a segment of the street they intersect, since they can only be collected by passing down that street segment. They must be collected by walking down, backing down, or making a U-turn. Left turns may be kept to a minimum by collecting dead-end streets when they are to the right of the truck.
 8. Steep hills should be collected on both sides of the street while the vehicle is moving downhill for safety, loading ease, collection speed, vehicle wear, and fuel conservation.
 9. Higher elevations should be at the start of the route.
 10. For collection from one side of the street at a time, it is generally best to route with clockwise (right) turns around blocks.
 11. For collection from both sides of the street at the same time, it is generally better to route with long, straight paths across the grid before looping clockwise.
 12. For certain block configurations within the route, specific routing patterns should be applied.
 13. Corner-lot residents should be asked to place their waste on specific streets to eliminate the need to traverse an intersecting street.
-

isolated segments not serviced, deadhead distances, the appropriateness of particular routing patterns, and unique characteristics of the area such as low bridges or weight limits.

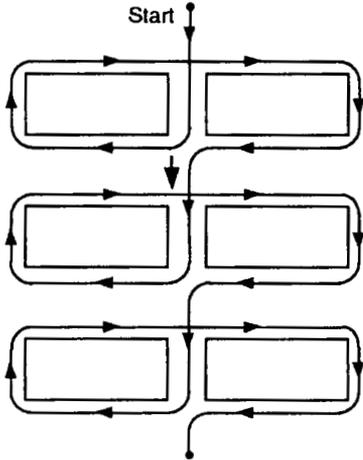
Source Separation

Materials can be recovered from the waste stream for recycling through source separation, the setting aside of waste materials at the point of generation. The glass, paper, metal, and other separated materials are most often collected, sold, and recycled. Two types of source separation programs are in common use. One type uses centralized recycling centers to which materials are brought by generators. The other type provides curbside collection of materials to be recycled. This discussion focuses on curbside collection of source-separated materials.

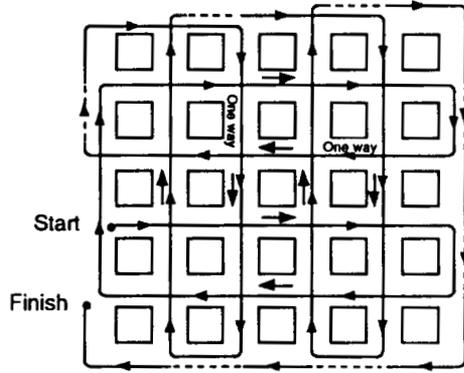
Collection Practices. To be effective, collection of source-separated material must be regular. Participants must be informed how to prepare materials for collection and when collection will take place. Generally collection intervals greater than two weeks significantly reduce participation in source separation programs.

Curbside collection is usually accomplished by using separate vehicles for each item, a single vehicle with several bins for each source-separated item, or by racks attached to the regular collection vehicle. Separate vehicles can cover more territory than the regular collection vehicle. Racks attached to regular collection vehicles usually fill before the packer body does. Strategically placed bins usually allow the racks to be emptied in about 15 min of off-route time.

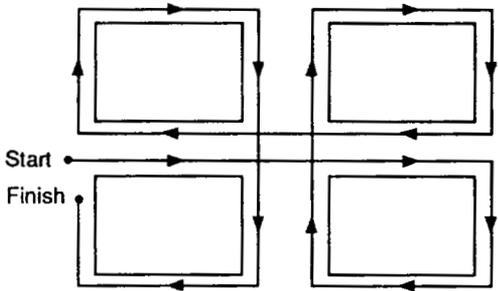
Separation Volumes. The approximate amount of material that can be collected through source separation may be calculated as follows:



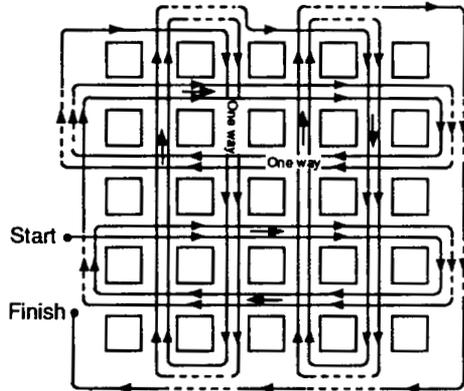
One- and two-way streets; one side collection



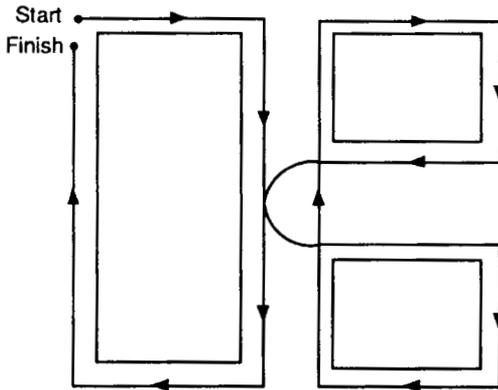
Both sides of the street collection; one-way streets



Four-block configuration applicable wherever four blocks are positioned as shown; two-way streets



One side of the street collection; one-way streets



Three-block configuration; two-way streets

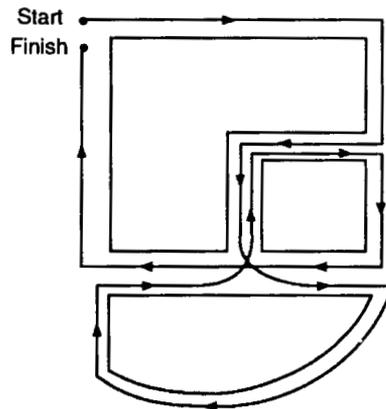


FIGURE 8.12 Basic heuristic routine patterns (25).

$$A = PHW \times H \times I \times P \times W$$

where A = the amount of items to be collected, lb (kg)

PHW = weight of total waste per household per week, lb (kg)

H = total households on the route

I = percent of total waste stream represented by the items separated

P = percent participation of households on the route

W = weeks between source-separated collections

Table 8.32 indicates the average density of various items usually collected in source separation projects.

Antiscavenging Ordinance. Due to the value of secondary materials, many cities have experienced difficulties with unauthorized persons picking up source-separated materials. An antiscavenging ordinance should be part of a source separation program. Antiscavenging ordinances should not preclude volunteer groups from collecting newspapers or scrap metal as one of their traditional revenue producers.

Rural Collections

Rural waste collection is of special concern. Many publicly sponsored rural systems have used bulk containers to substitute for the open dump and to facilitate collection. Bulk containers are located along major travel routes or near the site of previously operated open dumps. The resident takes waste to the container and deposits it for periodic collection by commercial-type collection vehicles.

Container sites should have ample room to allow a passenger vehicle or light truck to completely clear the highway to unload. In general, the container should be at least 10 ft (3 m) off of the road surface. Room must also be allowed for maneuvering the collection vehicle to pick up the container without interfering with traffic.

To estimate the amount of container capacity, the following data should be used:

Waste volume per tributary population	0.1 yd ³ (0.08 m ³) per week
Density in the container	75 to 150 lb/yd ³ (125 to 250 kg/m ³)
Maximum road distance container to contributor	5 to 6 mi (8 to 10 km)

Bulk container sites must be properly maintained. Spills or vandalism can make these sites unpleasant. Animals can upset small containers or forage in containers where lids have been left open. Large wild animals frequenting the site can discourage use of the bulk container system.

TABLE 8.32 Densities of Source-Separated Items

Item	Typical loose density	
	kg/m ³	lb/yd ³
Newsprint (bundled)	385	64.9
Aluminum (crushed cans)	55	92.7
Ferrous (crushed cans)	150	252.8
Plastics	20	33.7
Glass (crushed bottles)	870	1466.3

Transfer Operations

When the collection vehicle is filled, or at the end of the working day, the truck should be driven to a disposal site and emptied. In some areas, disposal sites are quite far from the collection area, resulting in too much off-route time for the collection vehicle and crew. To increase productivity, many collection agencies have established terminals where the route vehicles can empty their loads and return quickly to collecting refuse while a larger vehicle transports the loads of several collection vehicles to the disposal site. These terminals are called *transfer stations*. The purpose of a transfer station is to keep off-route time of collection vehicles to an acceptable minimum.

The most popular method of transfer is over-the-road haul using trucks. Barges are used by some coastal cities. Rail haul has been proposed as a method of removing solid waste from densely populated communities to rural areas for disposal in strip mines. Costs and public opposition have been largely responsible for the limited U.S. experience in rail haul.

In general, transfer stations have a main two-story building where collection vehicles unload into tractor trailers on the lower level. The refuse is usually compacted in the tractor to ensure economical loading. When a trailer is full, it is hauled to the disposal site and replaced by an empty trailer.

Transfer Station Economics. A transfer station is justified when the cost of transport from route to the transfer station, transfer, and haul to disposal using larger transfer vehicles is less than the cost of transport from route to disposal by smaller vehicles.

Past efforts at establishing transfer economics have often used rules of thumb, such as that a station is justified if one-way haul exceeds a certain distance. These rules are not adequate, because the off-route time of the collection vehicle is the controlling cost factor. Other factors affecting the economics of transfer include site considerations, such as traffic patterns, construction conditions, and capacity.

The costs of owning and operating a transfer station vary depending on station use. Data from several transfer stations in Ohio and Michigan yield the cost comparisons in Table 8.33.

Transfer Station Location. The principal reason for using a transfer station is to increase crew and truck productivity. A transfer station should be near the center of the collection area, convenient to good haul routes, and zoned industrially or, if necessary, commercially. The site should not be located in residential areas. A site can be at some distance from the center of production without excessive economic disadvantage.

Figure 8.13 illustrates a means of determining whether to use a transfer station or to have collection vehicles transport waste to disposal sites. The line of great est slope illustrates the cost of operating a collection vehicle, with a three-person crew, between the route and the disposal site. The intercept distance on the vertical axis is the cost of unloading time for this collection vehicle.

As the transfer station is seldom located at the end of a collection route, each collection vehicle will travel some distance to the transfer station. Therefore, the origin of the transfer costs line must be offset to account for the time of travel to the transfer station by the filled collection vehicle. As long as the combined

TABLE 8.33 Transfer Station Costs

	Compactor stations		Noncompactor stations	
Average transfer cost per ton (metric ton)	\$ 7.13	(\$ 7.86)	—	—
Average haul cost per ton (metric ton)	\$ 7.14	(\$ 7.87)	—	—
Average disposal cost per ton (metric ton)	\$ 8.35	(\$ 9.20)	—	—
Total cost of system per ton (metric ton)	\$22.26	(\$24.93)	\$12.53	(\$13.81)

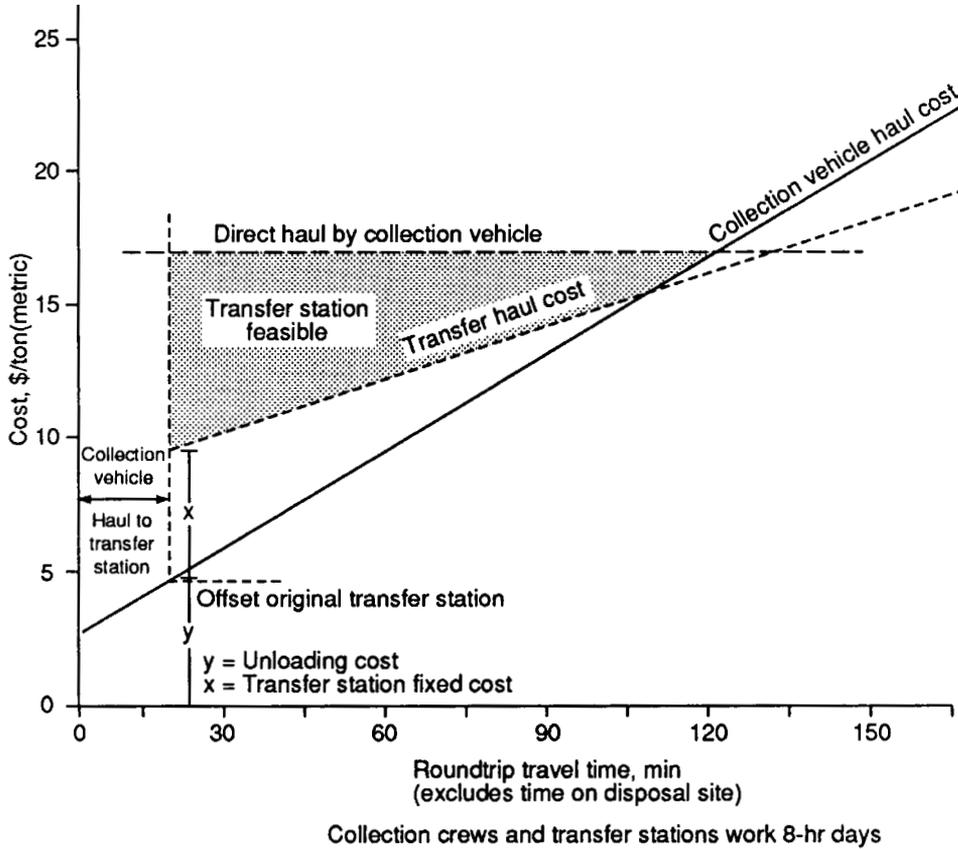


FIGURE 8.13 Economic analysis for transfer station siting.

costs of travel by the collection vehicle and transfer to disposal remains below the cost of direct haul by the collection vehicle, the transfer station is economical.

Transfer Station Design. Transfer stations should be attractive, convenient, and safe, with adequate storage. Station site, structure, equipment, staffing, routing, and backup can be designed following general design criteria. Local and state regulations should be reviewed before beginning design.

Site. The site should have enough space for buildings, storage, vehicle maneuvers, and expansion. Topography should encourage drainage and have sufficient elevation change to accommodate a two-level building. Foundation conditions should be able to support heavy industrial buildings and imposed equipment loads. Approximately 3 acres (1.25 ha) is the preferred size, although smaller sites are operating.

Access to the site should be from good, all-weather roads. Adjacent streets should be wide enough to allow transfer trailers to enter and leave the site without interfering with smooth traffic flow. Traffic should be visible from the gate house. On-site roads should be designed for all-weather operation and gates should control access.

The ramp outside the building should slope slightly away from the structure. Ramps should extend at least 100 ft (30 m) in front of the doors and 150 ft (45 m) or more if semi-trucks will use the facility.

A listing of preferred utilities, roadway characteristics, fencing, signage, and miscellaneous site features is presented in Table 8.34.

Structures. Structures mask the operation from neighbors, help control noise and blowing paper, and protect the operation from weather. Two structures are commonly found at transfer stations: the gate house and the transfer structure, or main building. In some stations, the gate house is incorporated into the main building design.

Drivers should have easy access to the gate house. Wicket-type windows will serve in dry weather, but drivers appreciate getting out of the rain or snow to sign usage slips. Doors opening onto the driveway should open inward to prevent them from being damaged by traffic. Canopies and overhangs should be at least 16 ft (5 m) above the roadway. The gate house should have sufficient room for the scale head, a desk, and file.

The main building should be enclosed. Partially enclosed or open transfer operations should be discouraged because it is difficult to control dust, noise, and debris at such stations. Open sites also may be attractive nuisances for children.

The building should be functional yet attractive in design. Steel modular buildings with pedestrian doors at each level are often seen. Windows should be shatterproof. The building should be high enough to accommodate a 16-ft (5-m) high door. If gravity discharge vehicles are expected, at least 24-ft (7-m) high doors are required to assure clearance if a driver leaves with the bed up. Full-width doors are preferable to

TABLE 8.34 Design Considerations for Solid Waste Transfer Station Sites

Design consideration	Specification
Preferred utilities	
Potable water	
Flushing water	100 gpm (6.4 L/s) @ 60–80 lb/in ² (42,200–56,250), also fire protection.
Sanitary sewers or septic tank and leach field	
Storm drainage	
Telephone	
Electricity	220 V, 60 Hz, 3 phase
Natural gas	
Roadways and ramps	
All-weather design	Asphaltic concrete or Portland cement concrete
One-way roadways	12 ft (3.5 m) minimum width
Two-way roadways	20 ft (6 m) minimum width
Thickness	According to base material and vehicle loading
Turning radius	50 ft (15 m) inside wheel
Grades	Preferred, 6%; normal maximum, 8%; absolute maximum, 10%
Ramps	Preferred minimum width beyond building-75 ft (23 m), if semi-trailers accepted, 150 ft (45 m)
Parking	Employee and visitor, 5–6 spaces. Trailers—adequate for number owned.
Scale platform	70 × 10 ft (21 × 3 m)
Fencing	8 ft (2.5-m) chain link with 3-strand barbed wire, 20-ft (6-m) entry gate
Signage	Speed limit Directional Caution/safety Site identification-keep to minimum
Miscellaneous	Fuel storage: vehicle and equipment, 3–4 months Hydraulic oil: 1 reservoir refilling stored in 55-gal (208-L) drums (minimum)

doors and columns. Roofs should be high enough to accommodate the open rear gate of ejector vehicles or the lifted body for gravity dump vehicles.

The building should be supplied with water to flush floors, which should be sloped toward floor drains. A small climate-controlled operator enclosure housing the controls and providing an unobstructed view of the dumping area is needed. Table 8.35 lists the principal design features of the transfer station main building.

Transfer Trailer Tunnel. Transfer stations usually have a tunnel for transfer trailers. The trailer is pulled into the tunnel and spotted at a loading station or coupled to a stationary compactor. Tunnel widths should be at least 16 ft (5 m) with vertical clearances not less than 16 ft (5 m).

Tipping Area. The tipping floor is the area where the collection vehicle unloads.

Transfer Techniques. The collection vehicle may discharge waste onto the tipping floor, into pits or hoppers, or directly into transfer trailers. When vehicles are emptied onto the tipping floor, a front loader is needed to place the refuse into the transfer trailer or into the hopper of an external compactor.

Direct discharge of loads into trailers requires the trailer top or hopper to be no more than 1 ft (30 cm) above the tipping floor level. Hoppers may be used to guide refuse into the opening of self-compacting trailers.

When stationary compactors are used to load vehicles, packer loads may be emptied into hoppers connecting to the compactor or into pits. Hoppers should accommodate at least one packer load of refuse. Where many vehicles are expected, the hopper or pit should be large enough to store several vehicle loads.

The tipping area should be wide enough to accommodate the number of vehicles expected in the peak hour of the average day. In general, this will be 2 to 2.5 times the average hourly number of vehicles expected on the average day. The average number of unloading spaces is obtained by dividing the daily number of

TABLE 8.35 Design Features of Transfer Station Main Building

Feature	Specification
Vehicle doors	16 ft (5 m) high, full width of approach lane, motorized
Door guards (minimum)	6-in (15-cm) pipe, filled with concrete, 4-ft (1.2-m) bury
Tipping floor	40–50 ft (12–15 m) deep to shelter crew during unloading; width to accommodate the number of vehicles
Operator enclosure	50–60 ft ² (5–6 m ²)
Transfer trailer stall(s)	16 ft (5 m) wide × 16 ft (5 m) vertical clearance; deep enough to accommodate trailer and compactor, plus 6 ft (2 m) with concrete floor; provide stairs to tipping floor.
Lighting	Adequate for maintenance area
Heating	60°F (16°C) maximum except operator shelters to 70°F (21°C); infrared heaters will work well on the tipping floor
Climate control	Air conditioning in operator shelter; minimum of six air changes per hour in tipping floor and pit area
Fire protection	Ionization-type smoke detectors; extinguishers at each unloading station (type A, B, C); fire hoses and/or spray system
Safety equipment	Safety harness at each unloading area if pit or hopper equipped; emergency ram stop button at each unloading station (compactor station only); first aid equipment
Drainage	Slope tipping floor to drain; pit drains to sanitary sewer; storm drain at transfer trailer tunnel or stall entrance; sanitary drain for leakage from compactor
Communications	Telephone, intercom
Maintenance shop	Fully equipped for machine and building maintenance

anticipated vehicles by the time needed to maneuver and empty the average vehicle. The average packer truck can clear the tipping floor in 5 to 7 min from entry, while manually unloaded vehicles may take from 15 to 30 min. Each unloading lane should be at least 12 ft (3.5 m) wide.

Tipping floors should be constructed of concrete designed to carry the heavy trucks. Floors should be free of columns or other obstructions and designed for the arriving vehicle to back straight to the unloading point. Wheel stops should be provided where the collection truck backs to an open drop. The minimum distance from the door of enclosed stations to the edge of a push pit or hopper should not be less than 40 ft (12 m).

Where vehicles discharge their loads directly onto the tipping floor, ample space is needed to store the loads expected under peak conditions. Loads from compactor-type bodies may expand up to twice their compacted volume when emptied. Loads cannot be stacked except by front loaders or cranes, and then only in a limited way.

Transfer Station Equipment. Transfer stations require fixed equipment, stationary compactors, transfer trailers and tractors, and backup equipment.

Scales. Scales should have a device to signal the drivers when weighing is complete without requiring them to dismount. If billing is to be done on weighed loads, consideration should be given to automatic recording, card-operated scales with a capacity of 100,000 lb (45,350 kg). Scale platforms should be 70 ft (21 m) long.

Fixed Equipment. A variety of fixed equipment is required in a transfer station. Some stations receive refuse into hoppers, hydraulic push pits, or bridge-crane unloaded pits; others require vehicles to dump onto a tipping floor.

The hopper is the simplest loading device. Collection vehicles back up to the hopper and unload into its throat. The cycling ram of the compactor empties the hopper. Some stations have collection vehicles unload on the floor, using a front loader to push material into the hopper.

Push pits are designed for direct unloading. The push pit is equipped with a screw or hydraulically operated push plate that pushes refuse from the pit into the ram of the stationary compactor. Hydraulic pits have a maximum length of about 50 ft (15 m); stationary screw pits may exceed 100 ft (30 m) in length. The rate of feed should be controlled by the operator.

Where push pits are installed, the end wall at the compactor should be sloped at 10 to 20° toward the pit to direct material into the compactor. Controls for operating the push plate and ram should be in the operator's station, which should afford a clear view of the entire pit, including the ram of the compactor. Emergency shutoff switches should be located at each discharge station.

Pits are sometimes emptied by overhead bridge cranes, which deposit material into one or more hoppers attached to compactors. Bridge cranes should be of heavy capacity and have at least 0.5-yd³ (1.25-m³) buckets. Pits using cranes should be armored with steel rails set vertically in concrete.

Pits must have safety harnesses, use of which should be mandatory for persons conducting manual unloading operations. The harnesses may be suspended from the ceiling by a 3/4-in (2-cm) nylon rope permitting a fall of no more than 3 to 5 ft (1 to 1½ m).

Stationary Compactors. Transfer station rams are large stationary compactors. They are the heart of the compaction-type transfer station. Typical ram characteristics are outlined in Table 8.36.

Stationary compactors and auxiliary hydraulic equipment are usually placed on a concrete base on the lower level. The access opening to connect the trailer to the compactor should have a door and be at least 12 ft (3.5 m) wide by 16 ft (5 m) high. A sump and drain should collect liquid that runs from the trailer during loading and direct it into the sanitary system. Ionization-type smoke detectors should be located in the loading area with an audible alarm in the dumping area and operator's station.

Mobile Equipment. A front loader is an essential part of operations where collection vehicles are unloaded onto a tipping floor. The front loader should be equipped with a protective shield over the radiator,

TABLE 8.36 Stationary Transfer Compactor Characteristics

Item	Range
Rated displacement	5.8–11.2 yd ³ (4.4–8.5 m ³)
Rated capacity	525–975 yd ³ /h (400–745 m ³ /h)
Operating pressure	1300–1700 lb/in ² (914,000–1,195,000 kg/m ²)
Total thrust	75,000–127,000 lb (34,000–57,600 kg)
Total weight	19,900–37,000 lb (9025–16,870 kg)
Oil tank capacity	240–375 gal (905–1,420 L)

rock-type treads on foam-filled rubber tires, and rear weights. Foam fill reduces downtime due to tire failure. Rear weights supply traction. A backhoe, used in place of weights, is also useful in distributing loads and compacting open trailers. Standard safety equipment is required, including roll bars, seat belts, and backup warning.

Transfer trailers receive the waste from the tipping floor; when full, they are taken to the disposal site and emptied. Trailers may be open or enclosed. The following formula is a method of estimating the number of trailers required for a smoothly functioning transfer operation. At least one spare trailer should be provided at each transfer station. A summary of transfer trailer characteristics appears in Table 8.37.

$$N_t = T \times V_t \times (L + M + t_r) / WD$$

where N_t = number of trailers (round to the nearest whole number)

T = weight in tons (U.S. or metric) received on maximum day ($2.2 \times$ avg. day)

V_t = average volume of trailers, yd³ (m³)

L = loading time, mm

M = maneuvering time (hooking up, spotting, and unhooking), mm

t_r = round-trip travel time to disposal site (includes time on site), mm

W = working time in a day, mm

D = compacted density, lb/yd³ (kg/m³)

Open Trailers. Open trailers are used at noncompacting transfer stations. The transfer trailer is a semi-trailer with rearward expanding body shape and an open top. The top is covered during travel with lightweight metal mesh or canvas lids mounted on light frames hinged to the trailer sides. The lids can be put in place from the ground by one person.

TABLE 8.37 Characteristics of Transfer Trailers

Type	Volume, yd ³ (m ³)	Approximate weight, lb (kg)	Maximum height and length, ft (m)	Capacity, tons (metric tons)	Remarks
Open trailer	70–130 (53–99)	20,000 (9100)	14 and 50 (4.3 and 15)	15–20 (13.6–18.1)	Dumping may be by gravity, live bottom, or ejection
Closed trailer	45–75 (34–57)	20,000–25,000 (9100–11,300)	13 and 40 (4 and 12)	15–20 (13.6–18.1)	Dumping by ejection

A variety of methods are available to load open trailers by gravity from the tipping floor. These include hydraulic systems and mechanical unloaders. Hydraulic unloading systems are more reliable than mechanical unloaders.

Hydraulic unloading requires equipping trailers with engines, reservoirs, and hydraulic pumps mounted externally at the front of the trailer. The hydraulic system operates a ram and push plate, unloading a trailer in less than 3 min. Another approach uses hydraulically actuated floor planks moving in sequence. Unloading a 50-ft (15-m) trailer, thus equipped, takes about 6 min.

Open-top transfer trailers may also be mechanically unloaded using a chain and flight conveyor. Chain conveyors are high-maintenance equipment. Chains must be properly adjusted to keep them from slipping on sprockets or breaking. A chain conveyor can unload a trailer in less than 4 min.

Open-top transfer trailers are manufactured in lengths ranging from 38 ft (11.5 m) to 50 ft (15 m), and load capacity ranges from 45 to 130 yd³ (34 to 99 m³). For average municipal waste, densities of up to 400 lb/yd³ (237 kg/m³) can be obtained using external compaction. Densities of 200 to 300 lb/yd³ (120 to 178 kg/m³) can be expected without compaction.

Enclosed Trailers. Two types of enclosed transfer trailers are currently on the market. One is loaded by an external compactor; the other has a self-contained compaction system.

Externally compacted transfer trailers rely on a stationary compactor to achieve maximum load density. Units are locked to the stationary compactor during the loading while the compactor pushes the charge through the trailer's rear door. Care must be taken not to overload the trailer or rebound will prevent the doors from closing securely. An internal hydraulic ram and push plate are used to unload the trailers.

Self-contained compaction trailers use an internal push plate to compact the load. Solid waste is loaded into an opening in the top front of the trailer. The plate then pushes the material to the rear of the trailer, compacting it against the rear doors. The cycle is repeated until the trailer is filled. Care must be taken to prevent charging the trailer while the ram is extended so that refuse is trapped between the plate and the front of the trailer.

Enclosed compaction transfer trailers may not be able to achieve optimum compaction and remain within legal highway load limits.

Trailer capacities vary from 45 to 75 yd³ (34 to 57 m³). Maximum compacted densities for municipal solid waste range from 550 lb/yd³ (326 kg/m³) for the self-contained units to 800 lb/yd³ (475 kg/m³) for the units loaded by external compactors. Trailer lengths range from 35 to 50 ft (11.5 to 15 m). Trailer sizes must be chosen to avoid exceeding highway load limits (including frost laws in northern states).

Transfer Tractors. The number of tractors needed to haul transfer trailers depends on the amount of refuse being delivered, the round-trip travel time to the disposal site, and the capacity of the trailers. Enough units are needed to permit the maximum day's receipts to be delivered in an 8-h period, allowing for a productive time of approximately 5.6 h. Fifteen minutes should be allowed for maneuvering and emptying on site and 10 min to drop the empty trailer and pick up a full one. The formula below indicates the number of necessary tractors, including one standby unit.

$$R = \frac{H_w}{t_r + M}$$

where R = round-trip haul time

H_w = working hours per day per person adjusted for unproductive time.

t_r = Over-the-road travel time, round trip.

M = Maneuvering time (hooking up, on-site travel, spotting, unloading and unhooking)

$$N_T = \frac{(N_r - 1)}{R}$$

8.70

where N_T = number of tractors

N_r = number of trailers

R = round-trip haul time

Backup Equipment. Contingency plans are necessary in the event of major equipment failure. Mobile equipment, such as loaders and semi-tractors, can be rented on short notice. Stationary compactors and push pits will require diversion of collection vehicles to an alternate location if sufficient storage is not available on the site. At least one spare transfer trailer should always be available.

Transfer Station Staffing. Although one person can operate a transfer station, safety requires two persons to be on-site at all times. Additional staff will be needed where heavy volume requires multiple pits or hoppers. Larger stations are staffed by two or three persons, while smaller rural stations may be unstaffed.

PROCESSING

Solid waste has been subjected to a number of material processing operations, similar to many other materials. However, due to the unique composition and character of solid waste, great care is required in the application of the various unit processes. In some cases, unit operations that are very effective on powdery materials are completely ineffective on solid waste. This section describes the application of shredders, trommel screens, magnetic separation, air classification, and baling in solid waste processing.

Shredding

Solid waste is typically subjected to shredding prior to its introduction to any separation operation. Shredding not only breaks the material into smaller size, making it more uniform, but shredding also imparts a blending action to the material stream. Solid waste management programs that have resource recovery as an ultimate goal often install shredding systems and separation systems afterward only when sufficient funds are available to expand the system. Even though shredded solid waste still is disposed at the landfill, a number of advantages accrue. These are (1) volume reduction, (2) reduced vector problems, (3) reduced fire hazard, (4) reduced odor potential, and (5) reduced litter problems.

Types of Shredders. The purpose of shredding is to make the material more uniform and, hence, more predictable. This is done by chopping up the items and mixing the fragments together. Shredders are essentially a rotating shaft with shaped weights or devices fastened on the periphery of the shaft. The devices (generally called hammers) are for the purpose of impacting or shearing the materials, or both. These machines are manufactured in two basic configurations: horizontal and vertical shaft machines. As the name implies, the main shaft is either horizontal or vertical. Some machines are made with very heavy and massive rotors, and others are made for higher-speed operation with lighter rotors.

Flail Mills. Flail mills are lightweight, relatively high-speed machines. Their application is primarily for tearing open bags of refuse and breaking up bundles of material in addition to providing some mixing action. This type of machine is not applicable to the single-stage milling of refuse, but should be followed by a second shredder that performs a more thorough size reduction. These machines have either rods or chains fastened to the rotor shaft and “flail” the refuse as it is fed to the machine (see Figure 8.14).

Impactors. On the opposite extreme to flail mills, there are impactors. These are massive machines with solid rotors. As the name implies, they impact the refuse as it enters the machine. The machine consists of two basic components: the rotor and one or two impactor target plates. As the material enters the machine, the massive rotor (sometimes with teeth) impacts the refuse, breaking some of the material into smaller par-

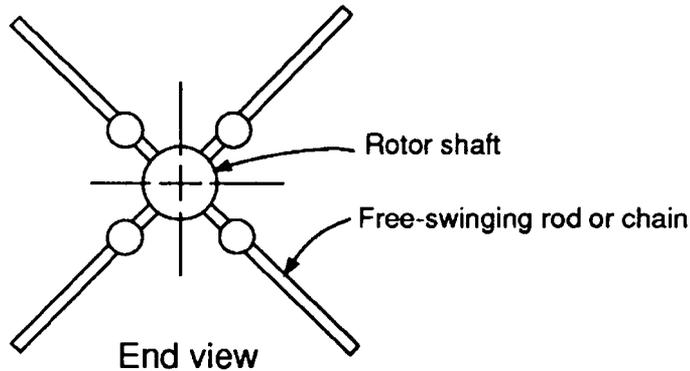


FIGURE 8.14 Flail mill.

ticles. The material is also “shot” toward the impactor plates where the refuse undergoes a second potential size reduction. These machines are not designed and built with grates or other devices for sizing the material. They are intended to break up the tough bulky items in the refuse stream for either subsequent milling or for feed to a refuse incinerator where oversize bulky items cannot be fed into the next process. The impactor performs excellently on friable materials, although those machines equipped with ripping teeth provide some size reductions by shearing the refuse.

Hammer Mills. Two types of hammer mills have been used for shredding refuse: the fixed hammer and the swing hammer type. The swing-hammer hammer mill has been most widely applied on refuse, but, under special conditions, the fixed-hammer machine may be more appropriate.

The fixed-hammer hammer mill is designed as a shearing-type machine. It is capable of reducing items, such as glass bottles, but generally it is a “light-duty” machine and is not applicable to mixed refuse. In situations where refuse has been presorted with bulky and other items removed, this type of machine will outperform most others and will provide the smallest particle size for a given amount of power input. The machine is essentially a series of knives or cutters bolted on the periphery of a disk (see Figure 8.15). A series

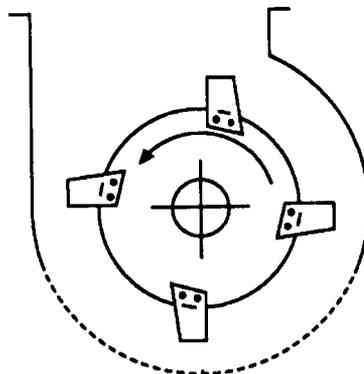


FIGURE 8.15 Fixed-hammer hammer mill.

of disks are stacked on a rotor shaft, forming a rotor assembly. The machines are also equipped with a grate bar or screen that restricts the flow of material until it has been reduced to some acceptable size.

The most commonly applied machine for the shredding of both mixed and sorted municipal refuse is the swing-hammer hammer mill. These machines can be obtained with either a vertical or a horizontal shaft; the latter being the more common design. The machines are similar to the fixed-hammer unit except that the hammer is allowed to pivot. There are a number of minor and some major variations in the design of each manufacturer's mill.

Design Considerations. Shredding designs tend to emphasize either capacity or capability; the two are not synonymous. In simplistic terms, capacity refers to the mass per unit of time a machine will shred while producing a given product, i.e., a certain particle size distribution. This tends to ignore the consideration of capability, which is the ability of the machine to shred a given item. Generally, designers have placed greater emphasis upon capacity for the reason that subsequent processes perform more efficiently, while equipment suppliers have emphasized capability, since this represents the machine's ability to perform at any given moment and, therefore, is an indirect indication of potential downtime.

Particle Size Distribution. Studies of particle size distributions of refuse from shredders have concluded that the particle size distribution can be mathematically described by a Rosin–Rammler relationship as follows:

$$Y_x = 1 - \exp [-(x/x_0)^n]$$

where Y_x = the cumulative decimal fraction passing a given screen size

x_0 = size at 63.2% passing

n = the index of distribution

Characteristic Particle Size and Power Requirements. The 63.2% passing-screen size is considered the characteristic particle size. For a given characteristic particle size x_0 , the power consumption per ton of refuse can be projected. In addition, the grate bar spacing versus characteristic particle size can be predicted, and from the data, the proper motor size and grate bar spacing can be determined for a desired x_0 . Suppliers' data should be consulted.

Shredder Selection. Consider, now, the selection of a shredder in the situation where the plant will accept all types of refuse and minimum inspection and sorting will be performed. A shredder can be viewed as an energy-storage device accepting energy from the motor, storing it, and at some other point in time transmitting it to the refuse. Newtonian physics says that there is energy associated with a moving body

$$\text{Kinetic energy (KE)} = \frac{MV^2}{2}$$

where M = mass

V = velocity

Mass is defined as

$$M = \frac{w}{g}$$

where w = weight

g = acceleration due to gravity

Kinetic energy is also defined as

$$KE = \frac{wV^2}{2g}$$

but for rotational energy

$$V = \omega r$$

where ω = revolutions or radians per unit time
 r = the radius.

The effective radius is the radius of gyration, which is the distance from the center of rotation about which the entire mass can be visualized as acting.

$$KE = \frac{K\omega}{2g} (NR)^2$$

Usual engineering terminology is to describe ω as radians per second and N as revolutions per minute, and R as the radius of gyration. In this case, K is a factor for adjustment of the units and is constant for each system (i.e., U.S. Customary or SI). Regrouping, we have

$$KE = \frac{K}{2g} (\omega R N^2)$$

In examining the above equation, it is found that the kinetic energy released when the rotor slows down is not a linear function but an exponential one. Each halving of the speed produces four units of energy release. For a given speed change it can be seen that doubling the radius of gyration will also produce four units of available energy. While there is an optimum operating speed for each machine, the manufacturer can only control or truly influence the ωR^2 of the machine.

For any given speed of operation, the larger the ωR^2 , the more energy it can store and release for shredding a given object. In general, machines of different ωR^2 (sometimes called rotating moment of inertia) will not undergo the same speed change. Any given article contained in the refuse stream requires a given amount of energy to reduce it in size. That object will remain in the shredder until it has absorbed, at least, that minimum amount of energy and can pass through the grate bars.

A machine with a small ωR^2 will release all of its energy to a given item and have none left for the other refuse that was fed with the "tough" object. Two possible things will occur. Either the object must be fed in by itself and no additional refuse fed until the machine has cleared itself, or the machine will come to a stop. The first option requires careful operator control and reduces the actual production rate of the system in addition to a change in degree of homogenization. The alternate requires opening up the shredder and clearing the refuse out of the machine. Both situations are undesirable because of lost production time and because machines with inadequate ωR^2 are prone to damage because they are not heavy duty enough for the application.

Some general guidelines can be drawn and used as guides in the selection of shredders. No municipal refuse shredder should have a ωR^2 of less than 25,000 lb/ft² (1200 kN/m²) unless the refuse stream has been highly refined by hand sorting and the refuse originated from households only. Machines of relatively small ωR^2 can handle tougher items when fed only that item; however, wear and tear on the machine is not a linear function (see Figure 8.16). As the limits of the shredder's capability are approached, the maintenance will tend to increase very rapidly.

The following items should be excluded from all swing hammer mills regardless of ωR^2 :

1. Items that are likely to explode, such as propane or butane bottles and gasoline cans
2. Large, thick-walled metallic objects, such as gas cylinders and pressure vessels

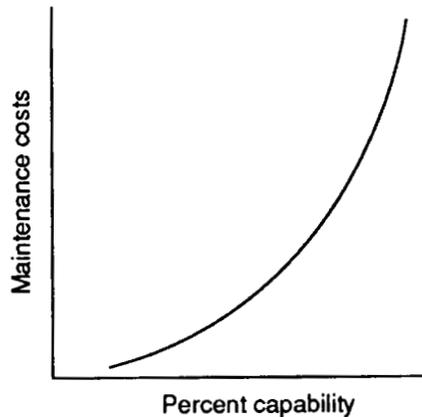


FIGURE 8.16 Shredder capability relative to maintenance costs.

3. Heavy wire rope and heavy industrial chain in lengths greater than circumference of rotor
4. Large blocks of plastic or rubber
5. Heavy truck and earth-moving-vehicle tires
6. Long, dense rolls of wovens, such as industrial carpeting, tarpaulins longer than rotor circumference, and long pieces of conveyor belting

Motor Size. Generally, the motor is sized by the manufacturer to match a specific shredder. This is a function of speed of rotation and ωR^2 . In practice, as the grate bar spacing is changed, the capacity of the shredder changes to match the available power. This is done by the system designer who uses the shredder motor amperage demand to control the feed conveyor speed (or vibrating feed frequency). The higher the shredder current demand, the lower the conveyor speed.

When the refuse is household-derived, a conventional motor sized for particle size control will generally prove satisfactory. When the application becomes more severe, the systems designer will want to select a high “pull out” torque motor. Simplistically, torque is a function of slip, or the difference between rated and actual rotational speed. In North America, common high-torque motors are in the range of 250%, while in many European countries, motors will range as high as 450%. The actual selection of motor specification is properly left to the electrical engineer, but the systems engineer should be cognizant of the severe duty required of shredder motors. In almost all cases, reduced voltage starting will be required. To determine the proper level of starting voltage, the electrical engineer will need to know the range of ωR^2 the shredder may have. The pull out level and slip determine how much kinetic energy can be reclaimed from the shredder and still recover without shutting the system down.

Particle Size. Because maximum particle size rather than average or mean particle size has been the major concern of the systems designer, most shredders are rated on the basis of nominal particle size. This is an arbitrary sizing, but is the screen size at which 90% of the material will pass a given opening. There seems to be some minor difference between vertical- and horizontal-shaft machines. Figure 8.17 shows typical particle size distributions for conventional grate spacings for horizontal-shaft machines.

In addition to changes in the capacity of a shredder resulting from changes in grate bar spacing, the systems engineer should note that when refuse density changes, the shredder capacity, measured on a weight basis, will change significantly (see Figure 8.18). Figure 8.18 was developed principally for a horizontal shaft shredder being fed raw medium-character refuse and shredding to 1.5-in (3.8-cm) nominal particle

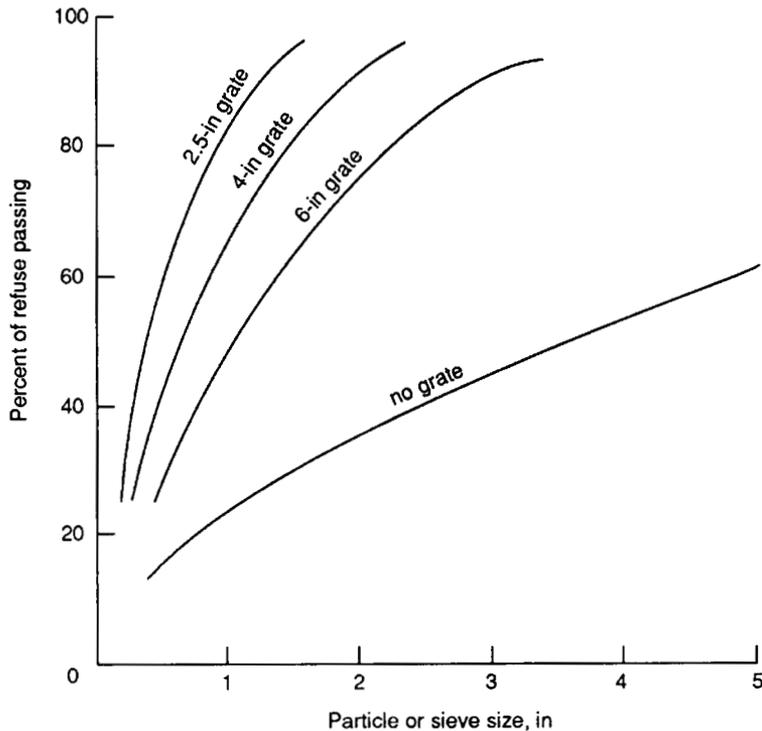


FIGURE 8.17 Particle size distribution for conventional shredder grate spacing.

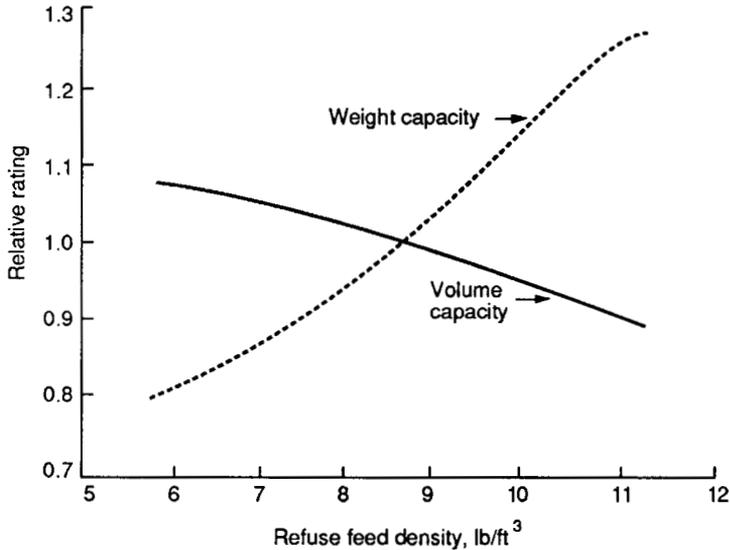
size. It should be noted that volumetric capacity is relatively insensitive to changes in density. In effect, shredders have an intrinsic capacity that is indicative of its working internal volume.

Shredder Rating. Table 8.38 presents a typical manufacturer's rating of its own shredders. (Note: The numbers have been rounded off slightly to eliminate specific manufacturer identification).

Special Design Aspects. Shredder design should also consider the following.

Lubrication. When selecting a shredder, the system engineer should be aware of the type of lubrication system being offered by the vendor. A circulating oil system with reservoir and oil filter is the most desirable, and manually lubricated grease fittings the least desirable, although this is a function of bearing size and rated life.

Hammer and Grate Wear. Hammers are rapidly wearing items, and grate bars also tend to require frequent replacement. Tough items tend to rotate with the rotor for some number of revolutions before being reduced sufficiently to exit from the machine. During this period, the grate bars perform some significant shearing action on the refuse as it passes by the grate bars. When the hammers wear down to the point that the clearance between the grates and hammers becomes excessive, two performance characteristics will change. An increase in particle size will occur as the result of the hammer trying to extrude the refuse out the grate, and shredder capacity will decrease as a result of partial plugging.



Note: $1\text{ lb/ft}^3 = 16\text{ kg/m}^3$

FIGURE 8.18 Shredder weight and volume capacity relationships.

Airflow. Because the rotor direction of travel (one side of the rotor) and the path of the refuse are the same, any restrictions in refuse flow will also affect the air current inside the shredder. This air current helps to carry the refuse through the machine. When refuse is first shredded, it fluffs up. After some vibration (going over idlers on the discharge belt conveyor, etc.), it may compact to a higher density than the original raw refuse. If sufficient space does not exist between the grates and the top surface of the belt conveyor, it will restrict the airflow. A minimum of 2 ft (0.6 m) should be kept clear between the bottom of the shredder and the top of the receiving conveyor.

A 4-ft (1.24-m) clearance is more desirable, and distances less than this should be used only when there are severe space constraints. A quick way to determine if a shredder is air-bound is to release smoke in the area where the rotor shaft goes through the shredder side. There generally is a gap between the shaft and the shell. If the machine is not air-bound, it will draw the smoke into the shredder; otherwise, the smoke will blow away.

TABLE 8.38 Shredder Ratings

Feed opening, in	Capacity, tons/h	Horsepower	ωR^2 , lb-ft ²
77 × 90	65	1000	248,000
60 × 90	55	800	180,000
54 × 90	45	800	100,000
54 × 80	35	600	55,000
54 × 50	25	400	45,000
35 × 50	15	250	12,800

Note: 1 in = 2.54 cm; 1 ton/h = 0.9 t/h; 1 hp = 0.75 kW; 1 lb-ft² = 0.042 kg-m².

Entrance Velocity. When the refuse is being charged into the shredder, it must have sufficient velocity. With adequate velocity, it will be able to penetrate the space between the rows of hammers. Otherwise, the material will sit on the tops of the hammers and “bounce” back. A rule of thumb is that the theoretical velocity of the refuse should be at least 10% of the hammer tip speed.

Feed Chute Design. In the design of feed chutes, there are some practical considerations that the designer must take into account. Sealing around a feed chute is important. If a vibrating feeder is used to feed the shredder, then heavy rubber can be used to seal the gap between the end of the pan and the feed chute on the bottom and sides. When a mechanical conveyor is feeding the shredder, a rubber seal should be designed between the bottom of the feed chute opening and the drip pan under the conveyor. Sealing the feed chute to the sides of the conveyor becomes difficult depending upon the conveyor drive arrangement. Shredders will reject material up the feed chute, and any openings will be found and the refuse will escape. Minor additional expense in sealing more than offsets the cleanup labor costs for a system in which little attention was devoted to this item. Whether a horizontal- or vertical-shaft shredder is selected, this type of action occurs. The top on the feed hood in some cases is sloped downward toward the opening. Any material striking the top will tend to be deflected away from the opening and contain the material within the hood. In addition, rubber sheeting should cover the opening and, when it is planned that the shredder will accept bulky material, chains should back up the rubber sheeting. The designer should also provide for minimum exposure of the feed equipment to the material that flies back. The head of the mechanical conveyor should not project into the feed chute. The sketches in Figure 8.19 illustrate some of the points discussed.

Trommel Screens

A separation method based upon particle size uses the trommel screen, sometimes called a rotary screen or rotary tube. At this point, the designer should consult the various equipment suppliers, since many trommel

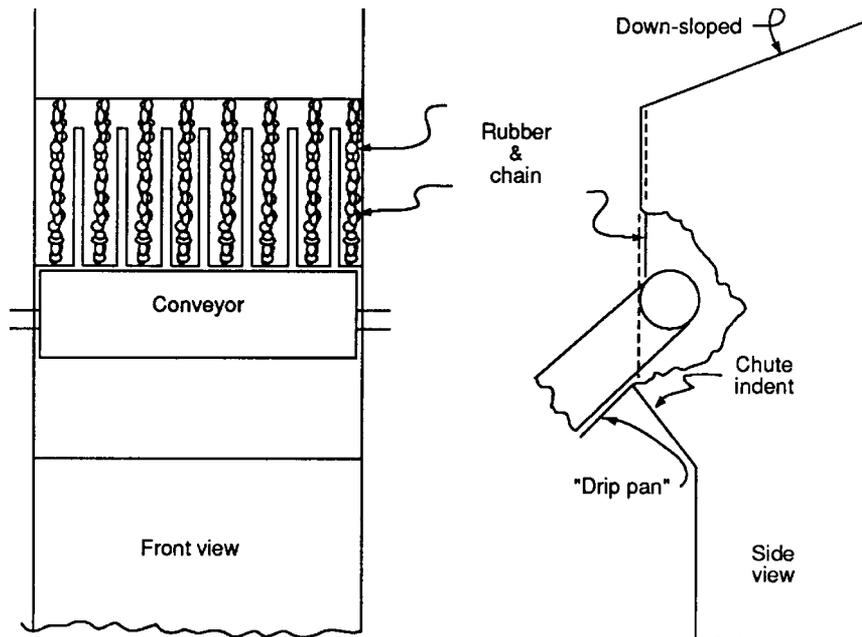


FIGURE 8.19 Shredder feed chute characteristics.

screens are custom manufactured and are of a specialized design. However, there are a number of manufacturers of “standard” trommel screens for which the following discussion is applicable.

Design Considerations. The material transport capacity of a rotating tubular device is described by:

$$V_F = 5.19N(R)^3 \left(\frac{H}{R} \right) \frac{\tan S}{\sin \alpha}$$

where V_F = volumetric flow rate of material, ft³/min (m³/min)

5.19 = dimensionless coefficient describing fundamental physics of unit

N = rotational speed, r/min

H = maximum bed depth of material, ft (m)

R = radius, ft (m)

S = slope angle of tube

α = angle of repose (dynamic)

In addition, there is a maximum rotational speed called the *critical speed*, above which the material travels full circle in contact with the drum. One can calculate the critical speed from

$$N_0 = \frac{1}{2\pi} \sqrt{\frac{g_0}{R}}$$

(H/R) is called the *bed depth fraction* and can be calculated from the following:

$$\frac{H}{R} = 1 - \cos \left(\frac{\theta}{2} \right)$$

where θ is the bed angle from Figure 8.20.

In practice, if the unit does not perform as well as anticipated, extra lifting blades can be added to the inside to slow down or speed up the flow of material. The higher the moisture content, the longer the detention time. Trommel length should not be less than the equivalent of about two diameters. It is also generally good practice to remain below 50% of the critical speed for the design conditions. A bed angle of 120° is fairly reasonable but should not be exceeded. This value gives slightly under a loaded area of 20%; that is, the tube is running 20% full at the inlet side.

Trommel Selection. Table 8.39 shows standard sizes and rates for two different manufacturers. For estimating electrical needs, a value of 1 hp per ton per hour of capacity (0.95 kw per 0.9 metric ton per hour) can be used by selecting the next-larger-size motor available.

Magnetic Ferrous Separation

Magnetic separation of ferrous scrap is a relatively easy and inexpensive operation to accomplish. Scrap reclaimed from municipal refuse generally is classified as No. 3 Dealer Bundle. When scrap is subsequently agitated and air-classified, it can in some cases be cleaned of foreign material sufficiently to be reclassified. The ability to clean ferrous scrap reclaimed from municipal refuse is a function of the amount of “balling” it received in the shredding process. Shredders that impart higher shear and lower impact to the refuse will tend to produce a cleaner scrap. The cleanliness of the product seems a matter of hammer style and shredder size, in terms of ωR^2 . Ring hammers, as opposed to hourglass or boise-style hammers, will tend to roll or

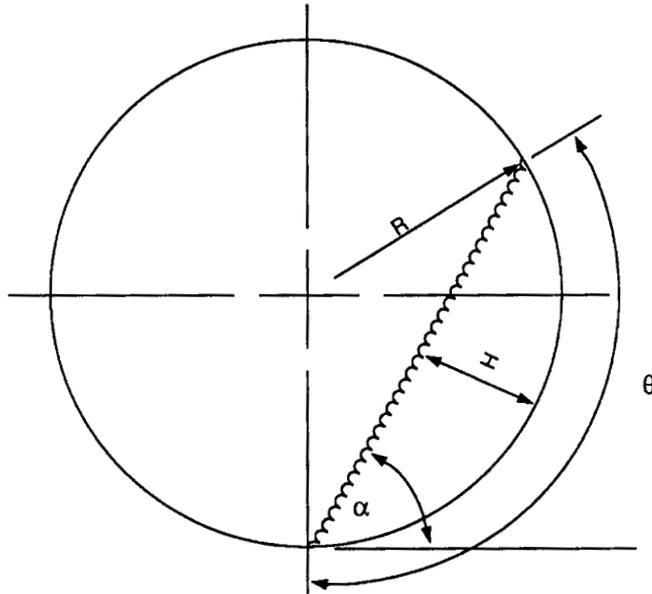


FIGURE 8.20 Trommel screen geometry.

grind the refuse as opposed to chopping it up. On the other hand, a shredder with a larger ωR^2 and heavy hammers, typically over 60 lb (26 kg) each, will tend to shear the material as a result of its ability to rip through the refuse with a minimum amount of hammer “lay-back,” i.e., hammer rotation about the hammer pin on impact. Hammer weight and ωR^2 are a function of shredder capability and size and not a function of its style.

Types of Magnetic Separators. Magnetic separation is a process that was originally used for the enrichment of iron ore slurries. They are drum-style (see Figure 8.21) with permanent magnets mounted on the inside in which the drum rotated while the magnet, covering only a segment, remained fixed.

TABLE 8.39 Standard Trommel Sizes

Capacity, ton/h	Diameter, ft		Length, ft		r/min (Mfg. B)	Horsepower	
	Mfg. A	Mfg. B	Mfg. A	Mfg. B		Mfg. A	Mfg. B
up to 15	7	4	15	20	22	15	7.5
26	—	5.33	—	25	16	—	20
37.5	—	6.66	—	30	13	—	30
45	8	—	31	—	—	50	—
60	—	10.66	—	30	0	—	75
80	10.5	—	47	—	—	80	—
100	12	—	62	—	—	100	—

Note: 1 ton/h = 0.9 t/h; 1 ft = 0.3 m; 1 in = 2.54 cm; 1 hp = 0.75 kW.

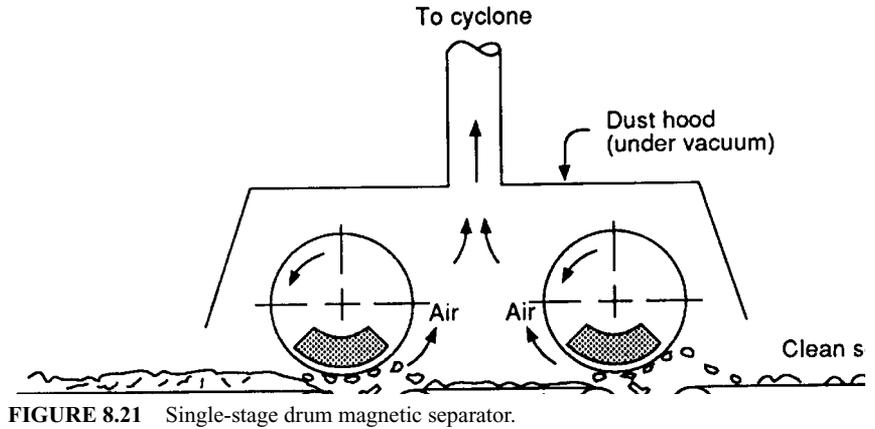


FIGURE 8.21 Single-stage drum magnetic separator.

In response to the need to produce a cleaner product, the various separator manufacturers developed a special line of refuse magnetic separators. These are suspended belt or “horseshoe” style and subject the ferrous product to a spinning or flipping action that tends to produce a cleaner product. The automobile shredding industry, faced with the same problems, has generally adopted a two-stage drum magnet system (see Figure 8.22) with a crude air-cleaning system between the two separation stages. This configuration produces a cleaner product but consumes more power. In situations where budget allows or the material specifications absolutely require the cleaner product, municipalities will adopt the system used by the scrap industry.

The most widely applied separators to municipal refuse is the suspended belt or “horseshoe” style, which

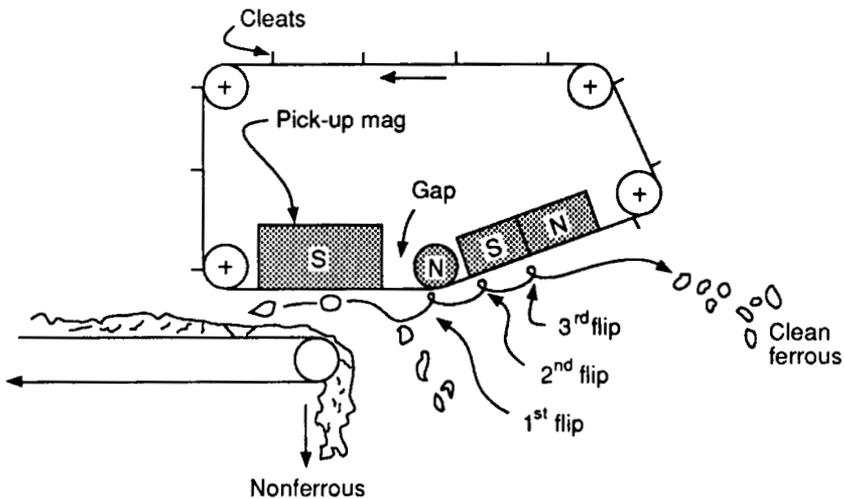


FIGURE 8.22 Two-stage drum magnetic separator.

is a series of reverse-pole magnets with a belt traveling across them (see Figure 8.23). The first magnet is the pickup magnet, and the subsequent magnets are reverse-poled sequentially. As the tin can, etc., is magnetized, the opposite pole attracts the end away from the belt, causing the item to flip or spin as it progresses. In some cases, a gap is included between the pickup and the next magnet to allow the item to temporarily drop away from the belt releasing any trapped material picked up with the metallic scrap. Steel pulleys and idlers can become magnetized and cause the ferrous scrap to remain on the belt as well as building up high static electrical voltages. An electrical engineer should review the possible need for grounding in this area of the conveyor. Consultation should also take place with the various separator manufacturers.

Design Considerations. Magnetism, like light, varies in intensity with the square root of the distance. This means that the performance will fall off drastically as the gap is increased small amounts. Generally, magnetic separators are designed for a 6- to 8-in (15- to 20-cm) gap between the conveyor belt surface and the surface of the separator pickup section. The surface speed of the separator is preferred to be about 50 ft/min (15 m/min) faster than the speed of the belt conveyor from which it is removing the ferrous material. The practice is to remove the ferrous scrap faster than it is being fed.

Recovery Efficiency. When considering recovery efficiency, it is recommended that the systems designer use a value of 95%. Recovery efficiency will depend upon flux density and thus gap distance. Just about all separators will operate at a far superior level (97 to 98%) most of the time; the recommended value takes into consideration periods when the refuse bed on the conveyor is particularly thick or some other variable is affecting performance. Table 8.40 can be used to select a magnetic separator.

Power Requirements. These units will vary in power consumption for the pickup magnet; the other magnets are almost always permanent magnets, and the belt drive will vary from 5 up to 10 hp (6.7 up to 13.4 kW) depending upon the type of belt used.

Serving Conveyors. Because of the impact force, points, and edges contained by the scrap, some of the units are equipped with aluminum armor plating in the center section of the belt. This reduces belt wear and increases belt life.

The chuting and covers around the discharge from a magnetic separator require careful consideration. Because of the high speed, the separated metallics generally have a high velocity when released from the belt or drum. A deflector plate with a replaceable liner should be designed for the reclaimed ferrous materi-

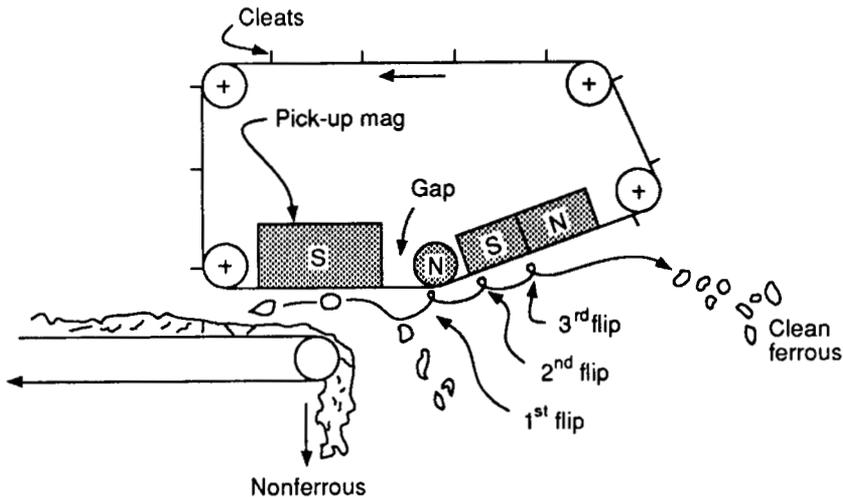


FIGURE 8.23 Suspended belt/horseshoe magnetic separator.

al to strike. The plate can be fabricated of steel when a distance of 2.5 ft (0.75 m) is maintained; otherwise, the metal will tend to interrupt the magnetic field pattern. The sides can be fabricated of rubber sheeting, but care should be taken that the sheeting does not get caught in the moving parts of the machinery. The head pulley and idlers in the area of the magnet should also be heavily rubber-covered or made of rubber.

Air Classification

Air classification has been, and will probably continue to be, the major unit process for the beneficiation of municipal solid waste second only to magnetic separation. In simplistic terms, a rising current of air effects the material separation because of the differential density of the refuse components. A key element in the separation is the proper preparation of the feed material. Air classifiers operate because viscous drag on some of the particles is able to overcome the force of gravity. If relative density is to be the criterion for separation, then the viscous drag on each particle must be uniform regardless of the material. The implication is that the more uniform the particle size, the more definable and predictable will be the separation. The further implication from fluid mechanics is that the larger the particle size, the higher the minimum upflow air velocity for fluidization of the material.

Design Considerations. As illustrated in Figure 8.24, an air classification system consists of (1) an airtight feeder, (2) a separation chamber with a top and bottom exit, (3) a receiving–settling chamber, (4) a prime mover (vacuum fan), and (5) a heavies takeaway conveyor. Generally, the entire system is provided by a single manufacturer or supplier, except for the feed conveyor and two takeaway conveyors.

Air Classifier Performance. In considering the performance of an air classifier, the design engineer needs to be concerned with the recovery efficiency E_1 and the rejection efficiency E_2 . The purpose of an air classifier is to separate the combustible from the noncombustible fraction. An excessive air velocity will minimize the rejection efficiency and recovery all of the burnable portion. However, the high degree of nonburnable fraction in the recovered portion may not be tolerable in the burning process. At the same time, insufficient air velocity will produce a clean recovered product, but the quantity may be so small as to make the separation process uneconomic.

$$E_1 = \frac{\text{combustibles recovered}}{\text{combustibles in feed}}$$

and

TABLE 8.40 Magnetic Separator Selection Data

Refuse feed rate, tons/h	Separator width, in	Conveyor width, in	Separator weight, lb
Up to 15	30	30	9,000
30	36	36	9,500
50	48	48	14,000
70	60	60	17,500
100	66	72	22,000

Note: 1 ton/h = 0.9 t/h; 1 in = 2.54 cm; 1 lb = 0.45 kg.

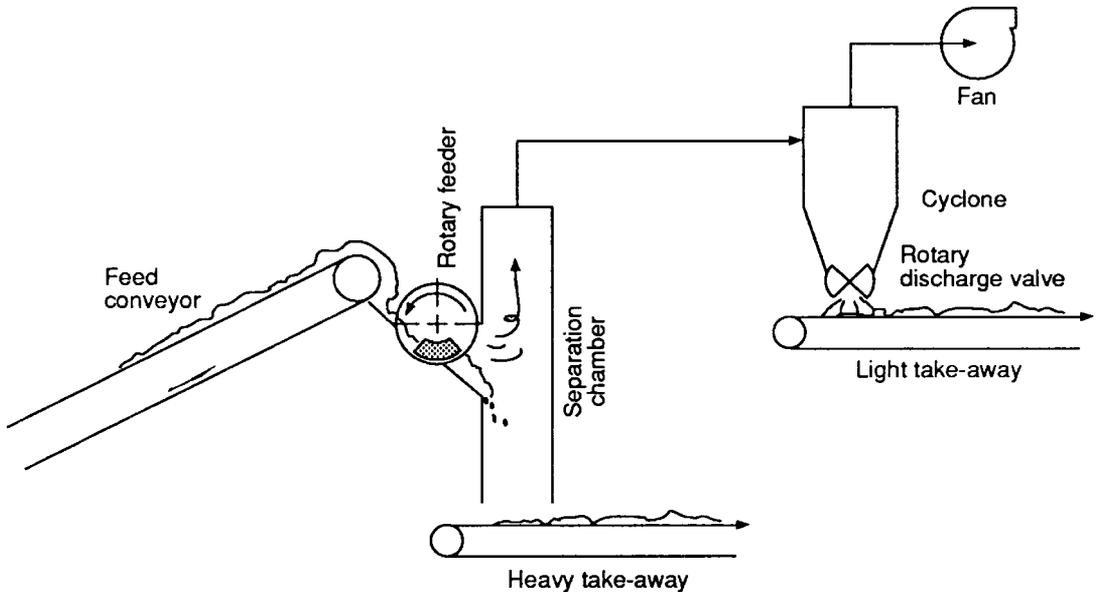


FIGURE 8.24 Air classification system.

$$E_2 = \frac{\text{noncombustibles rejected}}{\text{noncombustibles in feed}}$$

Since the two efficiencies are not independent of one another, Figure 8.25 shows efficiencies of an air classifier as a function of the throat velocity. The graph is typical of a common air classifier with a feed of municipal refuse prior to magnetic separation at 30% moisture content (E_1 and E_2 on moisture-free basis).

Baling

Baling is a relatively simple process that does not alter the physical or chemical nature of the solid waste. Rather it is a compression process for the significant reduction of volume occupied by the solid waste. In addition, baling produces a predictable product, the bale, that is both much easier to handle and produces much smaller voids at the disposal site. Baled solid waste is less prone to methane generation; generally will not support combustion; and produces a leachate of a less concentrated character.

Baler Operation. The solid waste is fed to a baler cavity; at this point the solid waste is then compressed in two of the three major axes to a fixed dimension. A ram then compresses the solid waste along the third axis until the ram pressure reaches some predetermined level. The bale produced will have two of its three dimensions fixed and the third dimension somewhat variable depending upon the amount and nature of the material originally charged to the baler. Depending upon the preset cutoff pressure for the third ram, the bale may require tying or binding up. Intermediate- and low-pressure baling will require banding, while high-pressure baling generally does not. The trade-off is between the cost of banding or strapping material and the cost of the additional energy to form a high-density bale. High-density balers are generally only available in the higher-capacity units, i.e., 50 tons/h (45 t/h) or greater capacity.

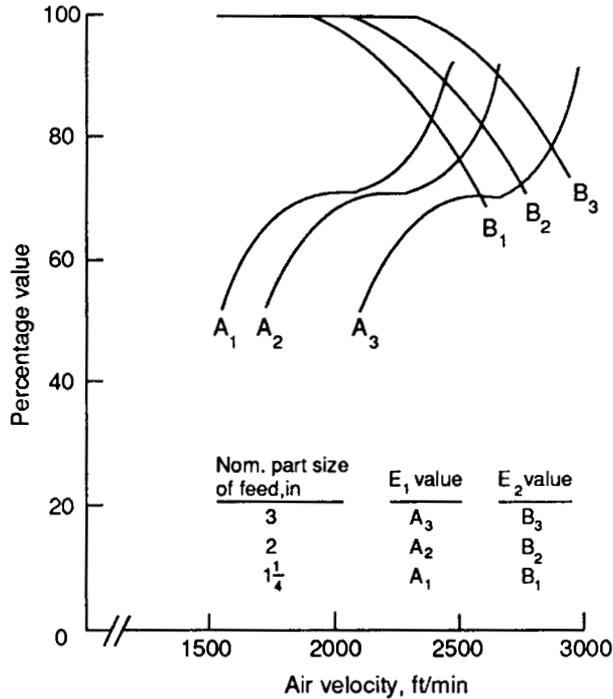


FIGURE 8.25 Air classifier efficiencies.

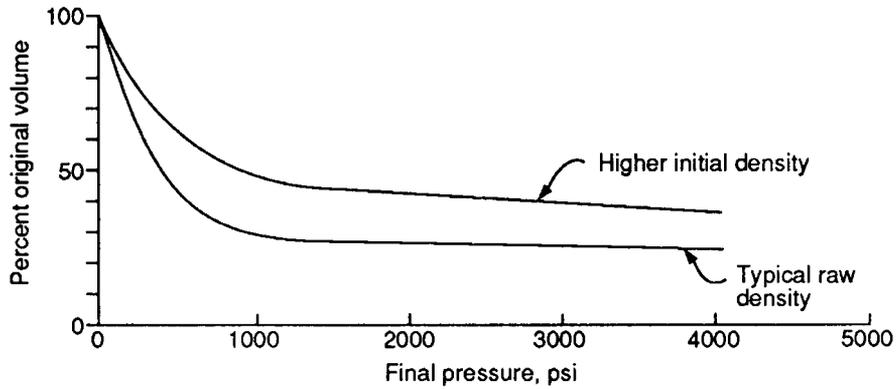
A number of factors affect the final density of the bale. These factors include not only the baler operation itself but also the amount and character of the refuse as well as time since the bale formed. Baler operational characteristics include pressure of application and time of pressure application. Solid waste parameters affecting the final bale include moisture content, mechanical properties (elasticity, etc.), and size and number of the individual items fed to the baler. Finally, all bales experience a “spring-back” effect, creating a variable time lag before bales reach their final density.

Baler Selection. The final density will be a function of the ram pressure. Below approximately 1000 lb/in² (6900 kN/m²), unstable bales will be produced regardless of the other parameters. Intermediate pressures up to approximately 3000 lb/in² (20,700 kN/m²) or slightly higher will produce stable bales if they are tied or banded. Above about 3500 psi (24,150 kN/m²), a stable bale can be produced without the requirement for tying. Figure 8.26 shows the relationship between baler ram pressure and volume reduction.

Figure 8.27 indicates the typical range for final bale weight as a function of connected power. The shape of the curve would be as expected from the pressure–volume change curve. The higher-power balers do not require wire tying even though the final bale weight is not significantly greater.

Bale Stability. Bale stability will increase as moisture content is increased. However, once a moisture level beyond 30% (on a dry weight basis) is reached, bale stability again starts to decline.

Specific refuse character will also affect bale stability. For example, it is not desirable for any bale to contain more than one tire. Large amounts of lawn wastes, such as grass or leaves, will reduce bale stability when any bale contains more than 50% of these kinds of materials. On the other end of the spectrum, balers



Note: 1psi = 6.9 kN/m²

FIGURE 8.26 Baler ram pressure and volume reduction.

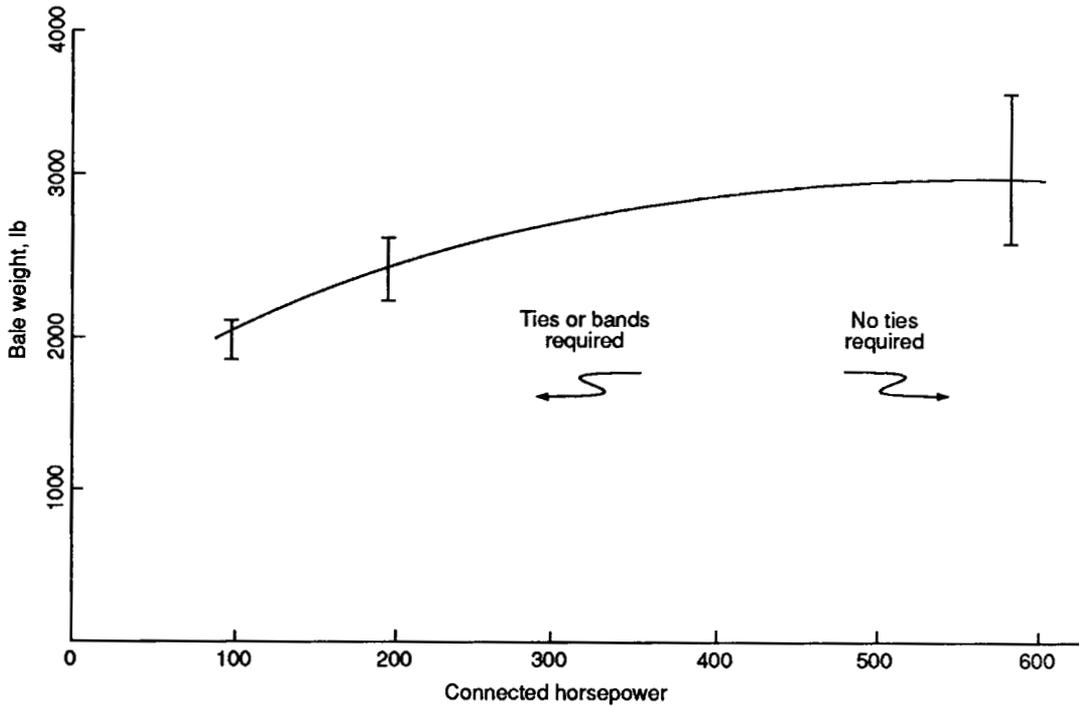


FIGURE 8.27 Bale weight and connected power relationship.

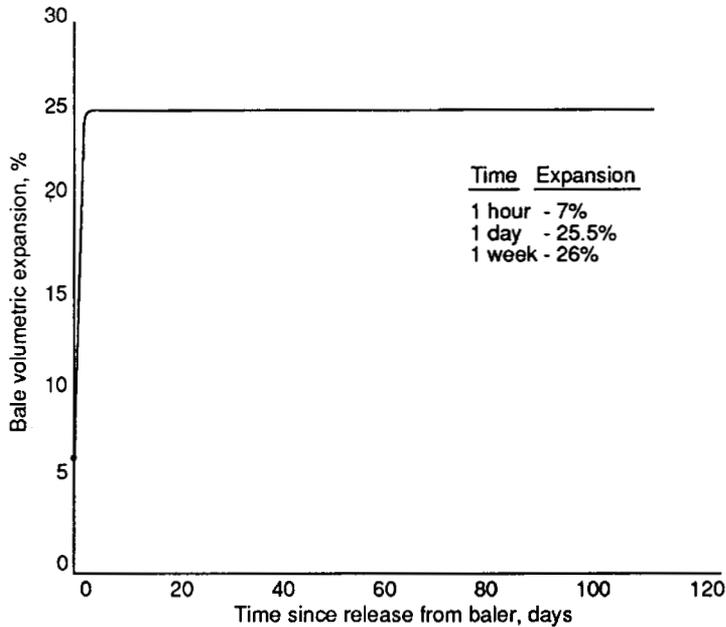


FIGURE 8.28 Bale “spring-back” characteristics.

are very effective on “white” goods, such as stoves and refrigerators, and other ductile types of materials (sheet metal, etc.).

All bales will experience a spring-back effect or expansion of the bale upon being released from the baler. Most of the expansion will occur rather immediately, but final volume may take a week or more to reach. Figure 8.28 shows typical volume change as a function of time after being released from the baler. The engineer should contact the various manufacturers for specific lines of solid waste balers.

Balefill. Disposal of solid waste bales (in a balefill) is an easier material-handling task to the extent that many operators of balefills claim that baling is more economical. Transportation of bales to the final disposal site as well as handling at the disposal site requires not only less sophisticated (and therefore less expensive) equipment but also requires less equipment in total. Balefills are generally operated by stacking bales end to end and three bales high. A balefill still requires daily soil cover, but due to the geometry of the bale, not only is volume saved but the soil cover volume is also less. Depending upon the original nature of the solid waste, a balefill may occupy as little as 10% of what the same material would occupy if the refuse were disposed in a conventional landfill.

RECOVERY AND REUSE

Recovery and reuse are essential elements of a integrated solid waste management plan. Energy recovery and production offer a direct economic benefit and lessen demands on processing and disposal require-

ments. Similarly, the cost-effective recovery of solid waste components for reuse in industrial production and source reduction techniques are improving overall control of solid wastes.

Pollution prevention or source reduction, including product reuse, is the first priority in an integrated solid waste management program. Source reduction includes the design, manufacture, purchase, or use of materials, such as products and packaging, to reduce either the amount or toxicity of materials before they enter the solid waste management system. Examples of source reduction activities include:

- Modifying residential, commercial, and industrial practices and functions to manage abusive waste of materials
- Managing nonproduct organic wastes, such as food scraps and yard trimmings, through on-site composting or other alternatives to disposal, such as leaving grass clippings on the lawn
- Designing products or packaging to reduce the quantity of materials or to make them easier to reuse
- Improving packaging to reduce the amount of damage or spoilage to products
- Reusing existing products or packaging
- Extending the useful life of products to postpone disposal

Energy from Solid Waste

As early as the turn of the twentieth century, solid wastes were used to produce steam and electricity. The types, quantities, and distribution of solid wastes with fuel potential, particularly urban solid wastes, have received increased attention over the past decade due to rapidly escalating energy costs. Companies or governments investigating converting solid waste into usable energy must evaluate the waste's heating value and competing recovery technologies in estimating the energy recoverable from a waste stream.

Heating Values. The heating value (or energy content) of most solid wastes is roughly one-third to one-half the heating value of coal. However, there are many ways heating values are reported, so caution is necessary when comparing and evaluating such data.

The quantity of heat generated by complete combustion of a fuel is known as the heating value, heat of combustion, or caloric value. The heating value of a fuel may be determined directly by measurement of the heat evolved during combustion of a known quantity of the fuel in a calorimeter, or it may be estimated from the chemical or physical analysis of the fuel and the heating value of the several chemical elements or physical components (28).

The higher heating value (HHV), or gross heating value, of a fuel is determined when the water vapor in the products of combustion of a fuel is condensed, and the latent heat of vaporization of the water is included in the heating value of the fuel. Conversely, the lower heating value (LHV), or net heating value, is obtained when the latent heat of vaporization of the water vapor is not included in the heating value of the fuel (28).

European practice is to use the LHV, while in the United States the heating value of a fuel specified is generally the HHV. For most solid wastes, the HHV will range from 7 to 15% more than the LHV due to hydrogen content variations, or (29).

$$\text{Rough HHV} = 1.11 \text{ LHV}$$

Heating values are usually expressed in units of Btu per cubic foot (kilocalories per cubic meter) for gaseous fuels, Btu per gallon (kilocalories per liter) for liquid fuels, and Btu per pound (kilocalories per kilogram) for solid fuels. The heating value of solid fuels may be reported four ways: as-received, dry, ash-free, or dry and ash-free (28). Therefore, it is vital that both the presenter and user of data properly qualify and understand the units applied to a given set of numbers.

Solid Waste Fuels. The heating value of nearly all solid waste fuels is a function of carbon content. Ash content is generally low, but the amount of moisture is highly variable and depends upon moisture generation plus the effects of processing, handling, and storage. On a dry, ash-free basis the heating value can be estimated at 8000 Btu/lb (4400 kcal/kg); more resinous materials contain about 9000 Btu/lb (5000 kcal/kg). Table 8.41 lists the characteristics of some typical solid waste fuels.

Municipal solid waste and most other solid waste fuels can be burned without auxiliary fuel in their raw, as-received state, over a rather wide range of compositions. In fact, solid waste fuels containing as much as 75% moisture and ash can be burned without auxiliary fuels (31). However, since water and noncombustible materials do not contribute to the heating value of the waste, handling and storing wastes to minimize their moisture content and processing wastes to reduce their ash content can greatly improve fuel quality.

Municipal Solid Wastes. The heating value of municipal solid waste (MSW) is dependent upon its composition. Typical heating values of MSW as received at a resource recovery facility or disposal site range from approximately 3000 to 6000 Btu/lb (1700 to 3300 kcal/kg). This variation can be attributed to seasonal factors that influence the moisture and noncombustible content of the waste. However, the average value is typically 4500 to 5000 Btu/lb (2500 to 2800 kcal/kg) as received. Heating values are discussed further in the section Refuse-Derived Fuels.

The heating value of a waste stream can be estimated if the waste stream composition is known. Various waste stream components have the following heating values (32):

Paper	7,750 Btu/lb	4,300 kcal/kg
Plastic	18,000 Btu/lb	10,000 kcal/kg
Wood	8,000 Btu/lb	4,400 kcal/kg
Other organics	2,000 Btu/lb	1,100 kcal/kg

Therefore, an example of how the heating value of a waste stream is approximated is indicated below (32):

TABLE 8.41 Characteristics of Solid Waste Fuels (30)

Solid Waste Fuel	Higher heating value, Btu/lb dry, ash-free	Moisture, % as received	Ash, % dry
Black liquor (sulfate)	6,500	35	40–45
Cattle manure	7,400	50–75	17
Coffee grounds	10,000	65	1.5
Corncobs	9,300	10	1.5
Cottonseed cake	9,500	10	8
Municipal solid waste	9,000	20–50	20–40
Pine bark	9,500	40–50	5–10
Rice straw or hulls	6,000	7	15
Scrap tires	16,400	0.5	6
Wheat straw	8,500	10	4

Note: 1.0 kcal/kg = 1.8 Btu/lb.

Typical waste stream	Heating value	
	Btu/lb	kcal/kg
Paper, 35%	$7750 \times 0.35 = 2712$	$4300 \times 0.35 = 1505$
Plastic, 10%	$18,000 \times 0.10 = 1800$	$10,000 \times 0.10 = 1000$
Other organics, 10%	$2000 \times 0.10 = 200$	$1100 \times 0.10 = 110$
Total	<u>4712</u>	<u>2615</u>

This estimating procedure for energy potential is very useful in evaluating resource recovery alternatives. Also, it can be used to approximate the impact of source separation and other recycling activities, or other changes in waste composition on the energy content of the waste stream.

Energy Recovery Technologies. The major technologies used to recover energy from MSW include

- *Waterwall incineration:* Combustion of unprocessed MSW (mass burning) or processed MSW in a furnace with integral boiler tubes.
- *Modular incineration:* Combustion of MSW in relatively small two-stage, starved-air furnaces with heat recovery boilers or heat exchangers.
- *Refuse-derived fuels:* A variety of technologies that produce solid fuel by processing MSW into combustible and noncombustible fractions. The resulting fuel can be cofired with fossil fuels or burned alone in a “dedicated boiler.”
- *Pyrolysis:* A variety of technologies that process MSW in an oxygen-deficient environment to produce gaseous, liquid, and/or solid fuels.
- *Anaerobic digestion:* A developmental technology adapted from anaerobic digestion of wastewater sludges.
- *Landfill gas recovery:* Collection of gas generated during decomposition of landfilled MSW.

Energy Recovery Efficiencies. Currently there is no standard, accepted way to evaluate the energy recovery efficiency of resource recovery systems. There are several major contributing causes for differing efficiencies, including

1. Alternative ways of treating energy used by the process itself
2. The choice of system boundaries for which the calculation is made
3. The use of higher (HHV) or lower heating value (LHV) of the waste
4. Including or excluding the energy content of nonfuel materials

Under current conditions, it is possible to produce energy recovery efficiency figures to either enhance or detract from the apparent attractiveness of a particular system (33).

Table 8.42 shows system energy efficiencies in terms of the energy content of the fuel produced, and in terms of the output energy available as steam and electricity. The energy efficiencies in this table are “net system outputs,” and, are based on the HHV of the product minus the energy used to operate the recovery system divided by the HHV of the input waste. The energy available as steam and electricity was calculated using boiler and turbine efficiencies appropriate for each recovery system (33).

While comparison of energy recovery technologies on the basis of available steam and/or electricity makes thermodynamic sense in terms of standard system boundaries, it ignores important characteristics of the various waste-derived fuels such as the quality of the fuel product and its transportability, which directly affect fuel economics (33).

Estimating Recoverable Energy. The quantity of energy that can be recovered from MSW depends on four factors:

- | | | |
|-------------------------|---|---------------------|
| 1. Waste input quantity | } | Thermal output |
| 2. Waste energy content | | |
| 3. Boiler efficiency | } | Thermal utilization |
| 4. System availability | | |

The thermal output of a recovery system is the product of the waste input quantity, waste energy content, and boiler efficiency. The quantity of waste generated or collected must be adjusted to reflect the recovery facility's operating schedule (If x tons are collected in 5 days and the recovery facility operates 7 days, the daily waste tonnages used for estimating energy recovery is $5x/7$). Generally, the energy content of MSW is approximated at 4500 Btu/lb (2500 kcal/kg) for unprocessed MSW and up to 8000 Btu/lb (4400 kcal/kg) for RDF.

Boiler efficiency is greatly influenced by the moisture content of the fuel. Typical boiler efficiencies are 60 to 70% for mass burning and 70 to 75% for RDF combustion. A more detailed discussion of waste energy content is provided in the next section, Refuse-Derived Fuels. The nomograph in Figure 8.29 provides a method to estimate the thermal output of a combustion system. For example,

Unprocessed MSW: If waste input is 500 tons/day, energy content 4500 Btu/lb, and at 60% boiler efficiency, then the thermal output is 1.1×10^8 Btu/h for 24 h/day.

RDF: If waste input is 2000 tons/day, and the fuel yield is 1400 tons/day, energy content is 6000 Btu/lb, and at 75% boiler efficiency, then the thermal output is 5.2×10^8 Btu/h for 24 h/day.

The system availability is simply the percentage of the time the system is functional. System availability depends on the recovery system chosen and the steam quality produced (see Figure 8.30). The availability of RDF firing units is typically higher than mass burning facilities. However, for RDF systems with minimal storage capacity, the system availability must take into account the reliability of the processing equipment and the firing units.

Thermal utilization is the product of boiler efficiency and system availability and represents the fraction of solid waste heat content converted to steam on an annual average basis (36). Estimation of potential revenues from energy recovery projects should be based on thermal utilization, not thermal output.

Figure 8.31 relates the thermal output of the incinerator to quantities of steam and electric energy produced. In general, steam of any pressure and temperature can be produced up to practical limits of about 1200 lb/in² and 900°F (8.3 MPa and 480°C). Above these conditions, it has been determined that corrosion mechanisms accelerate due to increased temperatures within the combustion chamber, leading to excessive deterioration (40).

The selection of steam pressure and temperature is largely based on market demand. For example, saturated steam of 250 lb/in² (1.7 MPa) might be produced to supply a chemical process plant, food processing plant, or district heating system; if electric generation were desired, steam might be produced at 850 lb/in² and 750°F (5.9 MPa and 400°C), resulting in a higher energy content of each pound of steam, and thereby reducing the size of electric generating equipment required (40).

For example, to determine potential steam generation, enter Figure 8.31 at 5.2×10^8 Btu/h incinerator output of Figure 8.29. Move vertically upward to steam conditions of either 250 lb/in² saturated, or 850 lb/in² at 750°F, for steam production of 443,000 lb/h and 498,000 lb/h, respectively (40).

To determine the potential electricity generated by the steam, move horizontally to the right from 498,000 lb/h to the turbine exhaust conditions of 6 in Hg absolute and 2 in Hg absolute. These conditions correspond with reasonable average turbine exhaust back pressures for air-cooled condensers and water-cooled surface condensers, respectively. Now, read vertically to determine potential power generation of 49,000 kW for the water-cooled condensers (40). Using air cooled condensers would reduce the power generation approximately 10%.

The preceding discussion provides methods for estimating the heating value and the energy recoverable from municipal solid wastes. These topics are expanded upon in the next section.

TABLE 8.42 MSW Energy Recovery System Efficiency Comparison

Recovery system and fuel classification [†]	Efficiency, %*									
	MSW to fuel [†]			MSW to steam			MSW to electricity			
	Ref. 33	Ref. 34	Ref. 35	Other	Ref. 33	Ref. 34	Ref. 35	Ref. 34	Ref. 34	Ref. 39
Waterwall incinerator										
Mass burning (RDF-1)	100	100	100	—	59	63	60	29	19	
Shredded MSW (RDF-2)	—	—	81	—	—	—	52	—	23	
Modular incinerator (RDF-1)	100	—	100	—	25—50	—	54	—	—	
Refused-derived fuels										
Fluff (RDF-3)	70	76	75	—	49	53	48	24	18—24 [‡]	
Dust (RDF-4)	80	80	—	68(36)	63	63	—	—	—	
Wet	76	76	—	—	48	43	—	—	—	
Pyrolysis										
Gaseous products (RDF-7)	6,4—84	65—74	—	—	42—58	46—58	—	19—23	—	
Liquid product (RDF-6)	26	33	—	—	23	23	—	13	—	
Anaerobic digestion (RDF-7)	33	—	—	—	29	—	—	—	—	
Landfill gas recovery (RDF-7)	—	—	—	10—40 (38)	—	—	—	—	—	

*The energy efficiencies are “net system outputs,” as discussed in the text.

[†]Discussed further in the section Refuse-Derived Fuels.

[‡]Cofired with coal.

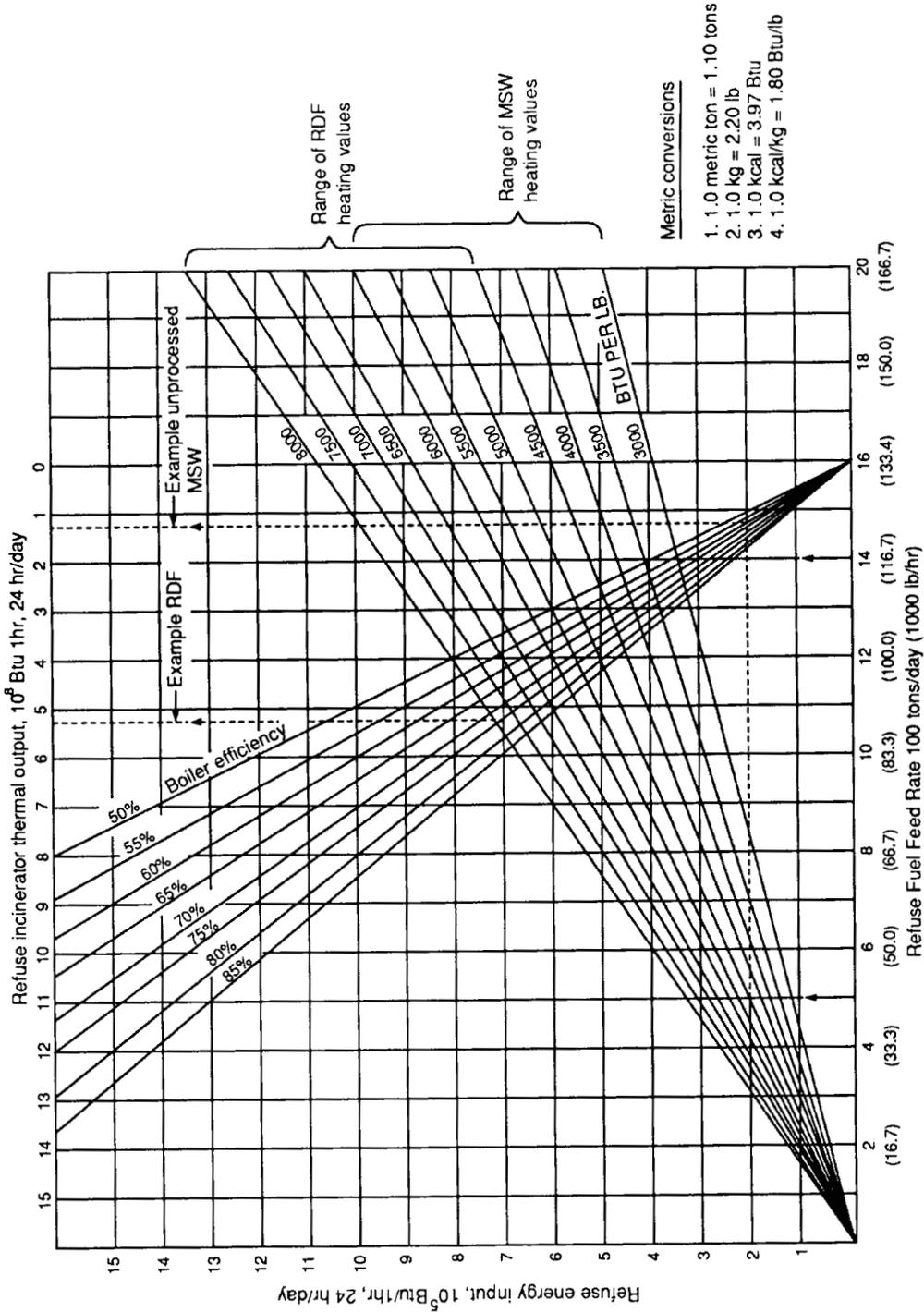


FIGURE 8.29 Waste input and thermal output (40).

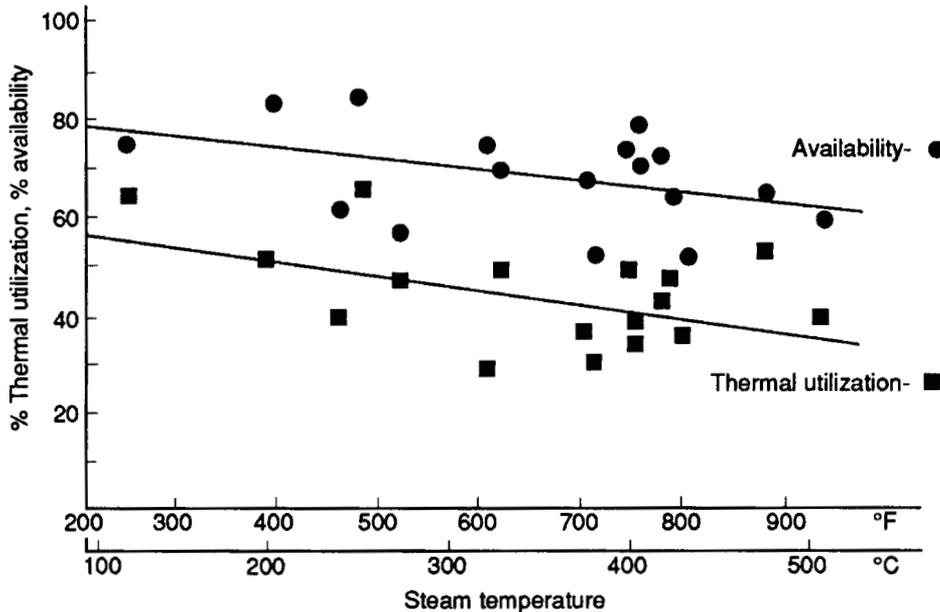


FIGURE 8.30 Thermal utilization and availability of waterwall incinerators (37).

The planning and implementation phases of a successful energy recovery project must address many topics beyond the scope of this discussion. System economics and community acceptance are typically the concerns that most frequently determine if a technically viable project is implemented.

Refuse-Derived Fuel

Utilizing MSW as fuels accomplishes two purposes—significant volume reduction and energy production. The basic technology for producing energy from MSW, incineration and steam turbine electrical generation, was developed during the early twentieth century. Over the past 15 years, new technologies have been developed to recover materials from MSW while also producing an improved fuel product, generically described as refuse-derived fuel (RDF). Some of these new technologies produce fuels compatible with existing power generation facilities or other energy users.

Municipal solid waste can be processed to concentrate the combustible components into an RDF by removing glass, ceramics, and metals. The resulting RDF will typically have an HHV of 4500 to 8000 Btu/lb (2500 to 4500 kcal/kg) (wet). RDF typically burns more efficiently than unprocessed MSW due to lower ash content, lower moisture content, and a higher degree of homogeneity. Because of these qualities, RDF requires less excess combustion air than mass burning processes. This in turn reduces the required size of all air pollution controls and air handling equipment (40, 41).

The removal of inert materials has the added advantage of producing more easily handled fuel, since the glass, stone, and metals can contribute to the deterioration of materials handling equipment. However, RDF production typically involves sophisticated integrations of many materials handling operations. Also, detailed market and waste stream evaluations are required in order to determine that technologies are appropriate in each specific situation (40).

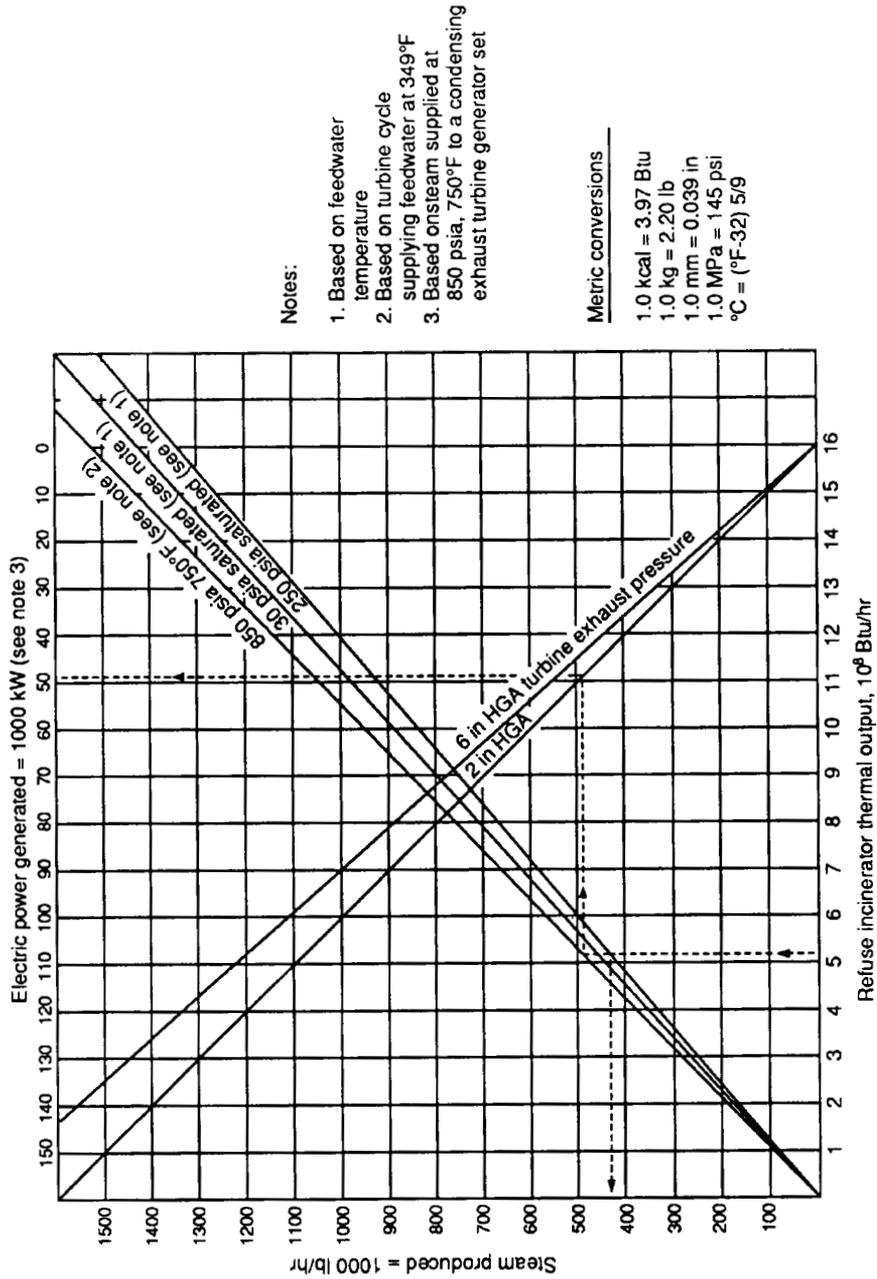


FIGURE 8.31 Thermal output, steam production, and electrical power generation (40).

The combustible components can be further processed to produce gaseous or liquid fuels via pyrolysis or anaerobic digestion.

Fuel Product Classification. The need for uniform definitions, specifications, and analytical procedures for producers and users of recovered commodities from refuse prompted the formation of ASTM Committee E-38 on Resource Recovery (44). This Committee has developed standard classifications for refuse-derived fuels (Table 8.43). ASTM has also developed a standard test method for gross calorific value (E 711-81) and a standard method for RDF data conversion (E 791-81).

RDF production processes and final fuel products have also been described by the terms in Table 8.44.

Heating Value. Typical heating values, ash contents, and moisture contents of prepared fuels are summarized in Table 8.45. These typical values illustrate that processing MSW can significantly lower ash content and moisture content as well as concentrating the waste's combustible components. However, processing can also significantly reduce the energy obtained from each ton of MSW due to controllable, but inevitable, losses of combustible materials. Typically, RDF contains 85 to 99% of the paper and plastics in the raw waste, while the majority of glass and metals are removed. These losses are reflected in process yields (weight percent) and energy yields, expressed as fuel Btus per pound (kilocalories per kilogram) of MSW.

Typical process yields and energy yields of fuel products are also summarized in Table 8.45. The data in Table 8.43 represent general ranges expected for prepared fuels. For example, the heating value of RDF-1 can range from 3000 to 6000 Btu/lb (1650 to 3300 kcal/kg) due to waste composition and moisture and ash variations, but a good average range that can be expected currently is 4500 to 5000 Btu/lb (2500 to 2750 kcal/kg) (46). The ranges presented are also affected by data availability, which varies with fuel product; the data bases for RDF-1 and RDF-3 are large, while data for the other products are somewhat scarce.

The heating value of prepared solid fuels generally increases from RDF-1 through RDF-4. The heating value of RDF-5 is comparable to or slightly higher than that of RDF-3 because RDF-3 is typically used to produce RDF-5.

The HHV of prepared fuels correlates with the fuel ash content and moisture content because ash and moisture contribute little, if any, heating value to the fuels. The strongest correlation exists between heating value and ash content plus moisture content. Figure 8.32 presents this relationship based on limited pub-

TABLE 8.43 ASTM Classifications of Refuse-Derived Fuels (44)

Category	Description
RDF-1	Municipal solid wastes (MSW) used as a fuel in as-discarded form [without oversize bulky waste].
RDF-2	MSW processed to coarse particle size with or without ferrous metal separation. [The particle size of this material is such that 95 weight percent passes through a 6-in (15-cm) square mesh screen.]
RDF-3	Shredded fuel derived from MSW that has been processed to remove metal, glass, and other inorganics. This material has a particle size such that 95 weight percent passes through a 2-in (50-mm) square mesh screen.
RDF-4	Combustible waste processed into powdered form-95 weight percent passing 10-mesh screening (0.035 in or 0.89 mm).
RDF-5	Combustible waste densified (compressed) into the form of pellets, slugs, cubettes, or briquettes.
RDF-6	Combustible waste processed into liquid fuel.
RDF-7	Combustible waste processed into gaseous fuel.

Note: [] indicate wording in tentative classifications deleted from definitions.

TABLE 8.44 Common Refuse-Derived Fuel Classifications

Common name	Typical ASTM class
Coarse fluff RDF	RDF-2
Fine fluff RDF	RDF-3
Powdered RDF	RDF-4
Densified RDF or d-RDF	RDF-5

lished data. This correlation provides an easy way to estimate RDF heating value using the results from two easily performed tests.

Municipal solid wastes can be processed into a variety of fuel products to improve their heating value, combustion efficiency, storage life, and marketability. However, the disadvantages of solid waste processing must be considered, particularly energy yield reduction and increased system complexity. The overall impact of energy yield reduction is demonstrated in the energy recovery efficiency comparison presented in the section Energy from Solid Wastes.

Ferrous Metals

There are several sources of scrap ferrous metals: from iron and steel production (home scrap), from conversion of steel to various metal parts and products (prompt industrial or processing scrap) and from discarded goods from end users (obsolete or postconsumers scrap). Of these three types of ferrous scrap, home scrap makes up 55 to 60% of the total. Prompt industrial and obsolete scrap account equally for the remaining portion. Virtually all home scrap and very large fractions of prompt industrial scrap are recycled directly within the production and conversion industries.

TABLE 8.45 Characteristics of Refuse-Derived Fuels

Fuel product	HHV, Btu/lb*	Moisture content, %	Ash content, %	Process yield, %	Energy yield, Btu/lb MSW	Reference sources
RDF-1	4500–5000	15–30	24	90–100	4000–5000	44–46
RDF-2	4600–4680	26–29	20–21	80–95	3680–4400	44, 45, 47
RDF-3	4800–6500	18–25	7–21	50–85	2970–4510	36, 44, 45–48
RDF-4	7500–8000	3–5	12–15	40–50	3120–3940	36, 48
RDF-5	5760–6640	15–23	9–23	†	†	42, 43
RDF-6	8490 (94,000 Btu/gal)	14.1	2.1	23	1950‡	45
RDF-7	270–320 Btu/ft ³	6.0	†	†	3380	45

*Unless otherwise noted.

†Not available.

‡Energy yield does not include energy in charcoal by-product.

Notes: 1.0 kcal/kg = 1.8 Btu/lb; 3.8 L = 1.0 gal; 28.3 L = 1.0 ft³.

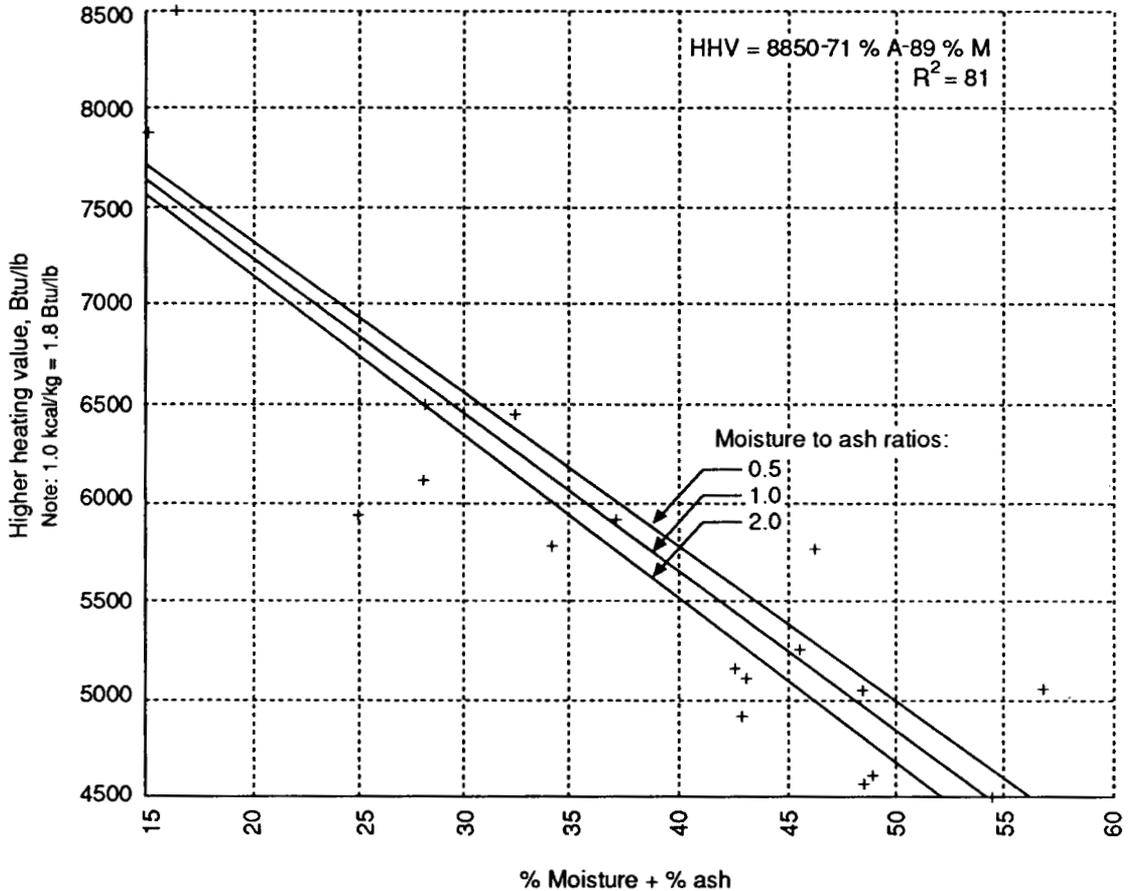


FIGURE 8.32 Higher heating values of prepared fuels versus moisture and ash content.

The categories of obsolete or postconsumer scrap most obvious to the public are automobiles and discarded ferrous metals in municipal solid waste. Spurred by economic incentives of increasing scrap prices, shredding and magnetic processing of junk automobiles has increased over the last decade. In 1980, a projected 35% of the recycled obsolete scrap came from junk automobiles. An estimated 10 million tons of ferrous scrap made up largely of steel cans was discarded in the municipal solid waste stream in the United States in 1981. In this same period, there were 48 source separation programs collecting ferrous metals and 27 facilities that had mechanized systems for separation of ferrous metals from municipal solid waste. A combined total of some 140,000 tons or 1.5% of the available ferrous cans were recycled.

Magnetic Separation. Magnetic separation of ferrous metals from solid waste is one of the most developed and simplest material separation processes in resource recovery. However, for magnetic separation to be effective, unprocessed waste must first undergo processing—generally size reduction or screening—to free the metal from bags and containers that would inhibit or prevent separation. In addition, it appears that shredding of the ferrous metals is necessary—either as part of the mixed waste or the recovered product—to work the metal to free attached or entrapped nonmetallic contaminants.

There are several types of magnetic separators that have been applied to solid waste: suspended drum magnets, suspended self-cleaning belt magnets, and magnetic pulleys. The suspended belt separator has multiple magnet assemblies over a single belt for the purpose of tumbling, dropping, and reattracting the metal to free loose contaminants.

Magnetic Pulley Separators. Pulley-type magnets have diameters between 1 to 4.5 ft (0.3 to 1.5 m) and widths of 1 to 6 ft (0.3 to 1.8 m) to match the conveyor belt on which they are mounted. Pulley-type magnets are generally not applied to primary separation of ferrous metal from solid waste because of the tendency to entrap and carry over significant amounts of organics with the ferrous product. They have, however, found application in secondary scalping of small amounts of ferrous metals in waste streams following primary magnetic separation. Pulley-type magnet drive rotational speed and power requirements will be determined by the design and capacity of the belt conveyor on which it is mounted. Pulley-type magnets normally contain permanent magnets that have no external power requirements.

Suspended Drum Separators. Suspended drum magnets range in diameters of 3 to 6 ft (1 to 1.8 m) and widths of 4 to 8 ft (1.2 to 2.4 m). The size is selected to match the width of the stream of material fed to the separator, the material height and particle size, and the field strength requirements (and thus the magnet assembly size) to achieve the required performance. Rotational speeds match the feed stream velocity and are generally in the range of 5 to 30 r/min. Magnet power requirements would be 3.8 to 11.3 kW for the drive motor and 7 to 18 kW for the magnet power supply.

Typical application and performance have been reported on two suspended drum magnets in a solid waste processing plant in New Orleans, Louisiana (4). One drum is designed for 95% ferrous recovery from a 36.5 metric ton per hour stream of minus 4 in (100 mm) trommeled undersize solid waste product. It is 3.5 ft (1.07 m) in diameter and 4.5 ft (1.37 m) in width, rotates at 28 r/min (3.8-kW drive) and contains electromagnetics with a 7-kW power supply. The drum is clefted and rotates against material flow. Testing at the design throughput of 36.3 metric tons per hour and a larger-than-design gap between the belt and drum surfaces—15 versus 12 in (0.38 versus 0.29 m)—the recovery efficiency of the magnet tested at 90%.

The second suspended drum separator was designed to recover 95% of the ferrous metals from a 75 ton (68 metric ton) per hour shredded solid waste stream. The drum is 4 ft (1.22 m) in diameter, 6 ft (1.83 m) wide, rotates at 25 r/min (7.5-kW drive) and powered by a 11-kW supply. Because of significantly higher and more variable feed material burden height than predicted, the gap between the drum and belt was raised 20 in (0.53 m) compared to the 14-in (0.36 m) design gap. Ferrous metal recovery efficiency was thus substantially below design testing at only 28%.

The location of the magnetic separator varies with the type of waste processing system. The primary magnet is normally located immediately after the initial shredding or screening equipment. Secondary magnets are often utilized to retrieve the 10 to 20% of the ferrous metals missed in the primary stage.

Experience suggests that it is necessary to magnetically separate the metal product a second time or possibly reshred or air-classify after separation to free and separate nonmetal contaminants. The contamination level for the suspended drum product in the New Orleans facility discussed above averaged 15%, well above the 4% specification and necessitating the addition of a ferrous product cleanup system. This system consisted of an air knife, light ferrous metal shredder, and a secondary suspended-belt-type magnetic separator. Performance tests indicated the ability to meet the 4% contaminant specification but indicated a reduction of 37% in the amount of the recovered ferrous product due to losses in the secondary processing.

Reshredding as part of secondary processing will also increase bulk density. A high bulk density, 1700 to 2500 lb/yd³ (800 to 1200 kg/m³) is desirable for transportation of the ferrous metal product and is required for some scrap markets. The secondary cleanup system in New Orleans increased the density of the recovered ferrous product from an as-separated density of 450 lb/yd³ (210 kg/m³) for partially crushed cans to 820 lb/yd³ (380 kg/m³) for the coarsely shredded cans.

Separation of ferrous metals from incinerator residue is generally more efficient than from processed waste because of the absence of organic materials that inhibit separation or contaminate the product. However, because of oxidation and alloying of metals and nonmetals during the combustion process, incinerated

ferrous scrap may not meet some industry specifications. In 1981, nine incinerator facilities were reportedly separating ferrous metals, although not all were able to find markets for the recovered product.

Markets. The potential markets from ferrous metals recovered from municipal solid waste would be reached through brokers, detinners, or scrap dealers and include iron and steel mills and foundries, the detinning industry, the copper industry (for use in precipitation processes) and production of ferroalloys. Each market has different requirements for chemical and physical properties of scrap. Standard specifications for municipal ferrous scrap covering each of these end uses are shown in Tables 8.46 and 8.47.

Contracts for purchase of municipal ferrous scrap typically establish the price as a percentage of the iron and steel industry composite prices, such as No. 1 heavy melting steel scrap or No. 2 bundles. The percentage factor will vary for the particular scrap quantity composition, location (transportation), and the

TABLE 8.46 Municipal Ferrous Scrap Chemical Specifications (48)

Element	Composition, % ^a				
	Copper industry precipitation process)	Iron and steel foundries	Iron and steel production	Detinning industry ^b	Ferroalloy production
Phosphorus, max	—	0.03	0.03	—	0.03
Sulfur, max	—	0.04	0.04	—	—
Nickel, max	—	0.12	0.08	—	—
Chromium, max	—	0.15	0.10	—	0.15
Molybdenum, max	—	0.04	0.025	—	—
Copper, max	—	0.20	0.10	—	0.20
Aluminum, max	—	0.50	0.50	4.00 ^e	0.15
Tin	—	0.30 max ^d	0.30 max	0.15 min ^f	0.30
Lead, max	—	0.03	0.15	—	—
Zinc, max	—	0.06	0.06	—	—
Iron (metallic), min	96.0	—	—	—	—
Silicon, max	—	—	0.10	—	—
Manganese, max	—	—	—	—	0.35
Carbon, max	—	—	—	—	0.6
Titanium, max	—	—	—	—	0.025
Total combustibles, max	0.2 ^c	4.0	4.0	—	0.5 ^g
Metallic yield, min	—	90.0	90.0	—	90.0

^aExperience has shown that material that has been incinerated probably will not meet these requirements.

^bA minimum of 95 weight percent of the material delivered shall be magnetic. Nonmagnetic material attached to the original magnetic article may be included in the minimum requirement.

^cThe scrap shall be appropriately processed (for example, by burning or chemical detinning) to be virtually free of combustibles.

^dFor steel castings, the requirement for tin content is 0.10 max percent.

^eNot based on melt analyses due to aluminum losses during melting; to be determined by a method mutually agreed upon between the purchaser and supplier.

^fRefer to sections on magnetic fraction and chemical analysis of tin in Methods E 701. Normal separation of white goods and heavy iron yields tin contents equal to or greater than 0.15 weight percent. Lesser tin contents would impact severely the value of the scrap to detinners.

^gThe scrap shall be appropriately processed (for example, by burning or chemical detinning) to be virtually free of combustibles.

respective market. Historically, the market prices for iron and steel scrap are quite erratic. This has made projections of revenues for recovered metal very tenuous, and during some periods of depressed markets, made ferrous scrap virtually impossible to sell at any price. Nonferrous Metals Nonferrous scrap is classified into three types: home scrap, prompt industrial scrap (new scrap), and old scrap (obsolete or postuser scrap). Most home and prompt industrial scrap is recycled internally by primary or secondary producers. The sources of old nonferrous scrap is mainly junk automobiles and municipal solid waste. The major nonferrous metals recovered from auto shredders include zinc, aluminum, copper, and stainless steel. MSW is a potentially large source of erratic. This has made projections of revenues for recovered metal very tenuous, and during some periods of depressed markets, made ferrous scrap virtually impossible to sell at any price.

Nonferrous Metals

Nonferrous scrap is classified into three types: home scrap, prompt industrial scrap (new scrap), and old scrap (obsolete or postuser scrap). Most home and prompt industrial scrap is recycled internally by primary or secondary producers. The sources of old nonferrous scrap is mainly junk automobiles and municipal solid waste. The major nonferrous metals recovered from auto shredders include zinc, aluminum, copper, and stainless steel. MSW is a potentially large source of nonferrous scrap, particularly aluminum. Approximately 1% of the MSW is nonferrous metals of which two-thirds is aluminum and the remainder primarily brass, copper, zinc, and stainless steel. During the 1970s, efforts at recycling of nonferrous metals from municipal solid waste were directed at recovery of aluminum.

In 1979, an estimated 1.15 million tons (1 million metric tons) of aluminum containers and packaging were available in MSW as old scrap for recycling in the United States. Of this, about 65% were all-

TABLE 8.47 Municipal Ferrous Scrap Physical Specifications (48)

End use	Property	
	Bulk density, lb/ft ³ (kg/m ³)	Form
Copper industry (precipitation process)	30 (480) max	Loose, shredded as agreed upon between purchaser and supplier, shall not be balled or baled ^a
Iron and steel foundries	50 (800) min	Loose, balled, or baled ^b as agreed upon between purchaser and supplier
Iron and steel production	75 (1200) min	Loose ^c or baled ^b as agreed upon between purchaser and supplier
Detinning industry	25 (400) max	Shredded, 95 weight percent shall be $-6, +\frac{1}{2}$ in ($-152, +12.5$ mm); shall not be balled, baled, burned, incinerated, or pyrolyzed
Ferroalloy production	50 (800) min	Loose, as agreed upon between purchaser and supplier

^aVarious consumers may establish gage limitations on the material they purchase.

^bIndustry practice is to specify a maximum bale size that may vary among users.

^cExperience has shown that if the size range is 95 weight percent, $-2, +\frac{1}{4}$ in. ($-50, +6.3$ mm), the bulk density requirement can be met and the material will be loose and free flowing.

aluminum cans. In 1981, some 30% of all-aluminum cans were recycled, virtually all through source separation programs.

Source separation refers to separation of selected components from solid waste at the point of discard and transfer of the separated fractions (by the homeowner or by separate collection) to collection centers, secondary dealers, or direct consumers of secondary materials. The separation is done by hand and involves limited use of mechanical equipment. In 1980, there were over 2500 locations in the United States for recycling of source-separated aluminum cans.

Mechanical Separation. Mechanical methods for nonferrous metal recovery include flotation (density), electromagnetic separation, electrostatic separation, and preconcentration and handpicking. A description of each method follows.

Flotation. Flotation systems utilize stages of controlled water elutriation (for effective specific gravities between 1.1 and 2.0), heavy liquids (for specific gravities of 1.5 to 3.0), and dense media (water slurries of magnetite, galena, or ferrosilicon for effective specific gravities of 2.5, 3.3, and 3.5, respectively). In systems applied to MSW, several flotation stages with specific gravities between 1.2 and 3.0 are used to separate glass, aluminum, and other nonferrous metals by differences in material densities.

Typically size reduction, air classification, magnetic separation, and screening precede a combination of water flotation and heavy media separation. Proper feedstock preparation is necessary to control the size and shape of the metals and to minimize the levels of organic and inorganic fines. Shredded aluminum with folds and pockets may entrap air, which bouys the particles, or entrap dense solids that settle and increase particle weight and media losses and product contamination. Organic materials can adsorb heavy liquids and media increasing operating costs. Organics and fine inorganics can change media specific gravity and viscosity thereby effecting separation efficiency.

Water elutriation and heavy media separators, in conjunction with magnets and screens, have also been used for recovery of nonferrous metals from incinerator residues in pilot-scale and experimental programs (51). Because nonferrous metals such as aluminum may oxidize, melt, and be lost in the undergrate siftings, or alloy with other materials during incineration, sufficient marketable nonferrous metals are usually not recoverable to justify commercial applications.

Electromagnetic. Electromagnetic or eddy current separators employ the principle of electromagnetic induction to separate conductive nonferrous metals. Utilizing modulating electromagnetic fields or the motion of the metal moving through the magnetic field of an array of permanent magnets, eddy currents are generated in conductive metal particles that in turn interact with the magnetic field and cause the particles to be deflected out of the separator.

The flow scheme for the electromagnetic nonferrous separation system in the New Orleans, Louisiana, facility is shown in Figure 8.33. This particular system utilizes two parallel electromagnetic separators operating on a feedstock from a series of screens, classifiers, and magnets designed to preconcentrate aluminum can stock. The importance of efficient preconcentration is indicated in results from this demonstration facility. Although the aluminum recovery efficiency for the electromagnetic separators was as high as 85 to 90%, because of losses of aluminum in preceding screening and classification processes, the corresponding overall aluminum recovery rate for the facility averaged 35 to 40%.

Each electromagnetic separator has four sets of magnets, each set having a magnet above and below a 22-in (0.56-m) wide conveyor belt. The gap between the top and bottom magnet was nominally 3.5 in (90 mm). The belt carried 1 to 3 tons/h (0.9 to 2.7 Mg/h) of waste at a belt speed of between 250 to 500 ft/min (75 and 150 m/min). Conductive nonferrous metals concentrated in the feedstock to about a 5% level are ejected laterally off both sides of the belt by the electromagnets. The magnets operate at 480 V and 60 cycles with each set consuming about 7.5 kW.

The effect of aluminum shape and belt speed on electromagnet separation efficiency is shown in Table 8.48. For nominal operation, each separator recovered between 85 and 90% of the aluminum from a feed stream of 1 ton/h (0.9 Mg/h) and belt speed of 350 ft/min (105 m/min).

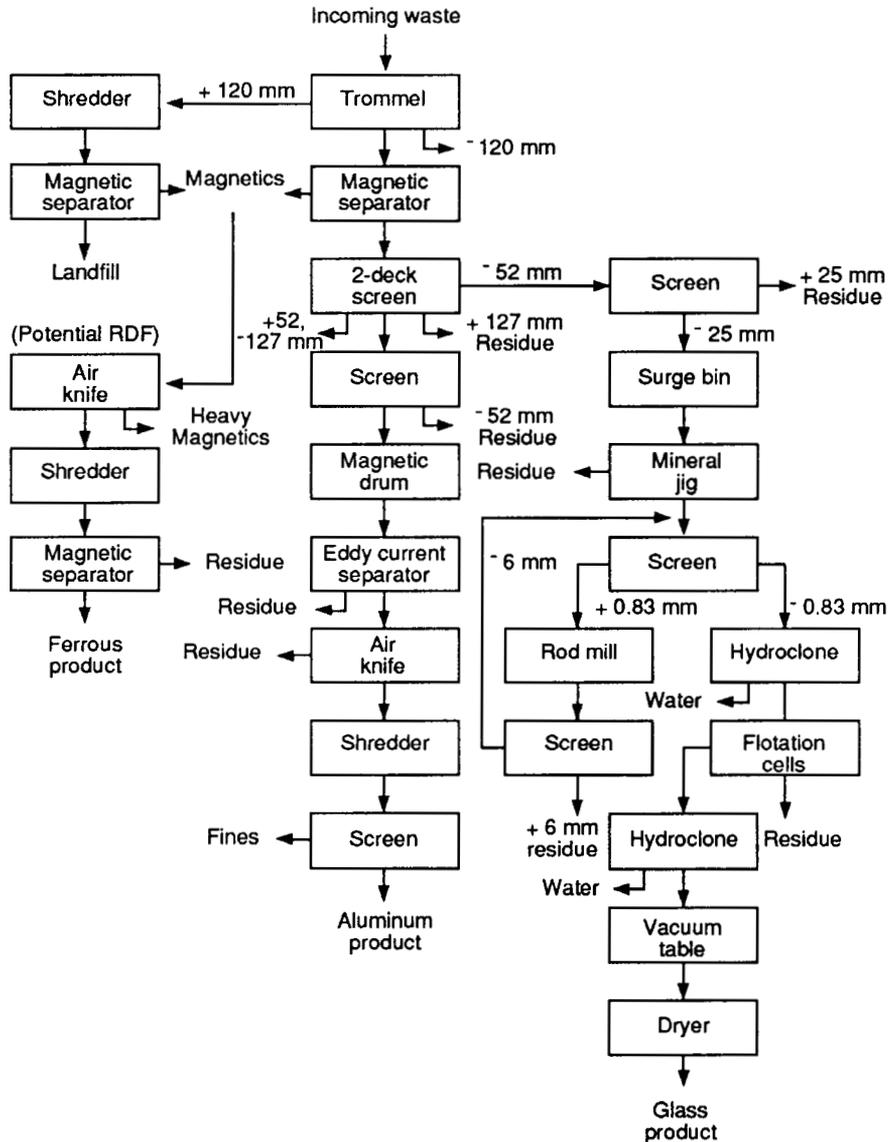


FIGURE 8.33 Ferrous, aluminum, and glass recovery processes at New Orleans.

Electrostatic. Electrostatic separators utilize a 30 to 50 kV field generated from electrodes located above a stream of particles as they flow onto a grounded metallic drum. The drum is usually 10 to 30 in (0.25 to 0.75 m) wide and 1.5 to 3 ft (0.5 to 1 m) in diameter. The nonconductors (typically glass and organics) retain a static charge long enough to be attracted and held to the drum, while conductors (metals) dissipate their charge quickly and are repelled from the drum and thereby separated. Processing of the electrostatic separator feedstock to even a greater degree than electromagnetic feedstock is necessary. The feed must be quite

TABLE 8.48 Effect of Can Shape and Belt Speed on Electromagnetic Separator Performance (49)

Belt speed m/min	Percent recovery by can shape			
	Flattened	Deformed	Whole	Total
91	97.7	98.4	100	98.4
122	92.9	98.4	100	98.0
152	90.7	95.0	86.6	94.5

Note: 1 m/min \times 3.3 = ft/min.

small [< 1 in (25 mm)] and very dry ($< 1\%$ moisture), and the level of organics (paper) kept at a minimum to avoid interferences and minimize separator volumetric loadings. These requirements have limited widespread application of electrostatic separators to nonferrous metal recovery. The application of an electrostatic separator for metal separation in a pilot glass recovery facility is discussed in Ref. 52.

Depending mainly on market specifications for the metals, the product from an electromagnetic or electrostatic separator may require additional processing to remove glass and loose organic contaminants carried with the metal product and to separate other nonferrous metals from aluminum. In the New Orleans facility, such nonmetal contaminants averaged 14% of the separator product. This cleanup may be accomplished with an air knife, screen, and/or additional stages of electromagnetic (or heavy media) separators. The aluminum product may also require shredding or baling to increase density for economic transportation. Shredding in New Orleans raised the density from 130 to 520 lb/yd³ (60 to 240 kg/m³).

Friction Slide. The first commercial preconcentration and handpicking system for separation of aluminum from mixed MSW is located in Houston, Texas. The process flow is shown in Figure 8.34. The plant is designed to process 60 tons/h (56×103 kg/h) of unprocessed MSW.

The friction slide used in the Houston plant exploits the differences in bounce and frictional resistance with an inclined flat-belt conveyor fed near the lower (tail) end. More rigid or round particles (metals, dense organics) will bounce, slide, or roll off the bottom of the slide and are conveyed to an air knife and handpicking while more flexible, damp material (paper, textiles, organics) is carried up and discharged as a residue off the top. Human pickers remove aluminum from a slow-moving belt and the product is then flattened and shipped to market.

Mechanical recovery of nonferrous metals from MSW has been discontinued at several plants and is not being applied in newer installations for several reasons: relatively low and fluctuating concentrations of metals in the waste, current absence of efficient and reliable technology for separation and sorting of mixed metals, and lack of widespread markets.

Product Quality. A specification for municipal aluminum scrap has been developed. It covers two classes (based on fines content) and six grades (based on chemical composition). Table 8.49 presents the chemical and physical requirements for municipal aluminum scrap. The market value and reuse of the recovered scrap (for example, as aluminum can stock or in wrought alloys) will depend on grade quantity and location of markets (transportation). Use of recycled aluminum does provide significant energy savings. Less than 5% of the energy required to produce an aluminum ingot from ore is needed to produce aluminum ingot from recycled metal.

Glass Products

With the exception of glass scrap (cullet) generated in glass manufacturer conversion plants, municipal solid waste offers the largest source of waste glass for recycling. On the average, 10 to 11% of the MSW stream is

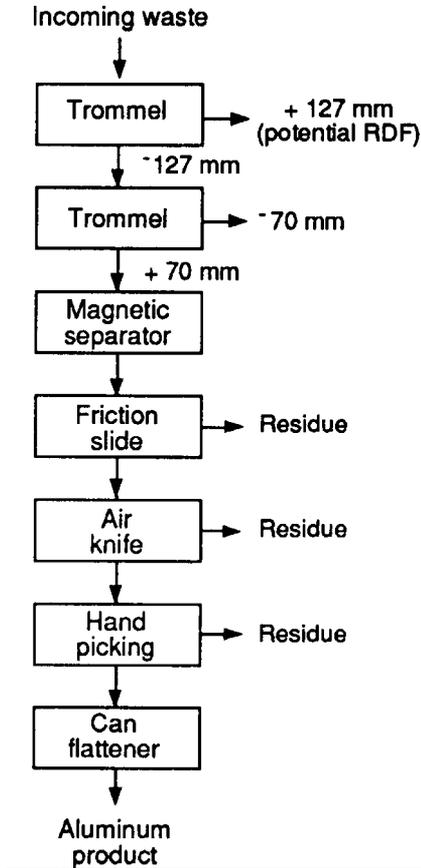


FIGURE 8.34 Ferrous and aluminum recovery processes at Houston.

glass, of which an estimated 90% is container glass. The raw materials for glass production are comparatively inexpensive and readily available in most areas. However, there are incentives for recycling glass in the reduction in energy and water consumption and improvements in the melt reaction for glass manufacturing and also in reduction in solid waste disposal requirements.

In the 1980s nearly all of the recovered and recycled glass from MSW was recovered through source separation programs, where the householder segregates the glass for separate collection or for delivery to a recycling center. However, during the 1970s, several approaches to mechanical separation of glass from mixed municipal waste were developed and applied in commercial-scale facilities.

The technology for mechanical recovery of glass cullet from mixed municipal waste involves either froth flotation or optical sorting. Due to relatively low product values coupled with the technical and operational complexities, none of the four first-generation froth flotation or optical sorting systems have operated regularly, and further commercialization of glass recovery has not extended beyond these earlier plants. Equipment and markets for recovery and use of glass for secondary applications (i.e., fiberglass, aggregate, glassware, architectural treatments) has also been explored in one full-scale facility.

TABLE 8.49 Municipal Aluminum Scrap Specification (50)

Chemical requirements						
Composition, maximum % allowable						
Element*	Grade 1	Grade 2	Grade 3	Grade 4	Grade 5	Grade 6
Silicon	0.30	0.30	0.50	1.00	9.00	9.00
Iron	0.60	0.70	1.00	1.00	0.80	1.00
Copper	0.25	0.40	1.00	2.00	3.00	4.00
Manganese	1.25	1.50	1.50	1.50	0.60	0.80
Magnesium	2.00	2.00	2.00	2.00	2.00	2.00
Chromium	0.05	0.10	0.30	0.30	0.30	0.30
Nickel	0.04	0.04	0.30	0.30	0.30	0.30
Zinc	0.25	0.25	1.00	2.00	1.00	3.00
Lead	0.02	0.04	0.30	0.50	0.10	0.25
Tin	0.02	0.04	0.30	0.30	0.10	0.25
Bismuth	0.02	0.04	0.30	0.30	0.10	0.25
Titanium	0.05	0.05	0.05	0.05	0.10	0.25
Others (each)	0.04	0.05	0.05	0.08	0.10	0.10
Others (total)	0.12	0.15	0.15	0.20	0.30	0.30
Aluminum	Balance	Balance	Balance	Balance	Balance	Balance

Physical requirements						
Density—To be agreed upon between purchaser and seller						
Fineness—Class A—Not less than 1 weight percent fines minus 12 mesh						
Class B—Not less than 3 weight percent fines minus 12 mesh						
Loose combustibles—Not more than 2 weight percent loose combustible material						
Moisture—Not more than 0.5 weight percent moisture						
Metal recovery—Minimum metal recovery of 85% (per ASTM procedure)						
Magnetics—To be agreed upon between purchaser and seller						

*By agreement between the purchaser and the seller, analysis may be required, and limits established for elements or compounds not specified in this table.

Froth Flotation Separation. Froth flotation is a process developed in the minerals industry when fine-sized particles [0.002 to 0.35 in (0.05 to 9 mm)] are selectively floated to the surface of a slurry by means of attached air bubbles. A surface conditioning agent (for example a coca amine) that preferentially coats glass particles and makes the surfaces hydrophobic (water repellent) is mixed with water and added to a pre-concentrated and finely sized (less than 20 mesh) fraction containing the glass and inorganic contaminants (ceramics, stones, and metal). The treated slurry is then aerated in a separation chamber where the glass is raised by attachment of air bubbles effected by the reagent and removed from the nonglass contaminants, which are separated in the tailings (sink) product. Additional secondary cells may be used to reclean the product or attempt to raise more glass from the tailings. A complete discussion of the froth flotation process is provided in Refs. 53 and 54.

Figure 8.35 shows the process flow for the prototype froth flotation glass recovery system located in a resource recovery plant in New Orleans. Also shown is the mass balance data for the glass fraction based on a municipal waste input of 100 tons/h (90 metric tons/h) or about 160% of the facility design capacity. The specifications for the principal equipment items are provided in Table 8.50.

The performance characteristics of the glass recovery system revealed during the shakedown and testing in New Orleans were as follows:

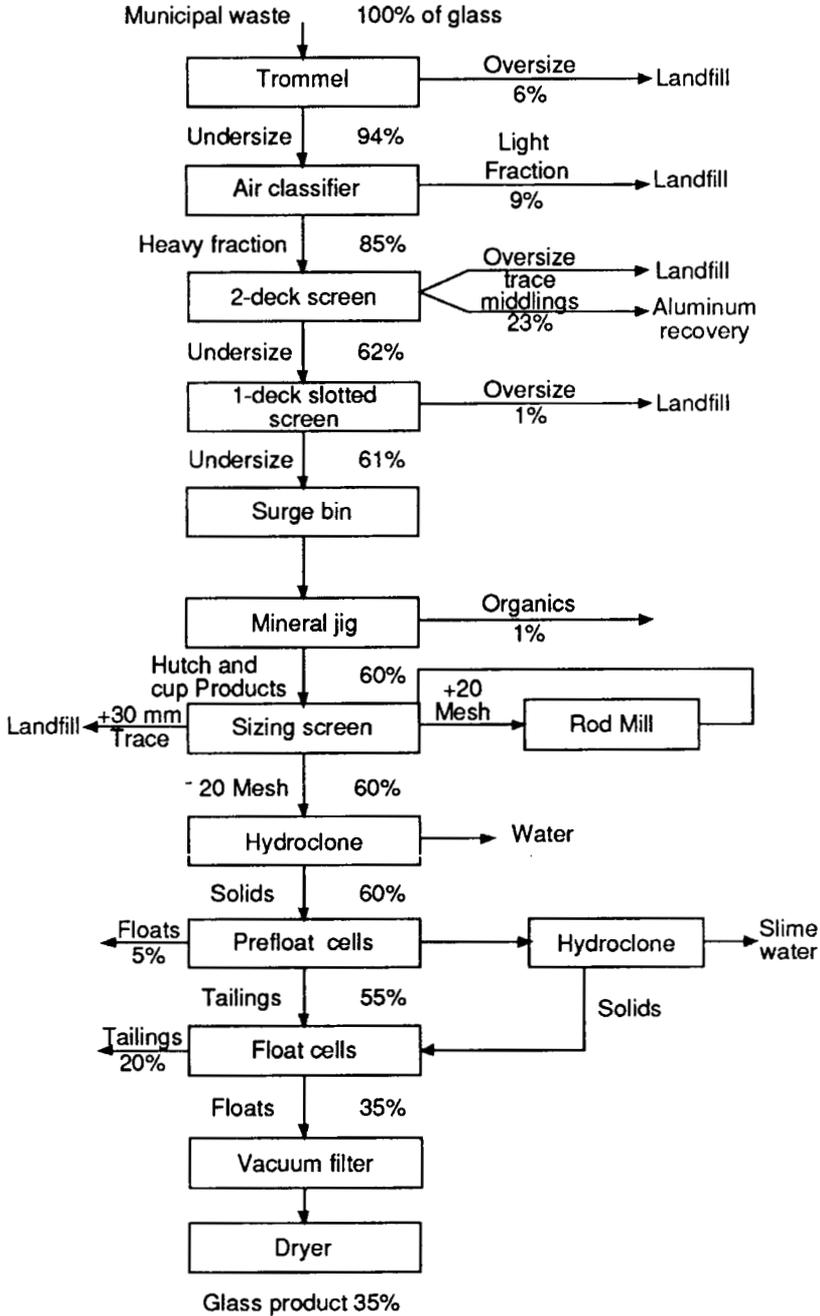


FIGURE 8.35 Nominal glass mass balance at New Orleans glass recovery system (49).

TABLE 8.50 New Orleans Glass Recovery Equipment Descriptions (49)

Item	Description
Two-deck screen	CE Tyler Model F-800; 3 m long by 1.5 m wide; 5° declined; 1.5-kW electromechanical drive; 114- and 57-mm-diameter perforated screen decks
One-deck screen	Vibraretics, 3 m long by 0.6 m wide; 0.38-kW electromechanical drive; 26 mm × 75 mm slotted screen deck
Mineral jig	WEMCO Remer jig; 4.9 m long by 1.5 m wide; 6-mm mesh screen panels; 9.5 m and 13 mm ragging; 5° declination; 1.5 and 5.6-kW drives
Rod mill	Marcy Mfg., 1.2 m diameter by 2.4 m long; center overflow discharge type; 37 grinding rods (typical); 25 r/min; 37.5-kW drive
Sizing screen	CE Tyler Model 1F-800; 3 m long by 1.5 m wide; 1.5-kW electromechanical drive; 6-mm and #20 wire mesh screen decks
Hydroclones (2) Flotation cells	Krebs Models D10B and D6B (modified); 254 and 152 mm diameters, adjustable apex Denver Equipment Model 18-Special, Type A; 0.67-m ³ cells arranged as two prefloat, three rougher, three cleaner
Flotation reagent	Sherex MG 83A at 5% concentration with 1% (wt) concentration pine oil frothing aid
Vacuum filter	Dorr Oliver; 1.2 m diameter; 430 mmHg vacuum
Dryer	Joy Manufacturing Holoflite Model I-D-2410; 1.25 MBtu/h oil fired heater for 650°F operation; twin revolving flights

- Glass losses in the dry portion of the processing system (prior to the mineral jig) were sensitive to the level and fluctuations on infeed rate, and ranged from 40 to 50%.
- The capacity of the “wet” portion of the glass recovery system (the mineral jig through flotation) was 4 tons/h (3.6 metric tons/h); however, it was operated at 1.7 tons/h (1.5 metric tons/h). Increased or fluctuating feed rates were detrimental to system performance and glass product quality.
- Glass recovery in the wet portion of the glass recovery system tested at 80% of the input glass.
- Prefloat cells containing water in reagent were found necessary to float and remove grease and other organic materials carried over from the jig.
- The vacuum filter lowered moisture content of the glass product to 5% and the dryer to less than the 1% required in the glass product specification.

Table 8.51 lists the glass product specifications as established in ASTM E708-79, Standard Specification for Waste Glass as a Raw Material for Manufacturer of Glass Containers, as well as the mean results of analysis of the glass product from New Orleans. A comparison shows that the glass product met the specifications in all categories except in moisture and minus 140 mesh fines.

Pilot tests on a froth flotation of glass incinerator residue has been reported (51) but no commercial application has developed.

Optical Separation. In an optical sorter, the intensity of light transmitted through a particle passing through the device is measured by a photocell. By means of sensing or comparing the intensity to a background slide, the particle is identified as opaque (nonglass) or flint, green, or amber and accepted or diverted by a rapidly acting ejector located below the sensor. Because most sorters are binary devices, several stages may be required to make these multiple separations.

Optical sorting also requires considerable processing to prepare the appropriate feedstock. Clean, dry particles sized in the range of 0.25 to 2 in (6 to 50 mm) and as free of metal and other contaminants as possible are required. Particles smaller than 0.25 in (6 mm) cannot be recovered with this equipment, which eliminates shredders or flails in the feed preparation system. Alternative size reduction schemes might em-

TABLE 8.51 Glass Specifications and Product Analysis Result from New Orleans (55)

Parameter	ASTM Specification Value	Mean recovery I result
Percent moisture, wet-weight basis	<0.5%	1.0
Percent plus 6-mm, dry-weight basis	0%	0
Percent minus U.S. std. #140, dry-weight basis	≤15%	15.8
Percent organic materials, dry-weight basis	≤ 0.2% or ≤ 0.4 ± 0.05%	0.16
Percent Fe ₂ O ₃ mixed-color glass	>0.1 and/or 0.16	
Percent Cr ₂ O ₃ mixed-color glass	>0.0015 %	Trace
Percent SiO ₂	65–75 %	73.4
Percent Al ₂ O ₃ soda-lime	1–7	2.0
Percent CaO + MgO glass	9–13%	10.3
Percent Na ₂ O	12–16%	13.5 %
Total magnetic materials mixed-color glass	≤0.14%	—
Percent total inorganic nonmagnetic materials	≤0.6%	0.59
Refractories: number of particles per pound		
Minus #20, plus #40 mesh	≤2	0
Minus #40, plus #60 mesh	≤20	12
Nonmagnetic metals: number of particles per pound		
Plus #20 mesh	≤1	0

ploy primary screening, shear shredders, or hydropulping. Further concentration of the glass can be accomplished by air classification and screens, separation of organics, and washing of the glass by flotation; metal separation by flotation; or electrostatic separation and drying. The complete process flow for the first full-scale commercial application of an optical sorting system in Hempstead, N.Y., is shown in Figure 8.36.

Paper and Plastics

On the average, paper comprises about 35% and plastics about 6% of the United States MSW stream. The proportion of individual paper categories varies considerably with location, but typically newspapers comprise 25%, corrugated 35%, and other categories and mixed papers, the remaining 40% of the paper fractions. Of the plastics, polyolefins (including polyethylenes and polypropylenes) account for 75% of the plastics with styrene polymers contributing 15% and polyvinylchlorides (PVCs) about 10%.

In spite of their relatively high concentrations, mechanical separation of either paper or plastics from mixed MSW in the United States has not developed beyond limited research or pilot-scale programs. The significant quantity of paper that is being recycled [18 million tons (16.2 million metric tons) or 26% of the total U.S. consumption of paper and paperboard in 1981] is virtually all from commercial and residential source separation prior to discard into the waste stream. In 1981, there were 228 residential source separation programs recycling paper, of which three-quarters collected newspaper and one-quarter collected mixed waste paper. Source-separated waste paper is primarily used to make new paper and paperboard products (86%), with the remainder used for such products as cellulose insulation and building products (3%) or exported (11%).

Mechanical recycling of paper and plastics from mixed wastes has been limited by both technical and economic factors. For paper, testing has shown that the yield of recycled fiber is very low (less than 30% of

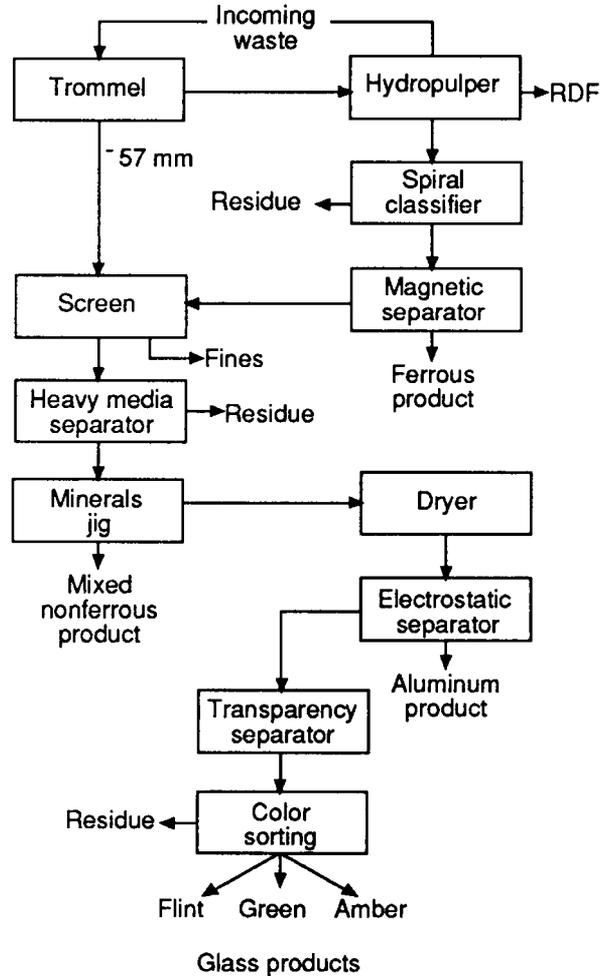


FIGURE 8.36 Ferrous, aluminum, mixed nonferrous, and glass recovery process at Hempstead, N.Y.

the available paper fiber) due to losses in the dry-screening and air-classification stages required to concentrate the fiber fraction and further fiber losses in the pulping, centrifugal cleaning, screening, and dispersion systems required to remove contaminants. No compositional data on production material are yet available to establish the markets and thus the economic viability of paper recovery.

Recycling of plastics from mixed municipal waste is complicated by lower concentrations, a variety in sizes, density, and other properties (melting points), and frequent lamination with other plastic and nonplastic material. The technologies that have been explored for concentration of a mixed-plastic product from waste include multiple stages of air classification, thermal contraction of film plastics followed by air separation, and use of electrostatic separators.

Even were a clean mixed-plastic product recovered, economic reuse requires further separation of the mixed product by plastic (or resin) type. Surface wetting and density separation techniques have been ex-

ploded but not demonstrated on a large scale. The potential for reuse of mixed plastics is primarily with the thermoplastics fraction that can be remelted. Thermoplastics comprise some 80% of the plastics in the waste, while the remaining 20% are thermosetting plastics “set” after forming. Markets for reformed thermoplastics need to be sought.

TREATMENT AND DISPOSAL

Following separation, including by-product recovery, and volume reduction processes, solid wastes are prepared for treatment (stabilization) and final disposal. The most common method is by use of a landfill. Other methods of interest are land reclamation, composting, incineration, pyrolysis, and wet oxidation.

Whether reclaiming a strip mine or developing a recreation area, the engineer must consider costs and design limitations and subsequent maintenance by good soil management practices. In general terms, the principal considerations are land requirements, site soil and topographic conditions, site access, and land and operating costs. In some cases, ground and surface water monitoring may be required during and after the disposal process.

Sanitary Landfill

Sanitary landfilling is defined as “an engineered method of disposing of solid waste on land in a manner that protects the environment, by spreading the waste in thin layers, compacting it to the smallest practical volume, and covering it with compacted soil by the end of each working day or at more frequent intervals if necessary” (56). Sanitary landfills are and will continue to be the principal method of solid waste disposal.

In the United States, the design and operation of sanitary landfills are regulated by 40 CFR 258, which establishes minimum national criteria for all solid waste landfills that are not regulated under Subtitle C of RCRA and that receive municipal solid waste, or codispose sewage sludge with municipal solid waste, or accept nonhazardous municipal waste combustion ash. The U.S. Environmental Protection Agency provides extensive technical guidance to meet the regulation requirements (94, 95). Landfills that receive construction and demolition debris only, tires only, and nonhazardous industrial waste only are regulated under 40 CFR 257.

This section outlines the salient points that must be considered in the design of a sanitary landfill by the civil engineer. Because of the volatile nature of construction and equipment costs, the cost figures given in this section should be used as a guide only. Also, heavy equipment used in sanitary landfill operations is being improved and revised continually. Engineers who need to determine sanitary landfill equipment requirements should contact equipment suppliers and observe equipment used by existing facilities in the area.

Preliminary Determination of Landfill Requirements. Every proposed sanitary landfill has some basic requirements that must be determined prior to commencing any design activity. Typical determinations follow.

Estimate of Solid Waste Quantity. If accurate records are not available on the amount of solid waste generated in an area, then the amount may be estimated by one of the following methods.

1. The population to be served by the sanitary landfill must be determined. This population is multiplied by 5 lb (2.26 kg) per capita to arrive at an estimated daily weight of solid waste generated. The generation rate per capita estimate can be broken down as half residential and half commercial and industrial.
2. In communities with very large commercial or industrial establishments, it is prudent to estimate the large establishment's per capita solid waste generation. If this figure is significant, an adjustment in the per capita figure can be made.

3. A fairly accurate estimate of the amount of solid waste can be determined by counting the types of vehicles entering the existing landfill (or landfills) that will be replaced by the new facility. Table 8.52 lists typical unit weights for various collection vehicles.

Estimate of Landfill Space Required. The annual landfill space requirements can be determined by the following formula.

$$V_A = W/1100$$

where W is the annual weight in pounds (kilograms) of solid waste generated in the area, and 1100 is the refuse compaction in pounds per cubic yard (650 kg/m^3).

Landfills should be designed for a minimum depth of solid waste of 20 ft (6 m) and a minimum life of 10 years.

Type of Waste Delivered to Site. A survey should be made to determine if any nontypical solid waste is to be delivered to the proposed site. Particular attention should be paid to wastes that will require special handling by the operating personnel. It is also important to identify any potential hazardous wastes that presently are accepted at the existing sanitary landfill. Hazardous wastes should be excluded from sanitary landfills.

Traffic on Site. It is necessary to know how solid waste will be delivered to the site. The most important determination is the percentage of vehicles that will be off-loaded by hand. It can take 30 mm to unload a pickup truck with 300 lb (90 kg) of solid waste and 5 mm for a rear-load packer with 10,000 lb (3000 kg). Unloading times for all vehicles must be kept to a minimum.

Requirements of Landfill Operator. The operator of the sanitary landfill might have requirements that will affect the design. The anticipated hours of operation and number of days per week the site will be open must be determined. The operator might also have special equipment that he or she wants to use in the operation.

Requirements of State Regulatory Agency. Every state has its own set of landfill standards. These standards must be fully understood prior to undertaking any investigation. Many states have laws dealing with developments in wetlands, floodplains, or other types of environmentally sensitive areas. The regulating agency for these requirements might be different than the landfill regulating agency. Also, certain federal regulatory agencies may have jurisdiction.

Site Selection. Selection of a sanitary landfill site is often more a social-political process than an engineering process. The selection process should involve the evaluation of at least two potential sites within the study area. A typical site-selection scenario would be as follows.

Accumulate Available Data. Land-use maps, topographic maps, water well logs, soil conservation service soil maps, highway maps, and bridge loading information should all be used in the study.

Location Restrictions. In consideration of both the potential effects that a sanitary landfill may have on

TABLE 8.52 Typical Unit Weights for Various Collection Vehicles

Type of vehicle	Unit weight	
Car or pickup	200 lb	(91 kg each)
Rear-loading packer	550–820 lb/yd ³	(320–487 kg/m ³)
Side-loading packer	450–700 lb/yd ³	(267–415 kg/m ³)
Top loading packer	400–500 lb/yd ³	(237–297 kg/m ³)
Compacted roll-off container	450–550 lb/yd ³	(267–326 kg/m ³)
Open top roll-off container	250–400 lb/yd ³	(148–237 kg/m ³)

the surrounding environment and the effects that natural and human-made conditions may have on the performance of the landfill, location restrictions apply (94). Floodplains, fault areas, seismic impact zones, and unstable area restrictions address conditions that may have adverse effects on landfill performance that could lead to releases to the environment or disruptions of natural functions, such as floodplain flow restrictions. Airport safety, floodplains, and wetlands criteria are needed to the location of landfills in areas where sensitive natural environments and/or the public may be adversely affected.

Establish Minimum Site Size. The geometry of a site is very important. Required setbacks from roads and other natural features make portions of a site unusable. A square site maximizes the amount of land available for actual solid waste disposal activity.

The fixed cost of engineering, land, roads, buildings, and environmental controls for a site with a 20-year life are not much different than for a site with a 10-year life.

Obtain Soil Borings. Soil borings are required on the most desirable of the sites. Permission to take these borings from private landowners is sometimes difficult to obtain. For some sites, this dilemma can be overcome by requesting permission from the local road agency to take borings in the public right-of-way fronting the site.

Prepare Budgetary Cost Estimate. Budgetary cost estimates must be prepared for each of the selected sites. The cost estimates must be of sufficient detail to allow a comparison among the various sites. The capital cost items must be on an annual basis. Items included in the cost estimates should be

- Land cost
- On-site development costs (roads, fences, leachate control, liners, etc.)
- Off-site costs (bringing access roads up to anticipated load-carrying capacity)
- Cost of closing the site when it is filled
- Cost of perpetual care for the site, including the transportation and treatment of any leachate
- Anticipated annual operating cost

Select the Most Desirable Site. Barring any political or social constraints, the site to be recommended should be the one with the lowest cost (disposal plus trucking) per ton.

The landfill evaluation, design, and approval process can be a long and drawn-out ordeal with often negative results. For these reasons, long-term real estate options instead of an outright land purchase is desirable. A typical option would require the payment of a nominal sum during the option period (normally one year) with the balance to be paid only upon receipt of a sanitary landfill operating license.

Hydrogeologic and Soils Investigation of Selected Site. Prior to the detailed design of a sanitary landfill, a hydrogeologic and soils investigation of the site is performed. The state regulatory agency normally has some specific criteria for performing this task. The investigation should include as a minimum the following.

Topographic Map. A topographic map at a sufficient scale and contour interval so that the character of the study area is clearly defined should be prepared. Convenient scales are 1:1200 (1 inch = 100 ft) horizontal and 2 or 4 ft vertical contour interval.

Soil Borings. Soil borings should be taken at intervals and depths such that the nature of the soil strata can be determined. Soil samples should be collected at 5-ft (1.5-m) intervals. These samples should be saved for testing. When groundwater is encountered, it is important that several soil borings completely penetrate the aquifer and its aquiclude. The driller must accurately record the depth at which groundwater is first encountered. If possible, the hole should be left open several hours and a second measurement of groundwater depth taken. The elevation of the ground at each boring location should be determined to the nearest 0.15 ft (5 cm).

Existing Water Well Logs. Water well logs for wells drilled in the surrounding area should be collected from the appropriate agencies (normally a public health department). The well logs will provide some indication of the location, depth, capacity, and water quality of the aquifer.

Set Groundwater Observation Wells. A series of groundwater observation wells [2 in (5 cm) diameter] should be established around the perimeter of the site. These wells should be set in porous soils and penetrate the groundwater at least 10 ft (3 m). A survey crew should establish the top elevation of the well to the nearest 0.01 ft (0.5 cm). The cap of the well should be vented and provided with a locking device to prevent tampering. A sufficient number of wells (three minimum) must be set to accurately determine the slope and direction of groundwater flow. The wells can also be used as groundwater sampling points.

Groundwater Contour Map. The groundwater elevations recorded from the observation wells should be plotted on the site topographic map. A groundwater contour map should be superimposed on the site topographic map.

Soil Tests. It is important that tests be run on

- Clay soil that will serve as an in situ impermeable barrier or be used to construct a clay liner. Hydrometer, Atterberg limits, and permeability tests should all be run on selected clay samples.
- Saturated porous soils that require a grain size analysis.

Groundwater Recharge Area Impact. The impact the proposed landfill will have on the recharge capability of the groundwater aquifer must be determined. A large lined landfill might remove sufficient recharge area from the groundwater system to alter the character of the aquifer.

The landfill site investigation process is a progressive one. It must be designed to minimize the cost and allow the owner and/or regulatory agency an opportunity at each step to review the accumulated data. The hydrogeologic and soils study, which is the first actual on-site operation, must be sequenced to allow the engineer the opportunity to abort the project if conditions other than those originally anticipated are encountered.

Water Quality Control Requirements at Selected Site. Since leachate is by far the primary source of wastewater of concern at a landfill, the spatial and hydrogeological characteristics of the selected site are important. Also, the selected sites proximity to wastewater collection and/or treatment facilities needs to be determined.

In addition to leachate, other sources of landfill wastewaters are: gas collection condensate, truck/equipment washwater, drained free liquids, laboratory wastewaters, and contaminated stormwater. Additional sources of wastewaters generated by landfills may include contaminated groundwater, noncontaminated stormwater, and sanitary wastewaters. These wastewaters are described below.

- *Leachate* is liquid that has passed through or emerged from solid waste and contains soluble, suspended, or miscible materials removed from such waste. Over time, the potential for certain pollutants to move into the wider environment increases. As water passes through the landfill, it leaches pollutants from the disposed waste, moving them deeper into the soil. One measure used to prevent the movement of water pollutants from the landfill site is a liner integrated with a leachate collection system. Leachate also may be collected through the use of slurry walls, trenches or other containment systems.
- *Gas collection condensate* is liquid that has condensed in a gas collection system during the extraction of gas from the landfill. Gases, such as methane and carbon dioxide, are generated due to microbial activity within the landfill and must be removed to avoid hazardous conditions. The gases tend to contain high concentrations of water vapor that is condensed in traps staged throughout the gas collection network. The gas condensate contains volatile compounds and accounts for a relatively small percentage of flow from a landfill.
- *Drained free liquids* are aqueous wastes drained from waste containers, such as drums and trucks, or wastewater resulting from waste stabilization prior to landfilling. Landfills that accept containerized waste may generate this type of wastewater. Wastewaters generated from these waste processing activities are collected and usually combined with other landfill generated wastewaters for treatment at the wastewater treatment plant.

- *Truck/equipment washwater* is generated during either truck or equipment washes at landfills. During routine maintenance or repair operations, trucks and/or equipment such as loaders, compactors, or dump trucks used within the landfill are washed and the resultant wastewaters are collected for treatment. In addition, many facilities wash the wheels, body, and undercarriage of trucks used to deliver the waste to the open landfill face upon leaving the landfill.
- *Laboratory-derived wastewater* is generated from on-site laboratories that characterize incoming waste streams and monitor on-site treatment performance.
- *Contaminated stormwater* is runoff that comes in direct contact with the waste or waste handling and treatment areas.
- *Noncontaminated stormwater* includes stormwater that flows off the cap or cover of the landfill and does not come in direct contact with solid waste. Noncontaminated stormwaters discharged through municipal storm sewer systems or that discharge directly to waters of the United States are subject to National Pollutant Discharge Elimination System (NPDES) stormwater permit requirements.
- *Contaminated groundwater* is water below the land surface in the zone of saturation that has been contaminated by landfill leachate.

Preliminary Design. The next major step toward completion of the project is the preliminary design stage. Discussion follows of the major areas that must be investigated during this stage.

Review Accumulated Data. Information gathered during the earlier phase of the project is now accumulated and used to establish some criteria for the site. Examples of data use follow.

- The soils report will indicate the nature of on-site soils and the need for any liners or leachate collection system. Generally, porous soils [permeability greater than 0.00028 ft/day (1×10^{-7} cm/s)] will require a liner system and some type of leachate collection.
- The hydrogeologic report will provide the designer with information as to how deep the site may go below ground surface. The location of permanent groundwater monitoring wells can be determined.
- The site topographic map prepared for the hydrogeologic investigation will be used as the master sheet for the design. It may be necessary to have the map enlarged in order to properly layout all the details of the site. It is always a good idea to make several reproducible copies of this map.
- The original estimate of the site's daily traffic and quantity must be updated.
- The landfill operator's equipment limitations must be fully understood. This item requires special consideration, since a landfill is primarily a materials-handling operation.

Establish Operating and Design Parameters. In general, the standard operating requirements for a sanitary landfill should be developed to ensure the safe daily operation and management at the facility. Daily cover, liners, and leachate controls are common to all landfills. Other operating requirements include (94):

- *Hazardous waste exclusion.* A program must be developed and implemented to detect and prevent disposal of regulated hazardous wastes or PCB wastes at the sanitary landfill. Hazardous wastes may be gases, liquids, solids, or sludges that are listed or exhibit the characteristics described in 40 CFR 261. Household hazardous wastes are excluded from the regulation, and wastes generated by conditionally exempt small-quantity generators (CESQGs) are not considered regulated hazardous wastes for these purposes.
- *Disease vector control.* Disease vectors such as rodents, birds, flies, and mosquitoes typically are attracted by putrescent waste and standing water, which act as a food source and breeding ground. Normally, application of a daily cover is sufficient to control disease vectors; however, other vector control alternatives may be required. These alternatives could include: reducing the size of the working face; other operational modifications, such as increasing daily cover thickness, changing cover type, density, placement frequency, and grading; repellents, insecticides or rodenticides; composting or processing of organic wastes prior to disposal; and predatory or reproductive control of insect, bird, and animal populations.
- *Explosive gas control.* Methane production rates will vary spatially within a landfill unit as a result of

pockets of elevated microbial activity but, due to partial pressure gradients, differences in gas composition are reduced as the gases commingle within and outside the landfill unit. Monitoring is necessary to establish if the concentration of methane gas generated by the facility exceeds 25% of the lower explosive limit (LEL) for methane in facility structures, excluding gas control or recovery system components, and if the concentration of methane gas exceeds the LEL for methane at the facility property boundary. If these concentrations are exceeded, abatement measures are necessary.

- *Air monitoring.* Sanitary landfill operations must not violate any applicable requirements developed under a State Implementation Plan (SIP) developed to comply with the Clean Air Act. Open burning of solid waste, except for the infrequent burning of agricultural wastes, silvicultural wastes, land-clearing debris, diseased trees, or debris from emergency clean-up operations, should be prohibited.
- *Facility access.* The development of landfill facilities should include means to control public access to prevent illegal dumping, public exposures to hazards at the site, and unauthorized vehicular traffic. Generally, unauthorized persons are unfamiliar with the hazards associated with landfill facilities. Access to facilities should be controlled through gates that can be locked when the site is unsupervised.
- *Run-on/run-off control systems.* The stormwater control system must include a run-on control system to prevent flow onto the active portion of the landfill during the peak discharge from a 25-year storm and a run-off control system from the active portion of the landfill to collect and control at least the water volume resulting from a 24-hour, 25-year storm.
- *Surface water requirements.* The operation of the sanitary landfill should not create a point or nonpoint discharge of pollutants to surface waters. Otherwise, abatement measures must be included in the facility design.
- *Liquid restrictions.* Only household liquid wastes and leachate or gas condensate derived from the landfill operation are permissible. All other bulk or noncontainerized liquid wastes should be excluded from the sanitary landfill.
- *Record keeping requirements.* Records should be maintained to document the day to day activities at the sanitary landfill and to provide regulatory agency reporting data.

In addition, operating and design parameters should be established for the following:

- The maximum distance a heavy refuse truck can travel from an all-weather service road must be determined. A good design will minimize the distance a truck must travel over refuse. In wet northern climates the maximum desirable distance a refuse truck should travel over a refuse-filled area is 800 ft (245 m).
- The desired width of the landfill's working face must be determined. A narrow working face minimizes the daily cover required but could also result in excessively long waiting times for the refuse vehicles. If the mix of unloading vehicles contains more than 30% car and pickups, then serious consideration should be given to constructing an on-site transfer station for these hand-unloaded vehicles. Generally, the narrowest possible working face is the least costly to operate and control.
- Landfilled solid waste must be covered with a minimum of 6 in (15 cm) of earthen material at the end of each operating day, or at more frequent intervals if necessary, to control disease vectors, fires, odors, blowing litter, and scavenging. Alternative materials of an alternative thickness may be acceptable if demonstrated that the alternative material and thickness satisfies the daily cover objectives.
- The amount of daily cover must be calculated. The hauling of this cover is an important cost element in the landfill operation. The daily surface area of cover required is calculated by multiplying the width of the working face times the length of the compacted refuse. Compaction equipment requires a length at least equal to 10 times its wheelbase for proper refuse spreading and compacting.
- The cost of moving this daily cover with the operator's available equipment must be calculated. This cost is computed by multiplying the hourly rate for the cover equipment (operator wages included) times the number of hours required to place the cover. Hourly equipment rental rates are available from most heavy construction equipment dealers.
- An estimate of the in-place refuse density must be made. In well-run landfills using modern compaction equipment, an average in-place density of 1100 lb of refuse per cubic yard (650 kg/m³) of landfill volume

is a reasonable estimate. Typically, the in-place density of refuse will range from 950 to 1500 lb of refuse per cubic yard (560 to 890 kg/m³).

- Storm drainage facilities should, as a minimum, be designed to accommodate a 25-year storm. An optimum design would not allow any rainwaters to flow across the landfill work area. Storm drainage for the actual landfill work areas must be designed so that rainwater in contact with the working face and thus requiring possible containment and treatment is not mixed with other stormwaters. Stormwaters falling on disturbed but unfilled areas of the landfill should be kept out of the leachate collection system but should pass through sedimentation controls prior to exiting the site.
- Landfill costs are optimized when the site is designed to accommodate as great a depth of solid waste as possible. This is especially true when expensive liners and leachate collection systems are required. Therefore, the maximum height that the landfill may go above and below natural ground must be established. The below-ground distance is usually determined by the proximity of groundwater or the excavation limitations of available equipment. The above-ground height can often be a volatile social-political issue. It is important that the environmental and cost advantages of placing greater depths of refuse versus using more surface area be clearly documented. When going above ground with a landfill, the completed side slopes should be no steeper than 1 unit vertical to 4 units horizontal. This slope will allow grass mower operation and it will not erode as much as steeper slopes.
- All landfills must allocate some capital for on-site roads, fences, maintenance buildings, utilities, and a gate house. The following discusses some minimum requirements for each of these capital items.

Landfill roads must be of sufficient width and strength to handle the anticipated traffic in all kinds of weather. The biggest item that must be determined is the number of refuse compaction vehicles that will be using the site. These heavy vehicles, when loaded, meet or exceed allowable axle loads. Most state road agencies have established design standards for roads that would be similar in heavy truck traffic to the landfills. These standards should be used for determining the road's structural section. If design standards are not available locally, The Asphalt Institute has an excellent design manual entitled *Thickness Design-Asphalt Pavement Structures for Highways and Streets* (57). It is always a good idea to pave the entrance road, at least to the gatehouse. This paved road, in addition to its strength and smooth riding surface, will also provide a cleaning area for mud-caked tires on the vehicles exiting the site.

The landfill's entrance and those portions of the site fronting public roads should be fenced. Fencing the remainder of the site will depend on terrain, vegetation, population density in the area, and state and local requirements. In heavily populated areas or along high-traffic roads, it is a prudent practice to fence the entire work area.

Landfill equipment costs are the largest single operating cost. Proper maintenance of this equipment requires frequent oil changes, cleaning of the radiators, and cleaning of areas on the machines where oil and flammable debris can accumulate. During winter months and inclement weather, routine maintenance is best performed inside a heated building. A maintenance building helps reduce equipment costs.

An investigation of the availability of utilities is an important first step in any design. Typical questions that must be answered are

- Is natural gas available for building heat?
- Does the available electrical system meet the electrical needs of the pumps, compressors, and other equipment needed to operate the landfill? If the proper voltage is not available, what is the cost to deliver it to the site?
- Is a public water system available or must on-site wells be installed?
- Is a public wastewater system available for domestic sewage or must a septic system be installed?
- Can the public wastewater system accept the high-strength leachate that might be discharged, and if so where is the outlet point?

The type of gatehouse will be dictated by local conditions. The gatehouse should be located at least 200

ft (60 m) from the entrance. This will allow the backup of vehicles during rush hours without spilling out onto the public road.

The need for scales is a much discussed topic. For large sites with over seven years life, the cost of a scale becomes insignificant. The following are several advantages of scales:

1. Scales provide an equitable method for pricing disposal fees.
2. Tons is the accepted unit of measure for solid waste engineering. By weighing the refuse received, the operator is establishing a valuable data base. This data base can be used to compute in-place density and collection truck performance.

Daily Cover Requirements. Daily or more frequent placement of 6 in (15 cm) of cover is necessary to control disease vectors, fires, odors, blowing litter, and scavenging. This temporary cover controls disease vectors (birds, insects, or rodents that represent the principal transmission pathway of a human disease) by preventing egress from the waste and by preventing access to breeding environments or food sources. Covering also reduces exposure of combustible materials to ignition sources and may reduce the spread of fire if the disposed waste burns. Odors and blowing litter are reduced by eliminating the direct contact of wind and disposed waste. Similarly, scavenging is reduced by removing the waste from observation. The depth of cover and/or frequency should be increased to insure these objectives are satisfied (94).

Alternative materials of an alternative thickness may be acceptable if demonstrated that the alternative material and thickness satisfies the daily cover objectives. Demonstrations can be conducted in a variety of ways. For example, demonstrating alternative covers may be accomplished by: side by side (six inches of earthen materials and alternative cover) test pads, full-scale demonstration, and short-term full-scale tests.

Alternative daily cover materials may include indigenous materials or commercially available materials. Indigenous materials are those materials that would be disposed as waste; therefore, using these materials is an efficient use of landfill space. Examples of indigenous materials include (94): ash from municipal waste combustors and utility companies; compost-based material; sludge-based material, such as sludge treated with lime and mixed with ash or soil; construction and demolition debris (which has been processed to form a slurry); and shredded automobile tires.

Commercially developed alternative daily covers are available. Some of these alternative materials require specially designed application equipment, while others use equipment generally available at most landfills. Examples of the types of commercially available daily cover materials are (94):

- Foam that usually is sprayed on the working face at the end of the day
- Geosynthetic products such as a tarp or fabric panel that is applied at the end of the working day and removed at the beginning of the following working day
- Slurry products (e.g., fibers from recycled newspaper, wood chip slurry, clay slurry).

Liner Requirements. The prevailing theory of sanitary landfill design is to prevent water from entering the refuse mass and also to prevent leachate (water in contact with refuse) from entering ground or surface waters. Downward percolation of the leachate can be retarded if the in situ soils for the landfill are clays with a thickness and permeability sufficient to satisfy regulatory requirements. If the in situ soils do not satisfy the requirements, a liner must be designed. Liners may be constructed of either clay, synthetic fabrics (geomembranes), or soil-additive mixtures. Specific comments on each type of liner follow.

Clay liners are constructed by spreading the clay in thin layers and compacting each layer to a predetermined density. The minimum total thickness of these liners is 2 ft (0.6 m). The minimum thickness should be adequate to obtain adequate compaction to meet the hydraulic conductivity requirement, to minimize the number of breaks or imperfections through the entire liner thickness that could allow leachate migration, and to inhibit hydraulic short-circuiting of the entire clay liner layer.

The suitability of a clay for use as a liner is determined by laboratory tests on the clay. These tests are hydrometer, Atterberg limits, permeability, and density. The hydrometer and Atterberg limits tests will serve to identify and classify the soil. Permeability tests are run on disturbed samples of the clay. For each test, the

density of the clay must be determined. The minimum clay liner density will be the density occurring at the desired permeability.

Clay liners are relatively inexpensive to install if the clay is within a reasonable haul distance. The liner provides excellent protection against leachate migration. However, clay liners cannot be installed during wet or freezing weather. In northern climates, the days available for clay placement are limited.

Because of their inherent impermeability, the use of geomembranes in landfill liner construction has increased. Geomembranes are relatively thin sheets of flexible thermoplastic or thermoset polymeric materials. The design of the side slope, specifically the friction between natural soils and geosynthetics, is critical and requires careful review.

Geomembranes are made of one or more polymers along with a variety of other ingredients. The polymers include a wide range of plastics and rubbers differing in properties such as chemical resistance and basic composition (58, 59). The polymeric materials may be categorized as follows:

- Thermoplastics, such as polyvinyl chloride (PVC)
- Crystalline thermoplastics, such as high-density polyethylene (HDPE), very-low-density polyethylene (VLDPE), and linear low-density polyethylene (LLDPE)
- Thermoplastic elastomers, such as chlorinated polyethylene (CPE) and chlorosulfonated polyethylene (CSPE)

The polymeric materials used most frequently as geomembranes are HDPE, PVC, CSPE, and CPE. The thicknesses of geomembranes range from 20 to 120 mil (58, 59). The recommended minimum thickness for all geomembranes is 30 mil, with the exception of HDPE, which must be at least 60 mil to allow for proper seam welding (58).

Depending on the type of membrane, several bonding systems are available for the construction of both factory and field seams. Bonding methods include solvents, heat seals, heat guns, dielectric seaming, extrusion welding, and hot wedge techniques. To ensure integrity of the seams, a geomembrane should be seamed using the bonding system recommended by the manufacturer (60).

If locally available soils do not possess properties to achieve the specified hydraulic conductivity, soil additives can be used. Soil additives, such as bentonite or other clay materials, can decrease the hydraulic conductivity of the native soil (96).

Bentonite is a clay mineral (sodium-montmorillonite) and may be obtained in a dry, powdered form that is relatively easy to blend with on-site soils. Bentonite expands when mixed with water (hydration). This property allows relatively small amounts of bentonite (5 to 10%) to be added to a noncohesive soil (sand) to make it more cohesive (96).

The most common additive used to amend soils is sodium bentonite. The disadvantage of using sodium bentonite includes its vulnerability to degradation as a result of contact with chemicals and waste leachates. Calcium bentonite and other materials, including lime, cement, and other clay minerals such as atapulgit, may be used as soil additives (60, 97).

The percent of additive that is added to the soil is determined by preparing several different mixtures and running permeability tests on these mixtures.

Like geomembrane liners, soil-additive liners must be placed on a clean, smooth surface. The desired amount of additive is added to the soil surface and thoroughly mixed and compacted. The liner is placed in lifts, with each lift being no thicker than 6 in (15 cm). The total thickness of the soil-additive liner is usually the same as a clay liner. At least 1 ft (30 cm) of clean sand cover should be placed over the liner to protect it during refuse filling.

When determining the liner best for the project, it is important that the following questions be answered:

1. Can the liner be placed on the desirable side slopes?
2. Is liner material delivery schedule reliable enough or must an inventory be kept on site?

3. Can the liner be placed throughout the year or must sufficient liner be placed before the onset of winter or a rainy season?
4. Can the liner be placed with landfill personnel or must private contractors be retained?

All landfill liners must have leachate collection systems included in the design. The collection system must be designed to minimize the leachate hydraulic head on the liner. The liner design must also incorporate provisions to isolate portions of the liner system in the unlikely event of a liner failure. This isolation is accomplished by dividing the total liner system into a series of minisystems. Each minisystem has its own liner and collection system with lined berms or cell walls preventing leachate flow between minisystems.

Leachate Collection and Treatment. Leachate is the liquid that has passed through or emerged from solid waste and contains dissolved, suspended, or immiscible materials removed from the solid waste. The characteristics of leachate vary with the age of the landfill and the material placed in the landfill. Table 8.53 illustrates some typical data on leachate composition. Since all sanitary landfills generate leachate and leachate has very strong wastewater characteristics, collection and treatment of leachate is a necessary part of the overall facility.

Rainwater falling directly on the landfill and off-site stormwater flowing across the site are the major source of water for leachate generation. Good design dictates that as much off-site stormwater as possible be diverted around the landfill. The critical periods for leachate generation occur during operation. The landfill design must incorporate features that will allow all water not in direct contact with uncovered refuse to flow off the site and bypass the leachate collection system. It is important that the refuse mass be kept as dry as possible. The placement of refuse in trapped stormwater must be avoided.

The function of the leachate collection system is to collect and convey leachate out of the landfill and to control the depth of the leachate above the liner. The leachate collection system should be designed to meet the regulatory performance standard of maintaining less than 12 in (30 cm) depth of leachate, or "head,"

TABLE 8.53 Composition of Leachate from Landfills (61)

Constituent*	Range	Typical
BOD ₅ (5-day biochemical oxygen demand)	2000–30,000	10,000
TOC (total organic carbon)	1500–20,000	6,000
COD (chemical oxygen demand)	3000–45,000	18,000
Total suspended solids	200–1000	500
Organic nitrogen	10–600	200
Ammonia nitrogen	10–800	200
Nitrate	5–40	25
Total phosphorus	1–70	30
Ortho phosphorus	1–50	20
Alkalinity as CaCO ₃	1,000–10,000	3,000
pH	5.3–8.5	6
Total hardness as CaCO ₃	300–10,000	3,500
Calcium	200–3,000	1,000
Magnesium	50–1,500	250
Potassium	200–2,000	300
Sodium	200–2,000	500
Chloride	100–3,000	500
Sulfate	100–1,500	300
Total iron	50–600	60

*All units in milligrams per liter except pH.

above the liner. This head allowance is a design standard and may be exceeded for relatively short periods of time during the active life of the landfill.

Leachate is generally collected from the landfill through sand drainage layers, synthetic drainage nets, or granular drainage layers with perforated plastic collection pipes, and is then removed through sumps or gravity drain carrier pipes. The principal components of the leachate collection system are (96):

- *A low-permeability base.* The bottom liner should have a minimum slope of 2% for effective gravity drainage through the entire operating and postclosure period. Settlement estimates of the foundation soils should set this 2% grade as a postsettlement design objective (99).
- *A high-permeability drainage layer.* The drainage layer is constructed of either natural granular materials (sand and gravel) or synthetic drainage material (geonet) placed directly on the bottom liner, or on a protective bedding layer (e.g., geofabric) directly overlying the liner.
- *Perforated leachate collection pipes.* The collection pipes are located within the high-permeability drainage layer to collect leachate and carry it rapidly to a sump or collection header pipe.
- *Protective filter layer.* If necessary, a filter layer is placed over the high-permeability drainage material to prevent physical clogging of the material by fine-grained material.
- *Leachate removal system.* Collection sumps or header pipe system are installed at low points so leachate can be removed for holding/treatment.

The relative performance of design options for the leachate collection system and liner layers may be compared and evaluated by the HELP (Hydrologic Evaluation of Landfill Performance) model (100, 101). The HELP model was developed by the U.S. Army Corps of Engineers for the U.S. Environmental Protection Agency and is widely used for evaluating expected hydraulic performance of landfill cover/liner systems (96).

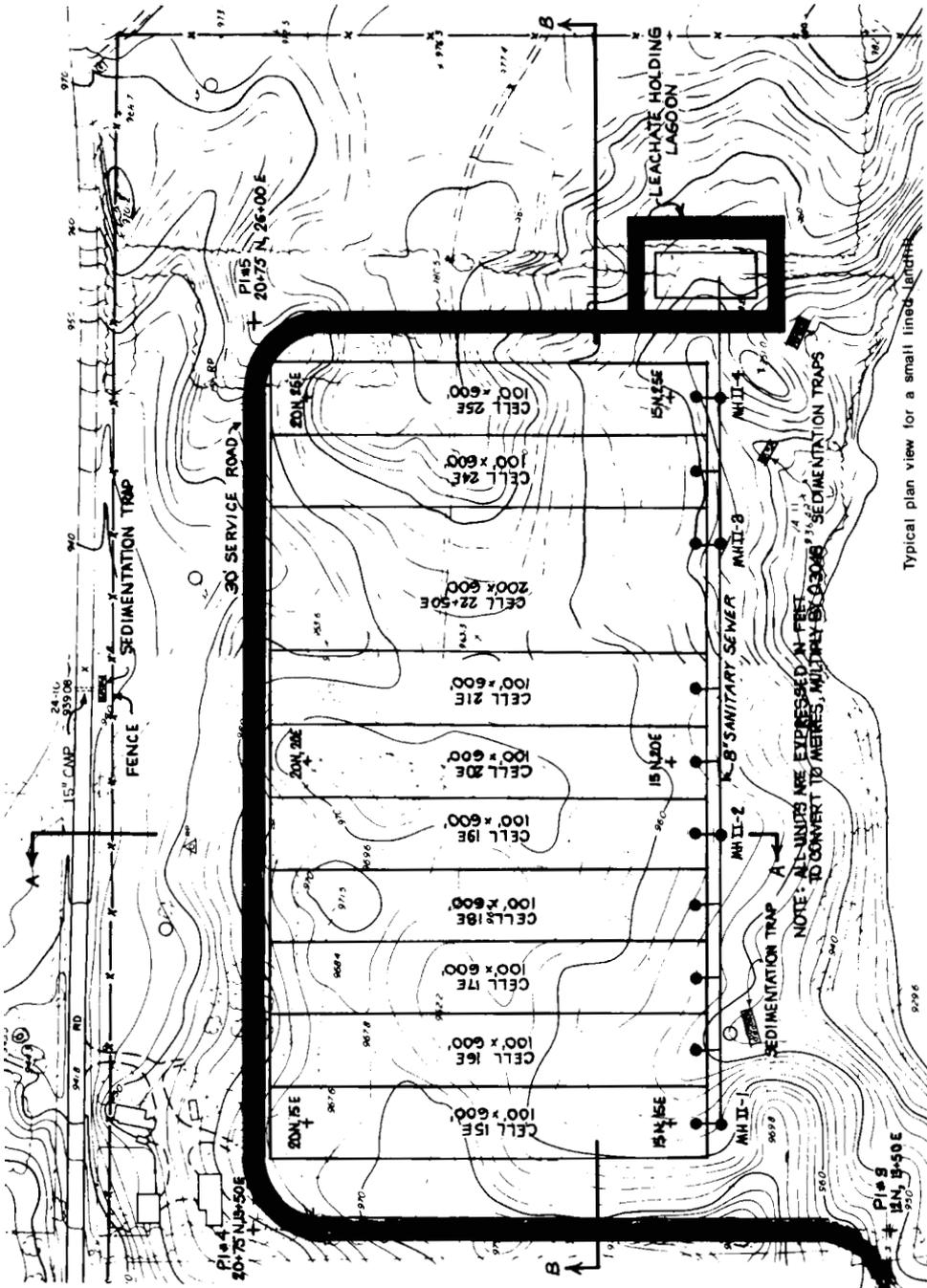
The HELP program calculates daily, average, and peak estimates of water movement across, into, through, and out of landfills. The input parameters include soil properties, precipitation and other climatological data, vegetation type, and landfill design information. Default climatologic and soil data are available but should be verified as reasonable for the site modeled. Outputs from the model include precipitation, run-off percolation through the base of each cover layer subprofile, evapotranspiration, and lateral drainage from each profile. A summary of outputs is produced, including average monthly totals, average annual totals, and peak daily values for several simulation variables (96).

The leachate collection systems must be designed to support heavy equipment loads during operations and the superimposed refuse load when the landfill has been completed. Figure 8.37 shows a typical plan view for a small lined site. Figures 8.38 and 8.39 are sections through the landfill and illustrate the layout for the leachate collection system.

Each lined cell has its own collection system, hydraulically independent from the other cells. In the event of a cell liner failing, the other liners will be unaffected.

Leachate discharging from the collection system should be stored in a lagoon. The lagoon serves as an equalization and pumping basin and sampling and monitoring point. The lagoon must be designed to hold the stormwaters from a 1-in (2.5-cm) 25-year storm plus the amount of leachate that would normally be generated over the anticipated holding period of the lagoon. (In areas where the leachate must be trucked to a disposal point or held for spraying on top of the landfill, the lagoon should have at least a 90-day holding capacity.) Leachate generation will be greatest during the operational period and will decrease once the site is capped with an impermeable final cover. Some jurisdictions do not require the impermeable final cover if a leachate collection system is provided. Because the final amount and strength of the leachate cannot be determined, it is best to install an impermeable final cover.

The high strength of the leachate makes it difficult to treat by itself. Fortunately, if the wastewater and landfill service areas are identical, the quantity of leachate is very small compared to the wastewater flow. [Leachate from a 15-year, 25-ft (7.5 m) deep landfill will generate less than 0.1% of the daily wastewater flow.]



Typical plan view for a small landfill

FIGURE 8.37 Typical plan view for a small landfill.

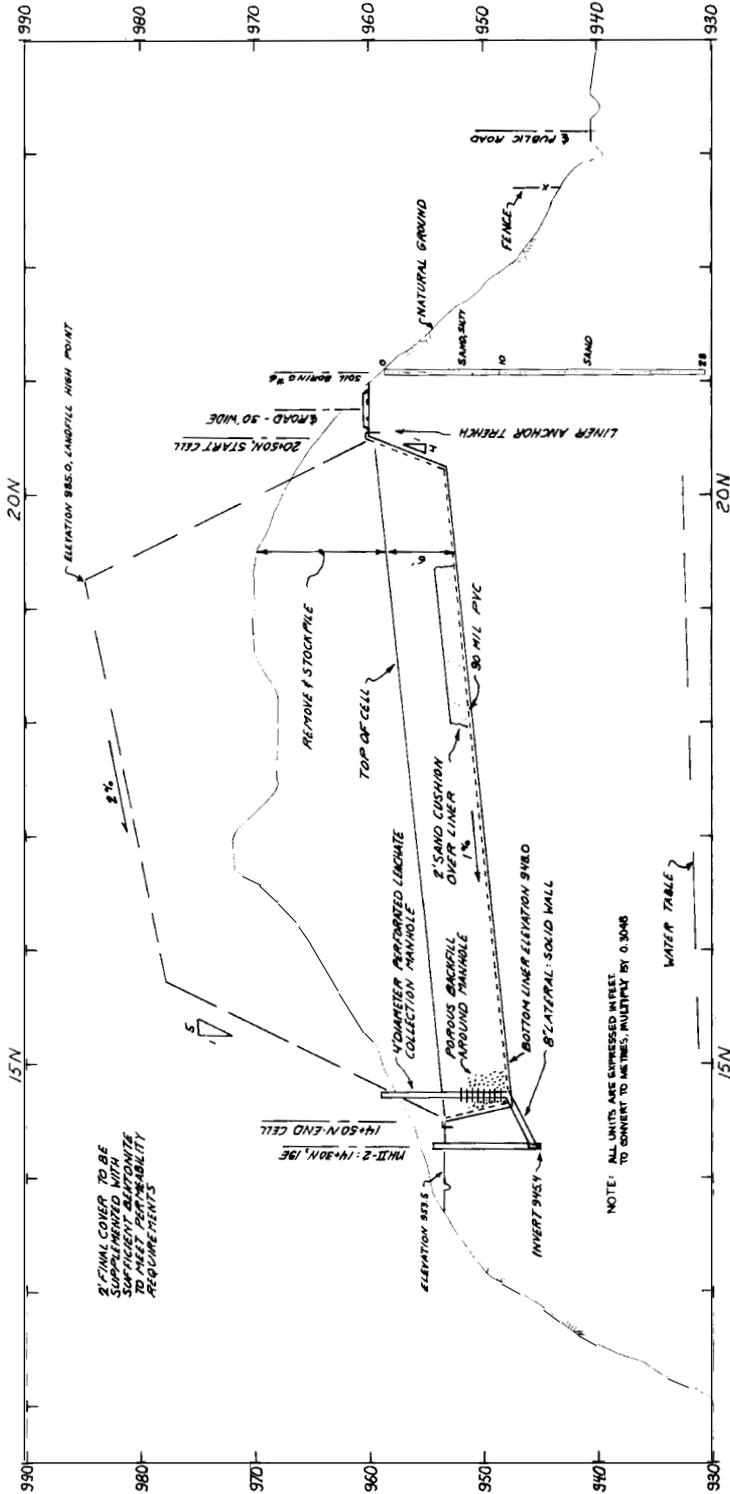


FIGURE 8.38 Typical section view along the width of a small lined landfill.

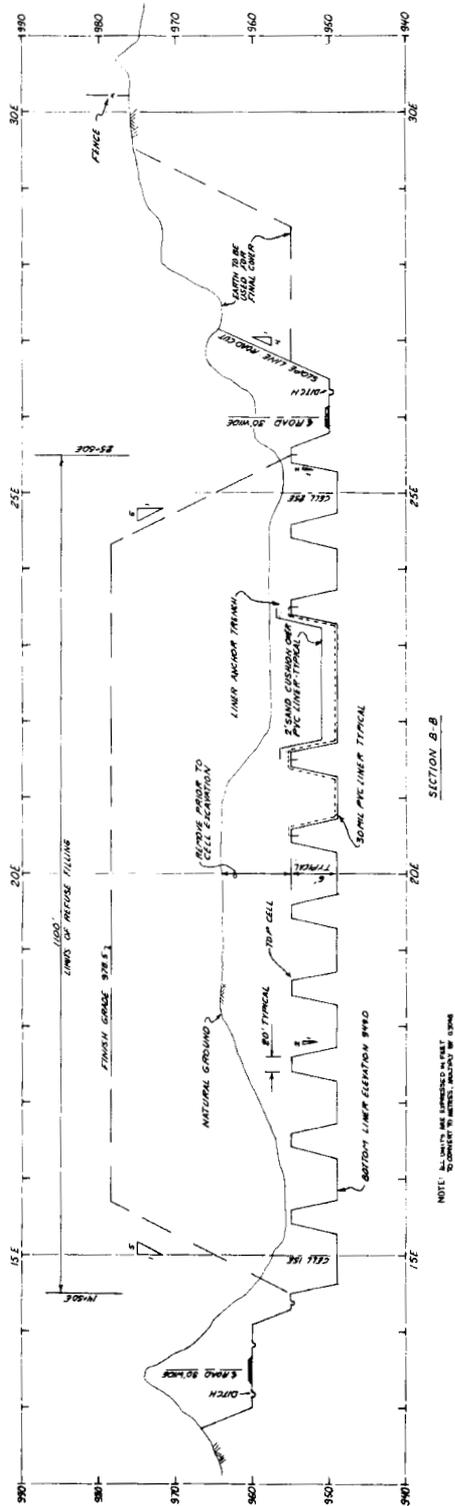


FIGURE 8.39 Typical section view along the length of a small lined landfill.

If possible, it is best to discharge to an existing wastewater plant. However, it is important to understand fully the pretreatment requirements of the receiving plant. Several leachates investigated have exceeded the receiving plants acceptable heavy metal concentrations and required pretreatment.

Recycling leachate through the existing landfill has been practiced. This has usually been accomplished by spraying leachate over the top of the operating landfill. If done in hot weather, some evaporation is also achieved. Tittlebaum (60) has reported that raising the pH of the leachate prior to spraying has proved beneficial. A major problem to spraying is odor control. Recycling might be difficult to accomplish after the site is closed, especially if the site is used for recreation.

Various methods of on-site leachate treatment are available and have been tested in the landfill industry. Among the physical/chemical treatment technologies in use are:

- *Equalization*. Equalization dampens variation in hydraulic and pollutant loadings, thereby reducing shock loads and increasing treatment facility performance.
- *Neutralization*. Neutralization dampens pH variations prior to treatment or discharge.
- *Coagulation/flocculation*. Coagulation/flocculation provides pollutant removal through aggregation of colloidal solids.
- *Gravity separation*. Gravity-assisted separation allows suspended matter to become quiescent and settle and free oils (lighter than water) to become quiescent and float.
- *Emulsion breaking*. The addition of a deemulsifier, such as heat or acid, breaks down emulsions to produce a mixture of water and free oil and/or an oily floc.
- *Chemical precipitation*. The addition of chemicals to wastewater converts soluble metal salts to insoluble metal oxides, which are then removed by filtration.
- *Chemical oxidation/reduction*. By chemical addition, the structure of pollutants are changed so as to disinfect, increase biodegradation and adsorption, or convert pollutants to end products.
- *Air/steam stripping*. Air/steam stripping involves the removal of pollutants from wastewater by the transfer of volatile compounds from the liquid phase to a gas stream.
- *Flotation*. Injection of fine air bubbles causes suspended solids to float to the surface where they are removed by skimming.
- *Sand filtration*. Monomedia or multimedia sand filtration involves a fixed (gravity or pressure) or moving bed of porous media that traps and removes suspended solids from water passing through the media.
- *Ultrafiltration*. Extremely fine grade filters are used to remove organic pollutants from wastewater according to the organic molecule size.
- *Reverse osmosis*. Reverse osmosis relies on differences in dissolved solids concentrations and selective semipermeable membranes to allow for the concentration of dissolved inorganic pollutants.
- *Fabric filters*. Fabric filters screen suspended matter by means of a cloth or paper barrier.
- *Carbon adsorption*. In this process, wastewater is passed over a medium of activated carbon, which adsorbs certain pollutants, primarily organics.
- *Ion exchange*. Selected resins placed in contact with wastewater remove contaminants of similar charge.

Biological wastewater treatment technologies used in the landfill industry for treatment or pretreatment of leachate include:

- *Aerobic systems*. Aerobic systems utilize an acclimated community of aerobic microorganisms to degrade, coagulate, and remove organic and other contaminants.
- *Activated sludge*. Activated sludge is a continuous flow, aerobic biological treatment process that employs suspended-growth aerobic microorganisms to biodegrade organic contaminants.
- *Anaerobic systems*. Anaerobic systems involve the conversion of organic matter in wastewater into methane and carbon dioxide by anaerobic microorganisms.
- *Facultative systems*. Facultative systems stabilize wastes by incorporating a combination of aerobic, anaerobic, and facultative (thriving in either aerobic or anaerobic conditions) microorganisms.

- *Rotating biological contactors.* Rotating biological contactors (RBCs) employ a fixed-film aerobic biological system adhering to a rigid medium mounted on a horizontal, rotating shaft.
- *Trickling filters.* In this process, wastewater passes over a structure packed with an inert medium (e.g., rock, wood, plastic) coated with a biological film of attached microorganisms capable of absorbing and degrading organic pollutants.
- *Sequential batch reactors.* A sequence of batch operations in a single reactor containing acclimated microorganisms is used to degrade organic material. The batch process allows for equalization, aeration, and clarification in a single tank.
- *Powdered activated carbon biological treatment.* The addition of granular activated carbon to biological treatment systems enhances the removal of certain organic pollutants.
- *Nitrification systems.* These systems use nitrifying bacteria to convert ammonia–nitrogen compounds to less toxic nitrate–nitrite compounds.
- *Denitrification systems.* These systems convert nitrate–nitrite to nitrogen gas under anoxic–anaerobic conditions.
- *Land application.* Spray irrigation or other techniques are used to apply the wastewater to the land for treatment by a combination of biological, chemical, and physical processes.
- *Wetlands treatment.* These systems employ natural or man-made wetlands systems that treat wastewater utilizing natural processes of sedimentation, adsorption, and organic degradation.

The treatment sequence employed at any particular facility may vary with the pollutant characteristics of the leachate generated at the landfill. The optimal treatment system at a facility depends upon many factors, including permit requirements, design considerations, landfill acceptance criteria, and management practices. The EPAS new source performance standards (40CFR445) for treatment of leachate and other nonhazardous waste landfill wastewaters before discharge to surface waters are presented in Table 8.54 (102–105)

Various forms of equalization and aerobic biological systems, including aerated lagoons, activated sludge systems, and sequential batch reactors, are the most widely used treatment technology in the landfill industry. These biological systems generally utilize high retention times to enhance performance by reducing variations in raw wastewater flow and pollutant loads.

Vent Landfill Gases. The two principal gases generated in a landfill are carbon dioxide (CO₂) and methane (CH₄). CO₂ is generated during the early (aerobic) stages of landfill life. CH₄ is generated during the later stages of landfill life (anaerobic). Figure 8.40 illustrates gas production and composition from an experimental landfill.

Table 8.54 Nonhazardous Leachate Treatment Limitations for Discharges to Surface Waters

Pollutant or pollutant property	Maximum for any	
	one day	Monthly average
BOD ₅ , mg/L	160	40
TSS, mg/L	89	27
Ammonia, mg/L	5.9	2.5
Zinc, mg/L	0.20	0.11
Alpha Terpineol, mg/L	0.059	0.029
Benzoic Acid, mg/L	0.23	0.13
<i>p</i> -Cresol, mg/L	0.046	0.026
Phenol, mg/L	0.045	0.026
Toluene, mg/L	0.080	0.026
pH, units (range)	6.0–9.0	6.0–9.0

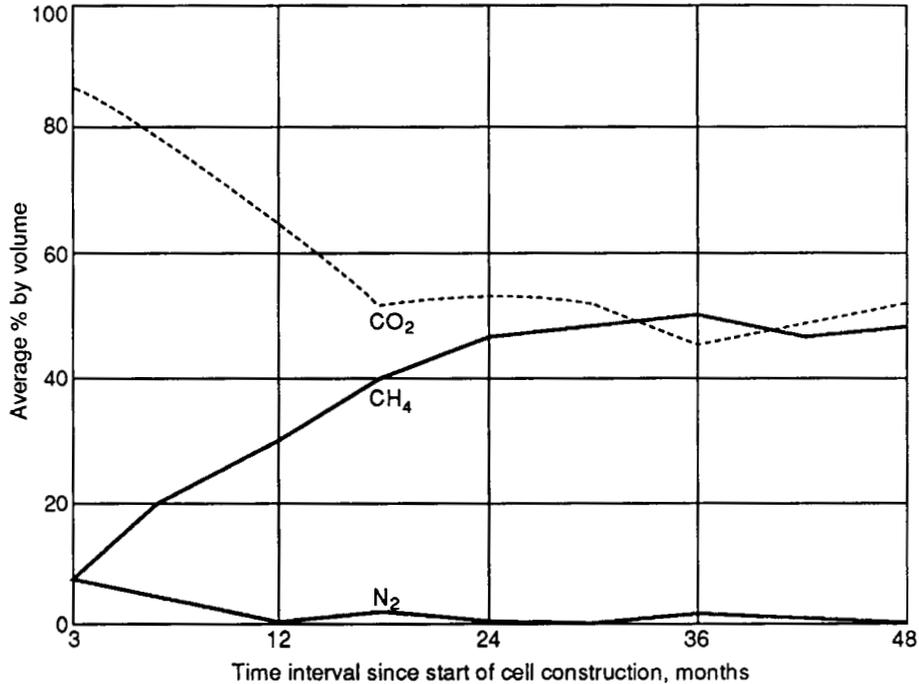


FIGURE 40 Gas production from an experimental landfill (62).

The explosive CH_4 is the most worrisome for landfill designers. The lighter-than-air CH_4 will rise until it meets an impermeable barrier. It is then deflected laterally until an escape is found. There are many ways to vent a new landfill. The best vents are probably the ones that are incorporated with the leachate collection manholes as shown in Figure 8.38. These manholes are on the perimeter of the site and have the best chance of being preserved during operations. Placing vents near the high point of the landfill after completion but prior to final cover placement is another method with a good chance of survival.

Prepare Preliminary Site Layouts. The last steps in the preliminary design stage are the determination of the operating methods, preparation of alternative site layouts, computation of earthwork quantities, and the preparation of the preliminary cost estimates for each alternative. The following is a summary of the required tasks.

Preliminary layouts for the site are most easily studied after plotting on the site topographic map. A map scale of 1:1200 (1 in = 100 ft) and a 2-ft contour interval is standard. The map used in the hydrogeologic study should be satisfactory.

Since earthwork computations will be required, cross sections through the site must be plotted. A grid system should be established and cross sections plotted every 100 ft (30 m) along the principal axis of the site and at 200-ft (60-m) intervals along the minor axis of the site. A good cross section scale is 1:600 (1 in = 50 ft) horizontal and 1:60 (1 in = 5 ft) vertical.

The work area for the site must be defined. The desired setbacks from roads and other physical barriers must be plotted. The designer must also determine the desirable methods of operation. The slope of the existing ground, direction of prevailing wind, amount of topsoil and dirt to be stockpiled and the equipment to be used will all influence operating methods.

Special attention must be given to the movement of all equipment on the site. Landfills are subjected to heavy traffic peaking, i.e., all refuse trucks will arrive at the site at the same time. Paved roads must be laid out so that the destructive tracked landfill equipment does not have to cross them.

The movement of cover material is one of the landfill's principal activities. The site layout must provide for efficient movement. Double handling of excavated material must be minimized. In northern climates, consideration must be given to preventing the cover material from freezing. During freezing weather, the movement of heavy equipment over cover material will drive frost deeper into it. The unfortunate result is the impossible task of excavating the frozen cover material.

Figure 8.39 shows a combination of the so-called trench and area fill methods. The trench fill method consists of excavating a trench (cell) below the ground surface. The trench is filled by depositing refuse at the top of the trench and spreading it down a ramp to the bottom. The trench can be excavated using draglines or scrapers. The excavated earth can be stockpiled on the adjoining trench for later use as daily cover. The trench method leaves a wedge of undisturbed earth between trenches that serves to isolate each trench. The amount of cover material needed for trench fills is less than other operating methods because only the top of the refuse must be covered. A disadvantage of the trench fill is the space taken up by the wedge between trenches.

The top portion of the landfill section illustrated in Figure 8.39 is an area fill operation. Area fill operations take place on large open areas. The refuse is placed on top of the area and spread and compacted. Daily cover is transported to the area, usually by scraper. Landfill operations above ground are normally area fills.

The locations of the active fill areas, roads, fences, buildings, and other physical features must be plotted on the site topographic map. The proposed top and bottom elevations of the fill are plotted on the cross-section sheets. These elevations should be plotted on both the cross sections drawn along the principal axis at 100-ft (30-m) intervals and the minor axis at 200-ft (60-m) intervals. The cross sections are used to compute earthwork quantities and to layout leachate facilities.

The volumes of earth to be excavated can be computed by the average end area method. The formula for volumes by this method is

$$V = L \frac{A_1 + A_2}{2}$$

where V = volume (ft³ or m³) between sections A_1 and A_2

L = length (ft or m) between sections A_1 and A_2

A = area, ft² or m²

The amount of earth excavated must be balanced with the needs of the landfill for daily and final cover. In addition, the dirt excavation operation must be evaluated to determine if it can keep ahead of refuse filling. Typical earth-moving operations for which quantities must be computed are

1. Stripping and stockpiling topsoil for future use
2. Excavating trenches or portions of the area fill to the desired grade
3. The placement and compaction of a clay or soil-additive liner
4. The placement of the earth cushion over any liner
5. The hauling and placement of daily cover
6. The excavation and placement of the final cover
7. The placement of topsoil over the final cover

Estimates of the development cost must be prepared. Typical line items in a cost estimate are

1. Site clearing
2. Strip and stockpile topsoil

3. Site fencing
4. Utilities
5. Gate house
6. Scale
7. Maintenance building
8. Leachate lagoon
9. Leachate collection system
10. Trench excavation (six months' needs)
11. Liner installation (six months' needs)
12. Earth cushion over liner
13. On-site roads, culverts, and berms
14. Off-site road improvements
15. Land purchase
16. Engineering
17. Equipment

An important part of any estimate is the determination of the cash-flow requirements for site development. This is especially true in areas where liner placement might be delayed for four months because of inclement weather. The establishment of a critical-path diagram for site development will be of great assistance in any cash-flow analysis.

Final Design. The final design documents are used to obtain a permit from the regulatory agency and develop the site. These documents must be sufficiently detailed to satisfy both requirements. As-built changes that occur during the operating life of the site are also recorded on these documents.

Prepare Plans for State Regulatory Agency. The plans submitted for a permit or license must show in sufficient detail the design for the landfill. Most states have specific submittal requirements. As a minimum the submittal should include

- The soils and hydrogeologic study.
- The landfill layout, clearly showing the location of fill areas, storm drainage, roads, fences, buildings, leachate facilities and borrow areas. This layout is plotted on the site topographic map.
- Sufficient cross sections to show the bottom and top elevations of the fill, liner systems, the finish side slopes, and the location of any leachate facilities.
- Plan sheets detailing the groundwater monitoring systems, sedimentation control structures, leachate systems, roads, and other facilities to be constructed on the site.
- An operating plan for the site should be prepared. This plan would outline the workers and equipment needed to operate the site. The planned method for collecting and disposing of the leachate must be addressed. The plan must also include the estimated daily tonnage, the estimated life of the site, and a statement on how the site will be monitored after closing.

Preparation of Construction Documents. Once a permit/license is obtained for a site, it is necessary to build it. Since most landfill operators do not have the capabilities to construct all portions of the site, it might be necessary to prepare plans and specifications for the various individual projects. Separate contract documents might be prepared for

- Fences
- Roads
- Buildings
- Liner systems
- Trench excavation
- Equipment

- Periodic testing of groundwater wells
- Scales

Much time and effort will be saved if the designer is able to use sheets of the plans submitted to the regulatory agency for the contract project plans.

Like all civil engineering projects, a permanent system of control points must be established in the field. These control points will be used during the construction and operation of the landfill for horizontal layout. A series of benchmarks must also be established around the site.

Prepare Final Cost Estimate. Table 8.55 is a typical final cost estimate for a sanitary landfill. Item 1 of the final estimate lists the development costs for the landfill. These costs will be incurred prior to the opening of the site. Normally, the money for this work is borrowed at current interest rates. This money will be repaid over the operating life of the site. Included in the estimate is an item for purchase of a truck and tank to haul leachate from the holding lagoon to a nearby treatment plant.

Item II of the final estimate is an estimate of the annual operating costs for the landfill. Included in this estimate is an item for the cost of final cover and perpetual care of the facility. These costs must be collected prior to the closing of the site.

Item III of the final estimate states the unit cost of operating the site.

Landfill Closure. Key objectives for landfill closure are establishment of a low-maintenance cover systems and minimization of the infiltration of precipitation into the waste. Landfill closure technology, design, and maintenance procedures continue to evolve as new geosynthetic materials are developed and as performance history is evaluated for the relatively small number of landfills that have been closed using modern procedures and materials (94).

At a minimum, the closure system should include an erosion (vegetative) layer and an infiltration layer. For specific site conditions, closure system options are a biotic, drainage, and/or gas vent layer. The placement or vertical position of these layers is presented in Table 8-56.

The closure cover system should be designed to minimize infiltration and erosion and be designed and constructed to:

- Minimize infiltration by the use of an infiltration layer that contains a minimum of 18 in (45 cm) of an earthen material
- Minimize erosion of the final cover (top layer) by the use of an erosion layer that contains a minimum 6 in (15 cm) of earthen material that is capable of sustaining native plant growth
- Control permeability to less than or equal to the permeability of any bottom liner system or natural subsoils present, or a permeability no greater than 1×10^{-5} cm/sec, whichever is less.

Other important technical issues in design development include the:

- Degree and rate of postclosure settlement and stresses imposed on soil and/or membrane liner components
- Long-term durability and survivability of cover system
- Long-term waste decomposition and management of landfill leachate and gases
- Environmental performance of the combined final cover system and bottom liner

Erosion Layer: The thickness of the erosion layer is influenced by depth of frost penetration and erosion potential but in no case be less than. This layer also is referred to as the vegetation layer. Vegetative covers are advantageous as they improve the appearance of the site, control erosion of the final cover system, and should require only minimal maintenance. The vegetation component of the erosion layer should have the following specifications and characteristics (106):

- Locally adapted perennial plants that are resistant to drought and temperature extremes
- Roots that will not disrupt the low-permeability layer

SOLID WASTE

TABLE 8.55 Sanitary Landfill Final Cost Estimate

Work item	Estimated quantity		Unit used		Unit price, \$		Amount, \$
	Customary units	SI units	Customary units	SI units	Customary units	SI units	
Item I: Development costs							
Install monitor wells	200	60.96	Foot	Meter	14.50	47.57	2,900.00
Install sediment traps	3	3	Each	Each	300.00	300.00	900.00
Clear site	36.6	14.8	Acre	Hectare	1,450.00	3,585.81	53,070.00
Excavate overburden	120,000	91,752	Cubic yard	Cubic meter	.95	1.24	114,000.00
Muck excavation	5,000	3,823	Cubic yard	Cubic meter	2.50	3.27	12,500.00
Construct earth berms	10,000	7,646	Cubic yard	Cubic meter	0.85	1.11	8,500.00
Install leachate manhole	7	7	Each	Each	400.00	400.00	2,800.00
Install sanitary manhole	5	5	Each	Each	600.00	600.00	3,000.00
Install leachate sewer	1,610	490.728	Foot	Meter	8.75	28.71	14,087.50
Install sanitary sewer	500	152.4	Foot	Meter	20.00	65.62	10,000.00
Excavate leachate lagoon	4,300	3,287.78	Cubic yard	Cubic meter	1.10	1.44	4,730.00
Install lagoon PVC liner	27,216	2,528.3664	Square foot	Square meter	0.41	4.41	11,158.56
Fence site	4,770	1,453.896	Foot	Meter	2.75	9.02	13,117.50
Grade entrance road	700	213.36	Foot	Meter	8.50	27.89	5,950.00
Place asphalt base	1,000	907.18	Ton	Metric ton	19.00	2094	19,000.00
Place asphalt surface	500	453.59	Ton	Metric ton	25.00	27.56	12,500.00
On-site transfer station	110	33.528	Foot	Meter	150.00	492.13	16,500.00
Gate house	I	I	Lump sum	Lump sum	15,000.00	15,000.00	15,000.00
Purchase and install scale	I	I	Lump sum	Lump sum	35,000.00	35,000.00	35,000.00
Maintenance building	I	I	Lump sum	Lump sum	120,000.00	120,000.00	120,000.00
On-site service road	500	152.4	Foot	Meter	6.00	19.69	3,000.00
PVC liner anchor trench	3,200	975.36	Foot	Meter	0.50	1.64	1,600.00
Place PVC liner	478,125	44,417.8125	Square foot	Square meter	0.36	3.88	172,125.00
Place liner sand cushion	478,125	44,417.8125	Square foot	Square meter	0.05	0.54	23,906.25
Place topsoil and seed	2	0.81	Acre	Hectare	780.00	1,925.93	1,560.00
Install waterwell	1	1	Lump sum	Lump sum	3,000.00	3,000.00	3,000.00
Install septic tank	1	1	Lump sum	Lump sum	3,200.00	3,200.00	3,200.00
Purchase 40CY containers	3	3	Each	Each	3,000.00	3,000.00	9,000.00
Purchase leachate truck	1	1	Each	Each	53,000.00	53,000.00	53,000.00
Purchase leachate tank	1	1	Each	Each	4,500.00	4,500.00	4,500.00
Purchase land	82.89	33.55	Acre	Hectare	1,495.96	3,695.98	124,000.00
Pave public approach road	5,820	1,773.936	Foot	Meter	9.50	31.17	55,290.00
Total development cost							<u>928,894.81</u>

Estimated life of site is ten (10) years

Current prevailing interest rate is 9%

Estimated annual development cost is \$144,900

144,900.00

TABLE 8.55 Sanitary Landfill Final Cost Estimate (*continued*)

Work item	Estimated quantity		Unit used		Unit price, \$		Amount, \$
	Customary units	SI units	Customary units	SI units	Customary units	SI units	
Item II: Annual operating costs							
A. Personnel							
Equipment operators (3)							85,000.00
Gatekeeper							20,000.00
Laborer (1)							22,000.00
Engineering time							28,000.00
Total annual personnel cost							155,000.00
B. Equipment							
Landfill compactor							75,000.00
Crawler loader							35,000.00
Earthmover							85,000.00
Leachate truck							15,000.00
Total annual equipment cost							210,000.00
C. Miscellaneous Costs							
Utilities, tools, etc.							22,000.00
Liners: installed							75,000.00
Leachate treatment							3,500.00
Other Contracts							15,000.00
Final cover and perpetual care							52,000.00
Total annual miscellaneous cost							167,500.00
D. Total annual operating cost							532,500.00
Item III: Unit cost							
Annual development cost							144,900.00
Annual operating Cost							532,500.00
Total annual Cost							677,400.00
Estimated tons per operating day	300	272.154					
Operating days per year	306	306					
Annual tons of solid waste	91,800	83,279.124					
Cost per ton (customary)							7.38

- The ability to thrive in low-nutrient soil with minimum nutrient addition
- Sufficient plant density to minimize cover soil erosion
- The ability to survive and function with little or no maintenance
- A variety of plant species sufficient to continue to achieve these characteristics and specifications over time

Deep-rooted shrubs and trees are generally inappropriate because the root systems may penetrate the infiltration layer, creating preferential pathways of percolation. Plant species with fibrous or branching root

TABLE 8.56 Solid Waste Landfill Closure System Layers

Minimum Requirements	Closure Cover	Site Specific Options
Atmosphere		
Erosion - Vegetative Layer		
		Biotic Layer
		Drainage Layer
Infiltration Layer		
		Gas Vent Layer
Solid Waste		

systems are best suited for this application and may include a large variety of grasses and shallow-rooted plants. The timing of seeding (spring or fall in most climates) is critical to successful germination and establishment of the vegetative cover (106). Temporary vegetative covers may be grown from fast-growing seed stock, such as ryegrass.

Selection of the soil for the vegetative cover (erosion layer) should include consideration of soil type, nutrient and pH levels, climate, species of the vegetation selected, mulching, and seeding time. Loamy soils with a sufficient organic content generally are preferred. The balance of clay, silt, and sand in loamy soils provides an environment conducive to seed germination and root growth (107).

In some cases, the erosion layer may be asphalt or concrete. These materials promote run-off with negligible erosion; however, they will deteriorate due to thermal expansion and deformation caused by subsidence. In other cases, crushed rock may be spread over the landfill cover in areas where weather conditions, such as wind, heavy rain, or temperature extremes, would be expected to cause deterioration of vegetative covers (106).

Biotic Layer: Deep plant roots or burrowing animals (collectively called biointruders) may disrupt the

drainage and the low hydraulic conductivity layers, thereby interfering with the drainage capability of the layers. A 30 cm (12 in) biotic barrier of cobbles directly beneath the erosion layer may stop the penetration of some deep-rooted plants and the invasion of burrowing animals. Geosynthetic products that incorporate a time-released herbicide into the matrix or on the surface of the polymer also may be used to retard plant roots. The longevity of these products requires evaluation if the cover system is to serve for longer than 30 to 50 years (108).

Drainage Layer. The drainage layer in a final cover system redirects percolating water that has infiltrated through the erosion layer after surface runoff and evapotranspiration losses. By removing water in contact with the low-permeability layer, the potential for leachate generation is diminished. Caution should be taken when using a drainage layer because this layer may prematurely draw moisture from the erosion layer that is needed to sustain vegetation.

A permeable drainage layer, constructed of soil or geosynthetic drainage material, may be constructed between the erosion layer and the underlying infiltration layer. If a drainage layer is used, a filter layer, composed of either a low-nutrient soil or geosynthetic material, may be placed between the drainage layer and the cover soil to help minimize clogging of the drainage layer by root systems or soil particles. With soil or geosynthetic material, the filter layer should be at least 12 in (30 cm) thick with a hydraulic conductivity in the range of 1×10^{-2} cm/sec to 1×10^{-3} cm/sec.

When granular drainage layer material is used, the filter layer should be sloped at least 3% at the bottom of the layer. Greater thickness and/or slope may be necessary to provide sufficient drainage flow as determined by site-specific modeling (106). Granular drainage material will vary from site to site depending on the type of material that is locally available and economical to use. Typically, the material should be no coarser than 0.375 inch (0.95 cm), classified according to the Universal Soil Classification System (USCS) as type SP, smooth and rounded, and free of debris that could damage an underlying membrane (106).

When geosynthetic materials are used, the filter layer (preferably a nonwoven needle-punch fabric) should be placed above the geosynthetic material to minimize intrusion and clogging by roots or by soil material from the top layer.

Gas Vent Layer. Landfill gas collection systems serve to inhibit gas migration and typically are installed directly beneath the infiltration layer to collect combustible gases (methane) and other potentially harmful gases (hydrogen sulfide) generated by microorganisms during biological decay of organic wastes. The collection system diverts these gases via a pipe system through the infiltration layer to the gas vent layer.

The gas vent layer is usually 12 in (30 cm) thick and should be located between the infiltration layer and the waste layer. Materials used in construction of the gas vent layer should be medium- to coarse-grained porous materials or geosynthetic materials with demonstrated equivalent performance. Venting to an exterior collection point may be provided by pipes configured laterally throughout the gas vent layer to channel the gases to vertical risers or lateral headers. If vertical risers are used, their number should be minimized, as they are frequently vandalized, and located at high points in the cross section (106). Since condensates will form within the gas collection pipes, the design should address drainage condensate drainage from low points.

Infiltration Layer. The infiltration layer must be at least 18 in (45 cm) thick and consist of earthen material that has a hydraulic conductivity (coefficient of permeability) less than or equal to the hydraulic conductivity of any bottom liner system or natural subsoils. If a membrane is in the bottom liner, there must be a membrane liner in the final cover to achieve a permeability that is less than or equal to the permeability of the bottom liner. For units that have a composite liner with a membrane liner, or naturally occurring soils with very low permeability, such as 1×10^{-8} cm/sec, the infiltration layer also should include a synthetic membrane as part of the final cover. Landfill units with poor or nonexistent bottom liners possessing hydraulic conductivities greater than 1×10^{-5} cm/sec must have an infiltration layer that meets the 1×10^{-5} cm/sec minimum requirement.

The infiltration layer is designed and constructed in a manner similar to that used for soil liners (107), with the following differences:

- Because the cover is generally not subject to large overburden loads, the issue of compressive stresses is less critical unless postclosure land use will entail construction of objects that exert large amounts of stress.
- The soil cover is subject to loadings from settlement of underlying materials. The extent of settlement anticipated should be evaluated and a closure and postclosure maintenance plan should be designed to compensate for the effects of settlement.
- Direct shear tests performed on construction materials should be conducted at lower shear stresses than those used for liner system designs.

Earthen material used in the construction of the infiltration layer should be free of rocks, debris, and other such material that may increase the hydraulic conductivity by promoting preferential flow paths. To facilitate run-off while minimizing erosion, the surface of the compacted soil should have a minimum slope of 3% and a maximum slope of 5% after allowance for settlement. Final side slopes, which often are greater than 5%, should be evaluated for erosion potential.

Membrane and clay layers should be placed below the maximum depth of frost penetration to avoid freeze-thaw effects (106). Infiltration layers may be subject to desiccation, depending on climate and soil water retention in the erosion layer. Fracturing and volumetric shrinking of the clay due to water loss may increase the hydraulic conductivity of the infiltration layer.

When a membrane is used as an infiltration layer, the membrane should be at least 20 mils in thickness. Some membrane materials may need to be a greater thickness; for example, a minimum thickness of 60 mils is recommended for HDPE because of the difficulties in making consistent field seams in thinner material. Increased thickness and tensile strengths may be necessary to prevent failure under stresses caused by construction and by waste settlement during the postclosure care period. The hydraulic performance, strength, resistance to sliding, and actual thickness of membranes should be considered.

Hydraulic Performance. The design of a final cover is site-specific and the relative performance of cover design options may be compared and evaluated by the HELP (Hydrologic Evaluation of Landfill Performance) model (100, 101). The HELP model may be used to estimate the hydraulic performance of the final cover system design. Information provided by the HELP model includes surface run-off, duration and quantity of water storage within the erosion layer, and net infiltration through the cover system to evaluate whether leachate will accumulate within the landfill (107).

Settlement and Subsidence. Waste decomposition and consolidation can cause excessive settlement and subsidence of the final cover system. This can impair the integrity of the system and can result in:

- Ponding of surface water on the erosion or infiltration layer
- Interference with operation of the gas collection pipe system
- Fracturing of low-permeability infiltration layers
- Structural failure of membrane liners

The extent and rate of waste settlement is an estimate at best. Records of the type, quantity, and location of waste materials disposed may be useful. Compacting the waste daily or landfilling baled waste will reduce the settlement from consolidation.

Sliding Instability. The slope angle, slope length, and overlying soil load limit the stability of component interfaces (membrane with soil, geotextile, and geotextile/soil). Soil water pore pressures developed along interfaces also can dramatically reduce stability. Unstable slopes may require remedial measures to improve stability as a means of offsetting potential long-term maintenance costs.

LAND RECLAMATION

Using the principles and practices of sanitary landfill design, otherwise useless land may be reclaimed with solid wastes without concern for development of health problems. In practice, land reclamation is most

commonly associated with selection of a landfill site that will provide for treatment and disposal of solid wastes.

Land reclamation may also be achieved when the end products of solid waste treatment are applied to poor-quality or disturbed lands. Compost and the residue from incineration and other thermal processes may be used. Soil conditioning is best achieved with the application of compost or codisposal with stabilized wastewater sludges.

Composting

Composting is the biodegradation of the organic constituents in wastes (solid wastes and wastewater sludges). Through the microbial activity taking place during composting, organic matter is decomposed into a stable, humus-like substance. At the same time the heat produced can result in pathogen destruction. Composting is an ancient practice whereby farmers have converted organic wastes into soil amendments. These amendments were used to stabilize soils from erosion, provide nutrients, and replenish depleted organic matter that was lost through intensive farming.

Composting of organic wastes and residues remained more of an art than a science until about 50 years ago. Until this period, there were several developments of mechanical or intensive systems in Europe, such as the Itano process in 1928, Beccari in 1931, and VAM in 1932. The Europeans continued to develop and install composting systems in Europe, South America, and Asia.

In 1974, the U.S. Department of Agriculture at Beltsville, Maryland, developed the “static pile” method. This method is currently being used by over 200 municipalities throughout the United States. Principal examples of municipalities using this process are Durham, N.H., Bangor, Me., Portland, Me., Greenwich, Ct., Camden, N.J., Philadelphia, Pa., Washington, D.C., Columbus, Ohio, and Windsor, Ontario, Canada.

Public Health Issues. The major public health issues associated with composting using solid wastes mixed with sewage sludge are pathogens, heavy metals, and odors.

Pathogens. Sewage sludge contains numerous pathogenic organisms. The four principal groups are bacteria, viruses, protozoa and helminths. Many of these pathogens survive the wastewater treatment and during the process are deposited in the sludge.

The heat generated during composting, as a result of the activities of thermophilic organisms, is capable of killing all four groups of pathogens present in sewage sludge. The efficiency of pathogen destruction depends on the ability of the process to subject the sludge to uniformly high temperatures. Figure 8.41 shows typical temperatures achieved by the static aerated pile method. Similar temperatures can be achieved using the “within-vessel” method. Windrow composting generally results in lower temperatures.

In September 1979, the U.S. Environmental Protection Agency (EPA) issued regulations title “Criteria for Classification of Solid Waste Disposal Facilities and Practices” (40 CFR Part 257). Two major process categories were described: (1) processes to significantly reduce pathogens (PSRP), and (2) processes to further reduce pathogens (PFRP). PSRP were designed to result in 90% reduction of bacteria and viruses, and PFRP were aimed at reducing pathogens to negligible numbers.

Heavy Metals. Heavy metals are present in sewage sludge as a result of domestic and industrial discharges. Normally, domestic sewage contains very low levels of heavy metals. However, industrial discharge can result in large quantities of various elements. The EPA is primarily concerned with those elements that can accumulate in food crops and are toxic to humans. The elements in sludge of greatest concern to human health are cadmium, lead, arsenic, selenium, and mercury. Only cadmium is normally found in sewage sludge at levels that, when applied to soils, can be absorbed by plants and accumulate in edible parts, thereby entering the food chain.

Odors and Vectors. Sewage sludge contains volatile malodorous compounds, such as mercaptans, skatoles, phenols, and hydrogen sulfide. During composting these compounds can produce unpleasant odors.

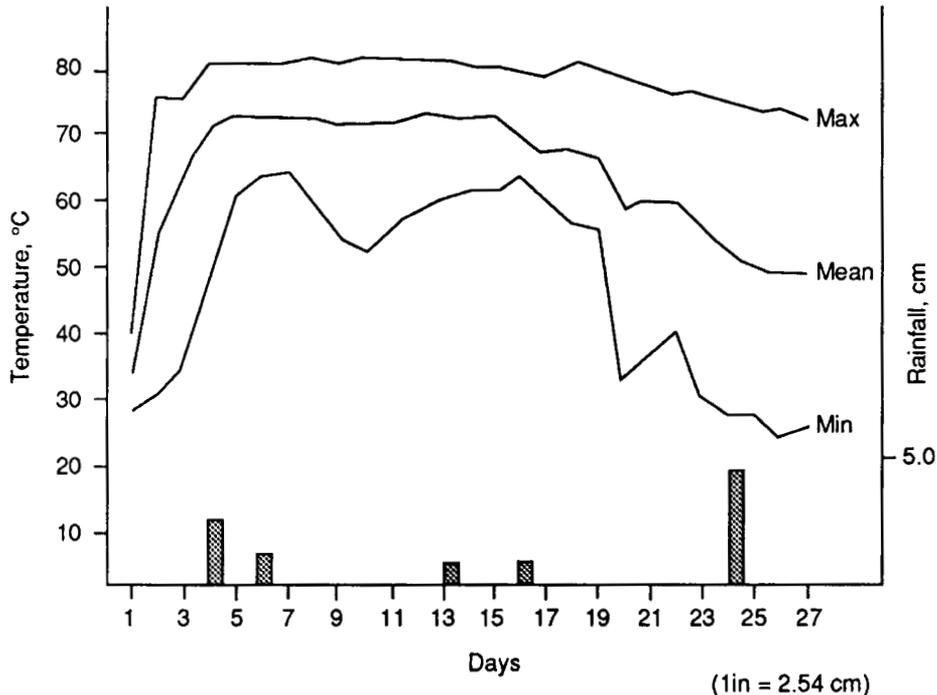


FIGURE 8.41 Maximum, minimum, and mean temperatures recorded during the composting of raw sludge by the Beltsville aerated pile method.

Selection of the proper process, designing the facility, and managing it properly will result in an operation free of malodors.

Vectors, such as rats and flies, are not found in sewage sludge composting operations. However, they may be present in solid waste or garbage composting operations if the facility is not kept clean or managed properly.

Biological Processes. Rapid biodegradation of the sludge and stabilization of the organic fraction as compost depends mainly on the optimal interaction of temperature, oxygen, moisture, and the carbon/nitrogen ratio.

Temperature. The microbial population changes continually during composting. As the temperature changes from ambient to mesophilic [104 to 113°F (40 to 45°C)] bacteria multiply and will reach levels of several million per gram. If the heat produced during the biological activity is contained within the system, the temperature will progress from the mesophilic stage to the thermophilic stage. Mesophilic bacteria are then replaced by thermophilic bacteria and other organisms, such as fungi and actinomycetes. Spore-forming bacteria as well as thermophilic actinomycetes are found at temperatures exceeding 158°F (70°C). Most of the decomposition occurs in the thermophilic stage. As indicated earlier, the high temperatures above 131°F (55°C) will effectively destroy most pathogens. After several weeks of composting, temperatures begin to decrease and eventually the thermophilic organisms give way to mesophilic ones.

Oxygen. Composting can occur under anaerobic (lack of oxygen) or aerobic conditions. Aerobic composting is not only faster but also does not produce malodors. Consequently, sewage sludge of solid waste

composting is generally conducted under aerobic conditions. Aerobic conditions can be obtained by periodically turning the mass of organic materials or by forcing air through the mass.

Oxygen levels between 5 and 15% are optimum. Oxygen levels below 5% may result in anaerobic conditions, which may slow the decomposition process and produce undesirable odors. Oxygen levels above 15% are indicative of excessive aeration and may result in loss of heat with slower decomposition and poor pathogen destruction.

Moisture. The optimum moisture levels for composting are between 50 and 60% by weight. Below 40%, decomposition is reduced, and above 60% the pore space necessary for aerobic composting is blocked by water and anaerobic conditions can occur. Moisture also affects the processing and materials handling in composting operations.

Carbon/Nitrogen Ratio. The carbon/nitrogen (C/N) ratio is one of the most important parameters affecting the rate of decomposition of organic materials. Carbon is an energy source for the microorganisms, whereas nitrogen is necessary for protein synthesis. The ideal C/N ratio is between 25 and 30. Sewage sludge has a low C/N ratio (10 to 15), whereas solid waste has a high ratio (35 to 80). The use of bulking materials with sewage sludge enhances the C/N ratio and the use of sewage sludge in combination with solid waste improves the latter's ratio.

Process Flow. There are two principal methods for aerobic composting. One method utilizes agitation or turning to induce aerobic conditions, whereas the other method employs mechanical means of forcing air into the system. Figure 8.42 illustrates the process flow for the forced-air system. The agitated system, often characterized by the windrow method, may not require a bulking material, or the bulking material may be the dried compost.

In most cases sewage sludge is mixed with a bulking material, solid waste, or dried compost. Good, thorough mixing is essential. In the case of solid waste, grinding is essential in order to increase the surface area and accelerate composting. The mixture is then placed in windrows, aerated piles, tanks, drums, or silos. The mixture is composted for 14 to 21 days. Depending on the facilities available, climate, and materials handling, drying may or may not be needed. Drying facilitates screening, bulking material recovery, and solid waste separation. Screening also produces a uniform product for distribution and marketing.

Design Considerations. The selection of the composting system and the design of facilities depend on such aspects as the site of operations, climate, sludge and solid waste characteristics, and types of bulking material available.

Site. The two most important site considerations are location and land availability. Ideally, the composting site should be located near a landfill or, in the case of sludge, adjacent to a wastewater treatment plant. This will reduce materials handling and facilitate operations. Proximity to residences or industry impact the

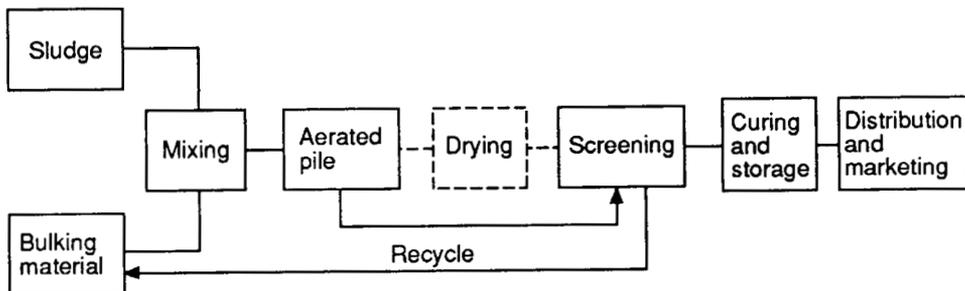


FIGURE 8.42 Typical flow scheme of the composting operation.

facilities required and costs. Nonintensive composting operations will generally require 1 acre (0.4 ha) of land for 6 dry tons (5.4 metric tons) of sludge solids produced per day.

Climate. Cold ambient temperatures rarely affect the composting process unless the solid waste or sludge is frozen. Precipitation primarily affects the mixing and screening phases of the process. In areas of high precipitation or when it occurs in periods of high intensity, certain phases of the process should be under cover. Cover may be necessary to facilitate materials handling and ease of operation.

Sludge Characteristics. The percentage of solids impacts materials handling, bulking material selection, and the quantities needed. A sludge with 17% solids could require 3 parts of bulking agent to 1 part sludge, whereas at 25% solids, between 1.5 and 2 volumes of bulking material are needed.

The chemical characteristics of the sludge may also impact the bulking material selection and quantity used. Since sewage sludge often contains undesirable heavy metals, the bulking material dilutes the heavy metal content and reduces their availability to plants. Furthermore, the bulking material enhances the C/N ratio and often improves the product.

Solid Waste Characteristics. Solid waste consisting of paper, metals, garbage, glass, and plastics needs to be ground. The finer the particle size, the faster and more efficient is the composting process. Separation of materials reduces materials handling and results in a better product. Some processes first separate while others compost and then screen. The former is preferable.

Bulking Materials. Bulking materials serve three functions. They adjust the moisture content of the mass, adjust the C/N ratio, and provide structure or porosity to the mass. The most common bulking materials are woodchips, sawdust, solid waste, shredded rubber tires, straw, leaves, brush chips, bark, and compost. Important characteristics are particle size, moisture content, and adsorbancy. Bulking materials also affect the processing time, materials handling facilities, and product characteristics. High carbonaceous or cellulosic materials generally require long curing periods and large particles need to be ground. In the static pile system, the preferred particle size is 1.5 to 3.0 in (3.8 to 7.6 cm).

Materials Handling. Municipal or industrial waste composting is essentially materials handling that must be cognizant of the biological requirements of the system. Mixing is best done by auger feed mixers or pug-mills. Conveyance to the specific composting site is accomplished with front-end loaders or conveyers. Removal of the material after composting depends on the method used and is done by front-end loaders, augers, and conveyers or digging equipment with elevators and conveyers. Numerous screens are available, from trommel to circular and horizontal shakers. Efficiency of the screen is primarily dependent on moisture content. Materials should not be screened if the moisture content exceeds 50%. Below 35% dust could be a major problem. Screen size depends on the use of the product and the desirability of recovering the bulking material. Most composting operations use screens with 0.25 to 0.50 in (0.6 to 1.2 cm) mesh size. The product can be marketed in bulk or bagged.

Methods of Composting. There are three principal methods for composting solid waste and sewage sludge. These are windrow, static pile, and in-vessel.

Windrow Methods. The windrow system consists of mixing the sludge with a bulking material or previously dried sludge and periodically turning the mass. Open windrow systems are often adequate for digested sludge but are not suitable for undigested (raw) sludge. Odor problems can be very severe when raw sludge is composted in the windrow.

Static Pile. This method was developed at the Department of Agriculture research station at Beltsville, Maryland, in 1975. Currently, this is the most widely used system in the United States. It consists of mixing the sludge with a bulking material and placing the mixture over perforated pipe, i.e., an aerating system (Figure 8.43). The mixture is then insulated with screened or unscreened compost. Air is introduced into the mixture through a blower system. Negative (suction) or positive (blowing) pressure is used and the rate of airflow is controlled to maintain proper oxygen and temperature. Air is also used to increase moisture removal. After composting for 14 to 28 days, the material is usually screened to recover the bulking material for reuse. If drying is necessary, the material can be moved for a short period into a drying shed or left out-

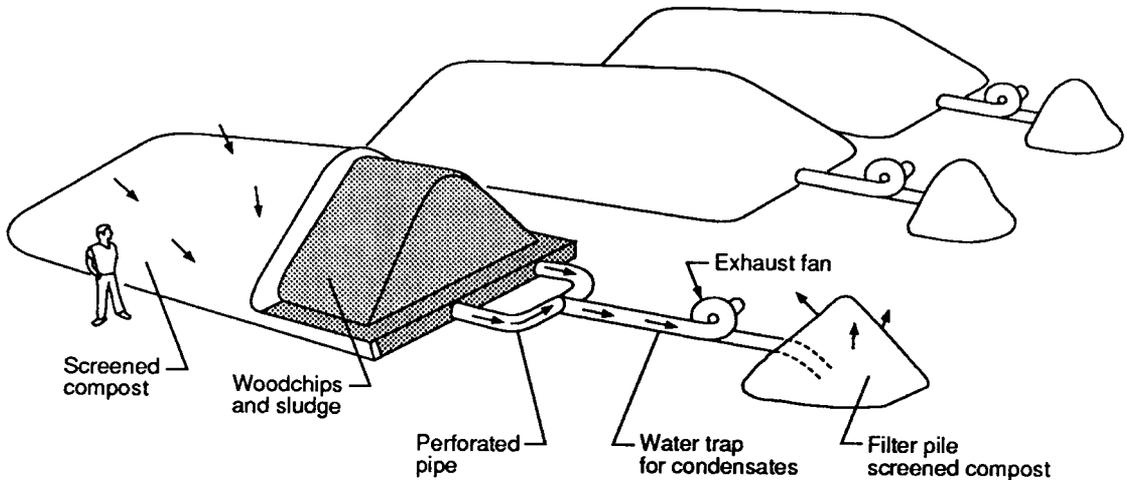


FIGURE 8.43 Composting with forced aeration.

doors to cure and dry. It is preferable to dry and screen before curing because it reduces the material to be handled and provides for more efficient use of the bulking material.

Vertical Systems. Vertical enclosed systems are usually free flowing and consist of either round or square bins, silos, or towers. Materials enter the top of the unit and are extracted at the bottom after composting for a given period of time. Hauling or carbonaceous material is added prior to entering the units. Aerobic conditions are maintained by forcing air through the sludge-bulking material mix from the bottom.

Off gases and odors can be controlled through removal and scrubbing. Temperature or other parameters are monitored through ports along the vertical wall. Moisture control is minimal because the air warms up as it progresses through the composting mixture. The heated air condenses at the top when it reaches the cool mass of new material.

Material is usually removed from the bins or silos after 14 days. At this time it undergoes curing. Curing can take place in a second unit essentially designed in the same manner, or the compost can be cured in a shed or outdoors. The length of the curing will depend on the facilities, i.e., covered or not, and whether air is induced into the compost. Curing usually takes place for several weeks.

Agitated Bed. This enclosed horizontal system consists of an aerated bed contained in a horizontal bin. A premixed bulking agent and sludge is introduced into the bin. The material in the bin can be periodically turned by mechanical means. Material is removed from the bin mechanically. Composting in bins occurs over a period of 14 to 21 days. Curing takes place outside the bins either in an open or covered area and can be accelerated by induced air.

Pugflow System. This horizontal type of system consists of a totally enclosed bin with a hydraulic ram that moves the materials through the unit. Only pilot models are available at present. One unit in Europe is being used in an industrial application. There is no history of operations or data on process efficiency.

Rotating Drum. The system consists of a large-diameter rotating drum whereby sludge and bulking materials are introduced and retained for short time periods (24 to 48 h). These systems have been primarily used for cocomposting of solid waste and sludge. Material ejected from the drum must be further composted and then cured. These additional steps can take place in the windrow or static pile. A cocomposting system is currently being operated in the United States.

Economics. Composting cost at various communities indicates a wide range from \$42 to \$144 per dry ton (0.91 t) of sludge. The primary reason for this wide discrepancy is the result of improper site and facilities

design, which result in excessive operating costs and rising costs of bulking material, The major factors affecting operating costs are bulking material cost, recovery product value and remuneration, and labor. Proper materials, handling, and equipment selection can result in efficient bulking material recovery and savings of 15 to 25% of the other costs.

Economy of Scale. The effect of facility's size on economics of composting is shown in Table 8.55. The hypothetical numbers were developed for facilities from 1 to 10 dry tons/day (0.91 to 9.1 t/day) and at two different sludge solids contents. Relatively little additional economy of scale will result above 10 dry tons/day because equipment and labor are already fully utilized. Solids content affects sludge volume, bulking agent mix ratio, land area, and materials handling costs. Equipment and labor are the two major items affected by the capacity of the facility.

The amount of land required for composting depends on the volume of sludge to be processed, solids content, and the system used. In its simplest form the static pile method requires approximately 1 acre for 5 to 6 dry tons (0.4 ha for 4.5 to 5.5 t) of sludge produced. The windrow system requires considerably more land since current windrow machines produce low [4-ft (1.2-m) high] windrows. The in-vessel system requires less space than the static pile method. Land costs do not greatly affect the overall cost. Composting operations should preferably be sited at the treatment plant.

Table 8.57 shows the sensitivity of capital and O&M costs for the simplified static pile system. This system does not include any buildings and assumes that the facility is part of the wastewater treatment plant operation. In cold, humid areas a drying building may be desirable.

The bulking agent and labor have the greatest impact on total costs for 5- and 10-dry-tons/day (4.5- and 9.1-t/day) facilities. Equipment cost is much more sensitive to facility size. Sludge solids content affects larger facilities to a much greater extent. The basic equipment requirements for composting facilities are mixing, materials handling, and screening equipment.

Product Market Value. Composting of sewage sludge results in a marketable end product. Depending on the quality and uniformity of the material, the product has a value as an organic soil conditioner.

Table 8.58 shows the effect of price change or remuneration for the sale of compost on the cost of composting. In many cases the value of the product is equivalent to potting media or topsoil. The value of \$115 represents a hypothetical figure for a facility and its O&M cost assuming the compost is distributed at no value. At \$5 per cubic yard (\$6.60/m³) the total costs (i.e., capital and O&M) would be reduced to \$60 per dry ton (\$66/t) of sludge. Thus, market development can result in substantial savings to a compost facility.

TABLE 8.57 Baseline Composting Costs, \$/Dry Ton

Item	Solids content					
	15%			25%		
	Solids content capacity, dry tons/day					
	1	5	10	1	5	10
Site	10	10	10	5	5	5
Equipment	46	15	12	46	15	8
O&M labor	55	33	22	55	23	16
Bulking agent	40	40	40	18	18	18
Other	17	17	17	17	17	17
Total costs	168	116	101	141	78	64
Sale of compost	38	38	38	22	22	22
Net costs	130	78	63	119	56	42

Note: 1 dry ton (2000 lb) = 0.9 metric tons.

TABLE 8.58 Comparison of Compost Market Value and Cost of Composting

Market value of compost, \$/yd ³	Estimated cost of composting, \$/dry ton sludge
Free	115
1	104
3	82
5	60
7	38
10	5

Note: 1 yd³ = 0.76 m³ 1 dry ton = 0.9 t.

Utilization and Marketing. Sludge compost is an excellent organic soil conditioner and low-analysis fertilizer that has been stabilized. It decomposes slowly and thus remains effective over a longer period of time. The addition of compost to soils improves their physical properties, as evidenced by increased water content, increased water retention, enhanced aggregation, increased soil aeration, increased permeability, increased water infiltration, and decreased surface crusting.

Table 8.59 lists the various potential users. However, the main users are the private nonfood plant growers, public agencies, and those involved in land reclamation. Use in agriculture or for food-chain crops can be limited as a result of industrial contamination of the sludge. The private nonfood users are primarily in

TABLE 8.59 Major Compost Uses by User Type

1. Private residential
 - a. Garden application for food
 - b. Nonfood applications
2. Private food
 - a. Field crops for food and feed
 - b. Garden crops for food and feed
 - c. Fruit trees
3. Private nonfood
 - a. Greenhouses
 - b. Nurseries
 - c. Golf courses
 - d. Landscape contractors
 - e. Turfgrass farmers
 - f. Industrial park grounds
 - g. Cemeteries
4. Public agencies
 - a. Public parks
 - b. Playgrounds
 - c. Roadsides and median strips
 - d. Military installations
 - e. Public grounds
5. Land reclamation
 - a. Landfill cover
 - b. Strip-mined lands
 - c. Sand and gravel pits

the horticultural sector. Compost is excellent for revegetation of disturbed lands. Table 8.60 shows data from the analysis of some composts from sewage sludge.

Distribution of the compost is an essential phase of the marketing program. Identification of distribution alternatives can assure the municipality that the compost will be removed from the site in a timely fashion without interfering with treatment plant or composting operations. The distribution system also helps to assure users of obtaining material when needed.

Incineration

Disposal of solid waste is one of the most troublesome problems facing urbanized areas at this time. Changes in packaging practices and improvements in the general standard of living have resulted in significant increases in volumes of solid waste generated over the past 40 years. Additionally, disposal of hazardous chemicals from residential areas and commercial establishments has created concerns regarding disposal of the as-collected waste in landfills.

One method of alleviating these problems is to process the solid waste by incineration prior to land disposal of the residuals. In the incineration process, when properly designed and operated, the combustible portion of the solid waste is burned, producing a residue essentially free of putrescible organic material. Benefits from this process include reduction of the volume of the solid waste and reduction in the potential for groundwater pollution from organic and hazardous constituents. Further, the potential exists for extraction and reuse of mineral constituents in the residue, use of remaining material from the residue as a fill or

TABLE 8.60 Elemental Composition of Composted Sludge from Different States

	Concentration*				
	Massachusetts	Maryland	New Hampshire	Utah	Connecticut
	Percent				
Nitrogen, total (N)	0.8	1.6	0.6	1.7	1.2
Phosphorus (P)	0.04	1.0	0.6	0.00	—
Potassium (K)	0.5	0.2	0.1	0.00	—
Calcium (Ca)	1.4	1.4	0.6	0.2	—
Magnesium (Mg)	0.3	0.4	0.2	0.1	—
Sodium (Na)	0.4	—	0.01	—	—
Sulfur(S)	—	—	—	—	—
Carbon, total(C)	26	23	—	—	—
	Parts per million				
Boron (B)	—	—	14	—	—
Cadmium (Cd)	4.0	7.6	—	0.9	5.0
Copper (Cu)	146	300	23	3	96
Lead(Pb)	77	290	—	0.1	11.3
Manganese (Mn)	135	480	180	—	—
Nickel(Ni)	11	55	—	—	7
Strontium (Sr)	—	—	30	—	—
Zinc (Zn)	107	770	60	13	162

*Values expressed on dry-weight basis.

TABLE 8.61 Waste Characteristics as Percentages (63, 109, 110)

Component	Oceanside, N.Y. 1966–1967	Central Wayne County, Mich. plant, 1979	United States average		
			1977	1984	1993
Paper materials	3272–53.33	28.55–35.65	35.0	42.1	37.6
Plastics	245–8.82	3.39–6.31	3.8	6.5	9.3
Rubber and leather	—	—	—	2.3	3.0
Textiles	224–397	1.59–5.51	4.3	1.9	2.9
Garbage or organics	7.23–16.70	0.35–5.15	14.9	7.3	6.7
Wood and lumber	1.22–6.58	0.56–1.50	3.8	3.4	6.6
Yard wastes*	0.26–33.33	36.1–45.4	16.3	16.1	15.9
Noncombustibles	22.47–14.36	18.18–10.86	21.9	20.4	18
Total	—	—	100	100	100
Percent recycled	—	—	6.6	10.2	21.7

*Includes grass, dirt, and leaves.

road base, and beneficial use of the energy generated in the combustion process. Problems that must be addressed in the design and operation of such facilities are maintenance of good combustion conditions and achieving proper treatment of emissions so as to limit potential adverse effects on the public to the greatest extent possible.

Nature of the Fuel. Residential and commercial refuse is composed of combustible and noncombustible material and moisture. Combustible waste is made up largely of paper, together with some wood, vegetable and animal waste, cloth, leather, rubber, and plastics. The noncombustible fraction is composed of metals, glass, dirt and stones, and other miscellaneous materials. Larger materials, normally classified as rubbish, are frequently found in municipal solid waste (MSW). Table 8.61 illustrates the type of variation that has been observed in studies defining site-specific and average solid waste composition in the United States.

The table shows a 28.55 to 53.33% variation in paper content, and a 77.53 to 89.14% variation in combustible content. Moisture content has been found to vary from 20 to 50%. With little or no regulation of the handling of refuse by the homeowner, extreme variations in moisture content (and, so also, heat content) may be observed in the solid waste. Thus, after a heavy rain, the moisture content of the solid waste may be so high that it may be difficult to sustain combustion.

TABLE 8.62 Variation in Heat Content of MSW

Moisture, %	Noncombustible, %			
	10		25	
	Combined %	Heat content, Btu/lb	Combined %	Heat content, Btu/lb
20	70	6580	55	5170
30	60	5640	45	4230
40	50	4700	35	3290
50	40	3760	25	2350

Note: 2.32 Btu/lb 1 kJ/kg.

Such studies of the average composition of solid waste are useful to develop a general understanding of the nature of the material that must be handled in an incineration facility. However, for the design of the total plant facility, it is important to understand the extent of variation in waste composition that will be experienced at an incineration facility. The combustion system must be able to handle the extremes in solid waste heat content, both high and low.

As indicated above, most of the combustible fraction of MSW is cellulose. The remainder of the combustible content is composed of various fats, oils, waxes, rubbers, and plastics. The heat released by burning cellulose is approximately 8000 Btu/lb (18.6 MJ/kg), whereas that released by virtually all of the other combustible components is significantly higher on a per pound basis. In recent years, it has been found that the higher heating value (HHV) of the combustible portion only of MSW (moisture and ash free) averages about 9400 Btu/lb (21.8 MJ/kg). Taking that figure as the moisture- and ash-free heat content of MSW, Table 8.62 illustrates the variation in as-received heat content that one would expect in solid waste with moisture content ranging from 20 to 50% by weight and a noncombustible content of 10 and 25% by weight.

For the design of feeding and residue-handling systems, one must have some information on the variability and extremes of the physical size and shape of the solid waste, together with the variation in noncombustible content. These have been troublesome areas of plant operation. If materials-handling facilities for feeding and residue handling are not dimensionally large enough to pass the largest bulky items in the MSW, or large enough and rugged enough to handle the quantities of materials required to meet plant design capacity, the plants will suffer continued expensive periods of downtime and might have to be derated.

The problems noted above may be compounded by inclusion of industrial and/or hazardous waste with the MSW. Because of the potential impact of large quantities of very high heat content industrial wastes on heat generation and air emissions, such materials should be specifically identified and quantified prior to facility design. Likewise, hazardous industrial wastes would potentially impose serious design constraints on such a facility, both from the standpoint of operator safety and public health. Public relations problems related to public health concerns probably would rule out accepting hazardous industrial wastes at most, if not all, facilities intended to combust MSW.

Plant Design. The capacity to be provided in an incineration plant is a function of (1) the area and population to be served; (2) the number of shifts (one, two, or three) the plant is to operate; and (3) the rate of refuse production for the population served. If records of collections have been kept, especially by weight, forecasts for determining required plant capacity can be made with reasonable accuracy. If records are not available, refuse quantities for establishing plant size may be approximated by assuming refuse generation rates of 4 lb (1.8 kg) per capita per day, when there is little or no waste from industry, to 5 lb (2.3 kg) per capita per day when there is some waste from industry (64). If substantial quantities of industrial wastes are to be handled in an incineration plant, they should be specifically identified as to quantity and suitability for disposal in the planned facility. Those wastes not suitable for disposal (highly flammable, low heat content, and hazardous wastes, etc.) should be specifically excluded. A small plant [100 tons/day (90 metric tons/day)] will probably operate one shift per day. For capacities above 400 tons/day (360 metric tons/day), or any size plant incorporating boilers, economic and/or equipment operating considerations usually will dictate three-shift operation.

An isolated site is preferred for any such plant to avoid, as much as possible, objections of neighbors. However, well-designed and well-operated incinerators which do not present a nuisance may also be installed in light industrial and commercial areas, thereby avoiding the economic burden of extended collection truck routes or extensive refuse transfer operations. Since considerable vertical distance is involved in passing refuse through an incinerator, there is an advantage in a sloping or hillside site. Collection and transfer trucks can then deliver refuse at the higher elevation while residue trucks can operate at the lower elevation with a minimum of site grading.

Refuse Receipt and Storage. Scales, preferably integrated into an automated record-keeping system, should be provided to record the weight of solid waste delivered to the plant. Sufficient length of entrance

road and tipping floor area should be provided so that refuse delivery trucks do not back up onto public highways during peak delivery times. Either the tipping area or the individual tipping positions should be enclosed so as to prevent nuisance conditions in the vicinity of the plant caused by blowing papers, dust, and/or odors. The tipping area should be large enough to permit more than one truck at a time to maneuver to and from the dumping position.

Since collections usually are limited to one 8-b daily shift five days per week (sometimes with partial weekend collection operations), while burning will usually be continuous, ample storage must be provided. This usually requires two to three days of refuse storage at most energy-from-waste plants. Seasonal and cyclic variations should also be considered in establishing plant storage requirements.

When computing the dimensions required for refuse storage facilities, the required volume should be computed on the basis of an MSW bulk density of from 300 to 400 lb/yd³ (180 to 240 kg/m³)(65). Other factors to consider in sizing and laying out the refuse storage facilities are that refuse flows very poorly and can maintain an angle of repose of greater than 90°. Thus, refuse is commonly stacked in the storage facilities to maximize storage capability. Refuse storage in larger plants is normally in long, narrow, and deep pits either extending along the front of the furnaces or split in two halves extending from either side of the front end of the furnaces. If the storage pit is over 25 ft in width, it will generally be necessary to rehandle the refuse dumped from the trucks. In smaller plants, floor dumping and storage of refuse is common practice.

Refuse Processing. With the increasing interest in utilizing the heat generated in the combustion of MSW, in the late 1960s and early 1970s, a number of people proposed that the refuse be processed to produce a combustible-rich fraction. The first such processing plant was built in St. Louis, Mo., as an EPA demonstration plant in the early 1970s. The combustible-rich fraction produced was called refuse-derived fuel, or RDF. The process consisted basically of shredding the as-received refuse, air classification, and separation of non-combustible materials into recyclable fractions. In some plants, the combustible fraction was shredded a second time, following air classification, to produce a smaller particle size. It was originally proposed that RDF be cofired in coal-fired utility boilers with the material being completely combusted while in suspension.

A number of problems were identified in the early operation of these plants. With the initial shredding of as-received waste, glass was shattered and fine shards were embedded in paper and other combustible material. The glass, once embedded, could not be effectively removed. This increased the ash content of the combustible material and increased the abrasion of the pneumatic RDF conveying equipment.

Further, although the heat content per pound of RDF reportedly was increased by 10 to 15% over the heat content of as-received MSW (66), there was a net loss of energy from the system. Thus, from Table 8.63, it can be observed that with a 10% increase in heat content and 70% capture of combustibles in the processing system, the resulting RDF contains only 77% of the heat of the original quantity of as-received MSW. With a 15% increase in heat content and 80% capture of combustibles, slightly more than 91% of the heat in the original MSW would be contained in the RDF. A much more extensive comparison of the efficiency of various waste-processing systems in extracting the energy contained in MSW is presented in Ref. 67.

Another problem noted (68) from operation of early RDF combustion facilities was caused by incom-

TABLE 8.63 Input Heat Capture in RDF

Material	Weight, lb	Heat content		
		Per lb	Total	Percent original MSW
As-received MSW	2000	4500	9,000,000	100.0
RDF (70% of MSW)	1400	4950	6,930,000	77.0
RDF (80% of MSW)	1600	5175	8,280,000	91.3

Note: 1 lb = 0.454 kg.

plete combustion of the RDF in suspension requiring use of dump grates to allow completion of combustion and reduce the load on ash-handling facilities. Also, there has been some evidence, although not entirely conclusive, of increases in slagging in the boilers, and some problems have been reported with metal wastage of boiler tubes, particularly in the lower waterwall areas.

Later RDF processing plants have used flail mills, or trommels with bagbreaking blades, to break apart bags containing the waste, allowing initial separation of glass and metal. The material passing through the trommel, primarily the light combustible fraction, is then shredded. Removing the glass prior to shredding alleviates the problem of contamination of combustible with glass shards. Most plants now anticipate burning RDF in boilers with spreader stokers rather than in suspension-fired utility boilers. Potential problems of slagging and boiler tube metal wastage must be considered when selecting boilers for RDF combustion.

Processes to produce powdered fuel or RDF fuel pellets, while interesting, have not been developed to a state of commercial availability. Some limited work is still being undertaken to improve the economics and operability of such systems. However, it appears as if commercial availability is many years in the future. Other processes such as pyrolysis have not been successfully applied to this field (69).

Refuse Feeding. Batch feeding of MSW and/or batch discharge of residue is undesirable because of variations in furnace temperatures of several hundred degrees that usually occur due to air leakage into the furnace, resulting in adverse impact on refractory materials, and increased air emissions. In smaller plants with floor dump and storage of MSW, feeding is accomplished on a semibatch basis by rams that push material directly into the furnace on approximately 6- to 10-min cycles.

In larger plants utilizing pits for refuse storage, the solid waste is normally moved from the pit to a charging hopper (70) by a traveling bridge crane and a grapple or organic-peel type of bucket (see Figure 8.44).

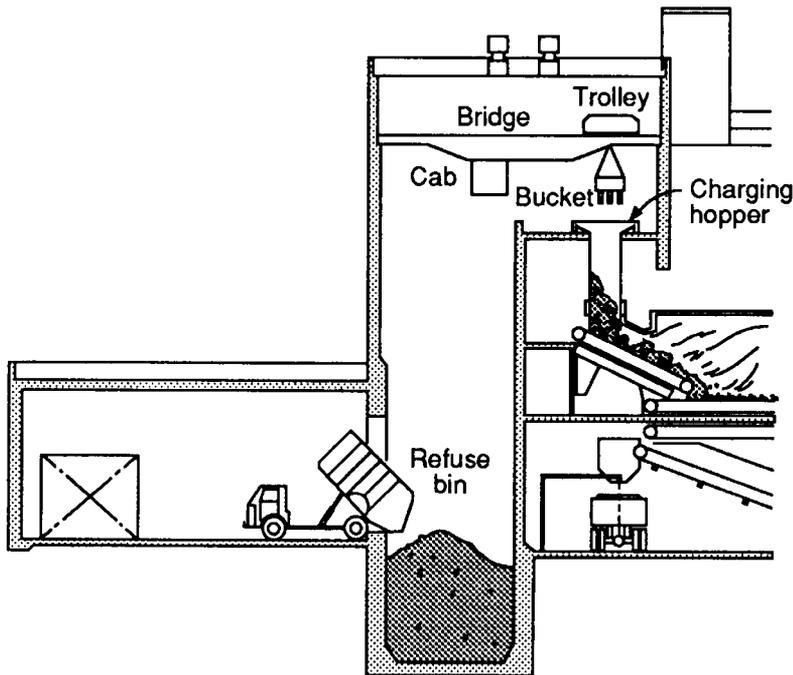


FIGURE 8.44 Bridge crane for municipal incinerator (70).

The grapple or bucket size is established by a duty-cycle analysis, taking into account the quantity of material that must be moved from the pit to the furnaces, the distances over that the material must be moved, allowable crane speeds, and the need to rehandle (mixing and/or stacking) material in the pit. Buckets or grapples used to date have ranged in capacity from 1.5 to 8 yd³ (1 to 6 m³).

The crane used in this service should be capable of meeting the severest of duty requirements (71). The load-lifting capability is established by adding to the bucket or grapple weight, 1.5 times the volumetric capacity of the bucket times a density of MSW of 600 to 800 lb/yd³ (360 to 480 kg/m³) (65). In the past, the crane has been operated from an air-conditioned cab mounted on the bridge. However, in many European plants, and with increasing frequency in the United States, crane operation is being centralized in a fixed control room usually located at the charging floor elevation and either over the tipping positions opposite the charging hoppers, or in the vicinity of the charging hoppers.

In modern furnaces burning as-received MSW on mechanized grates, the crane-mounted grapple or bucket is used to lift the refuse from the pit to deposit it in a charging hopper. The charging hopper, which is built large enough to prevent spillage on the charging floor and with slopes steep enough to prevent bridging, is placed on top of a vertical feed chute that discharges the MSW into the furnace. The feed chute is normally constructed of water-cooled steel plates or steel plates lined with smooth refractory material. The chute is normally at least 4 ft (1.2 m) wide, to pass large objects with a minimum of bridging, and 12 to 14 ft (3.6 to 4.2 m) long. It is normally kept full of refuse to prevent uncontrolled admission of air into the furnace. The refuse is fed from the bottom of the feed chute into the furnace by a portion of the mechanical grate, or by a ram. The ram generally provides better control of the rate of feed into the furnace than the older technique of using a portion of the mechanical grate for refuse feed.

In other plants, particularly those burning RDF, conveyors, live-bottom bins, and shredding and pneumatic handling of combustible material have been utilized. Problems related largely to properly sizing the equipment and higher than expected maintenance have been experienced to date with these facilities.

Residue Handling. The residue from a well-designed, well-operated mass-fired incinerator burning as-received refuse will include the noncombustible material in the MSW plus somewhat less than 5% of the combustibles. The nature of this material will vary from relatively fine, light ash, burned tin cans, and partly melted glass, to large bulky items such as 55-gal drums. The material may be discharged from the furnace through manually operated dump gates, or directly from the mechanical grate into a hopper where it is quenched and then discharged to a truck or container positioned below the hopper, through a bottom gate. The residue may also be discharged through a chute into a trough filled with water. Removal from the trough may be either by a ram discharger onto a conveyor or by a flight conveyor to an elevated storage hopper from which it is discharged to a truck. If a water-filled trough with a flight conveyor is used, normally two troughs are provided, arranged so that the residue can be discharged through either trough. The second trough serves as a standby.

A key feature in the design of ash-discharge facilities is provision for sealing the discharge end of the furnace to prevent uncontrolled admission of air. This seal is usually provided by carrying the ash-discharge chute at least 6 in (15 cm) below the water surface in the receiving trough. In the design of the conveyor mechanism, the proportions should be large because the material frequently contains bulky metal items and wire causing relatively frequent jamming, and it tends to be extremely abrasive.

Residue is taken to a landfill for final disposal. The volume of material remaining for ultimate disposal will range from 5 to 15% of that received at the plant. Many plants currently operating in the United States that weigh MSW received at the plant and residue discharged from the furnaces indicate that the weight of MSW is only reduced from 50 to 60%. However, as much as one-third of the residue weight in these plants may be attributed to incomplete drainage of the material prior to its discharge into the final transportation container. The ram-type ash discharger used in European and some of the new, large U.S. plants generally achieves much better dewatering of residues than older water-filled trough, ash drag residue-handling systems.

The residue from incinerators is generally inert, relatively sterile (with combustible content below 5%) and makes good landfill material, particularly if it is well dewatered at the plant. There is some indication that heavy metals tend to concentrate in fly ash collected in such plants, rather than in bottom ash. If more extensive data confirm these preliminary indications, and recognizing the increasing concern regarding groundwater quality, it may be necessary to handle plant residues separately prior to disposal in the future.

Furnace Design. The principal aspects of furnace design are capacity (grate area and furnace volume), mechanical grates, construction refractories, and combustion air supply.

Capacity. The basic design factors that determine furnace capacity are grate area and furnace volume. Also, the available capacity and method of introducing both underfire and overfire air will influence, to a lesser extent, furnace capacity. Required grate area, in a conservative design, is normally determined by limiting the burning rate to between 60 and 70 lb/ft²/h (290 and 340 kg/m²/h) of grate area (72). This is based on limiting the heat release rate loading on the grate to 250,000 and 300,000 Btu/ft² of grate per hour (2.8×10^6 to 3.4×10^6 kJ/m²/h).

Furnace volume required is established by the rate of heat release from the fuel. Thus, furnace volume is generally established by using heat release rates ranging from 12,500 to 20,000 Btu/ft³/h (4.6×10^5 to 7.4×10^5 kJ/m³/h) with the lower heat release rate being more desirable from the standpoint of developing a conservative design. A conservative approach to design in this area is desirable because of probable periodic operation at above design capacity to meet short-term, higher than normal refuse collections, and possible receipt of high heat content waste.

Waterwall units burning as-received MSW have been built as small as 75 to 100 tons/day (68 to 91 t/day) capacity. However, the cost per ton of rated capacity of such units is relatively high. A more common unit size is 250 to 300 tons/day (225 to 270 t/day), while waterwall mass-fired units have been built as large as 750 to 1200 tons/day (675 to 1090 t/day) capacity (73).

Grates. The primary objective of a mechanical grate is to convey the refuse from the point of feed through the burning zone to the point of residue discharge with a proper depth of fuel and sufficient retention time to achieve complete combustion. The refuse bed should be gently agitated so as to enhance combustion. However, the agitation should not be so pronounced that particulate emissions are unreasonably increased. The rate of movement of the grate or its parts should be adjustable to meet varying conditions or needs in the furnace.

In the United States over the past 20 years, several types of mechanical grates have been used in continuous-feed furnaces. These include traveling grates, reciprocating grates, rocking grates, and a proprietary water-cooled rotary combustor. The traveling grate conveys the refuse through the furnace on the grate surface. Stirring is accomplished by building the grate in two or more sections, with a drop between sections to agitate the material. The reciprocating and rocking grates both agitate and move the refuse material through the furnace by the movement of the grate elements and the incline of the grate bed. Additional agitation is obtained, particularly in the reciprocating grate, by substantial drops in elevation between grate sections. The rotary combustor slowly rotates to tumble the refuse material that is conveyed through the inside of the cylinder. The combustor is inclined from the horizontal so that gravity assists in moving the material through the unit.

The Europeans have developed other grate systems, some of which are currently being utilized in plants being constructed or in operation in the United States. The Volund incinerator (Danish) uses a slowly rotating, refractory-lined cylinder or kiln, which is fed by a two-section (drying and ignition) reciprocating grate. Refuse passes through the kiln and residue is discharged to a water quench when combustion is completed. The so-called Dusseldorf or VKW (German) incinerator uses a series of six rotating cylindrical grates, or drums, placed at a slope of about 30° (74). The refuse is conveyed by the surface of the drums, which rotate in the direction of refuse flow, and is agitated as it tumbles from drum to drum. Underfire air is introduced through the surface of the drums. Both the Von Roll and the Martin grates use a reciprocating motion to push the refuse material through the furnace. However, in the Martin grate, the grate surface slope is greater

and the grate sections push the refuse uphill against the flow of waste causing a gentle tumbling and agitation of the fuel bed.

Another variable feature in the various grate designs is the percentage of open area to allow for passage of underfire air (75). These air openings vary from approximately 2 to over 30% of the grate surface area. The smaller air openings tend to limit the quantity of siftings dropping through the grates and creates a pressure drop that assists in controlling the point of introduction of underfire air. Larger air openings make control of underfire air more difficult but allow for continuous removal of fine material, which could interfere with the combustion process, from the fuel bed.

Furnace configuration is largely dictated by the type of grate used. In the continuous-feed mechanical grate system, the furnace is rectangular in plan and the height is dependent upon the volume required by the limiting rate of heat release cited earlier. An optimum furnace configuration would provide sufficient volume for retention of gases in the high-temperature zone of maximum fuel volatilization for a sufficient length of time to ensure complete combustion, and would be arranged so that the entire volume is effectively utilized. Temperatures are usually high enough with present-day refuse for proper combustion. Turbulence should be provided by a properly designed overfire air system. Figure 8.45 shows an example of a mass-fired waterwall boiler system.

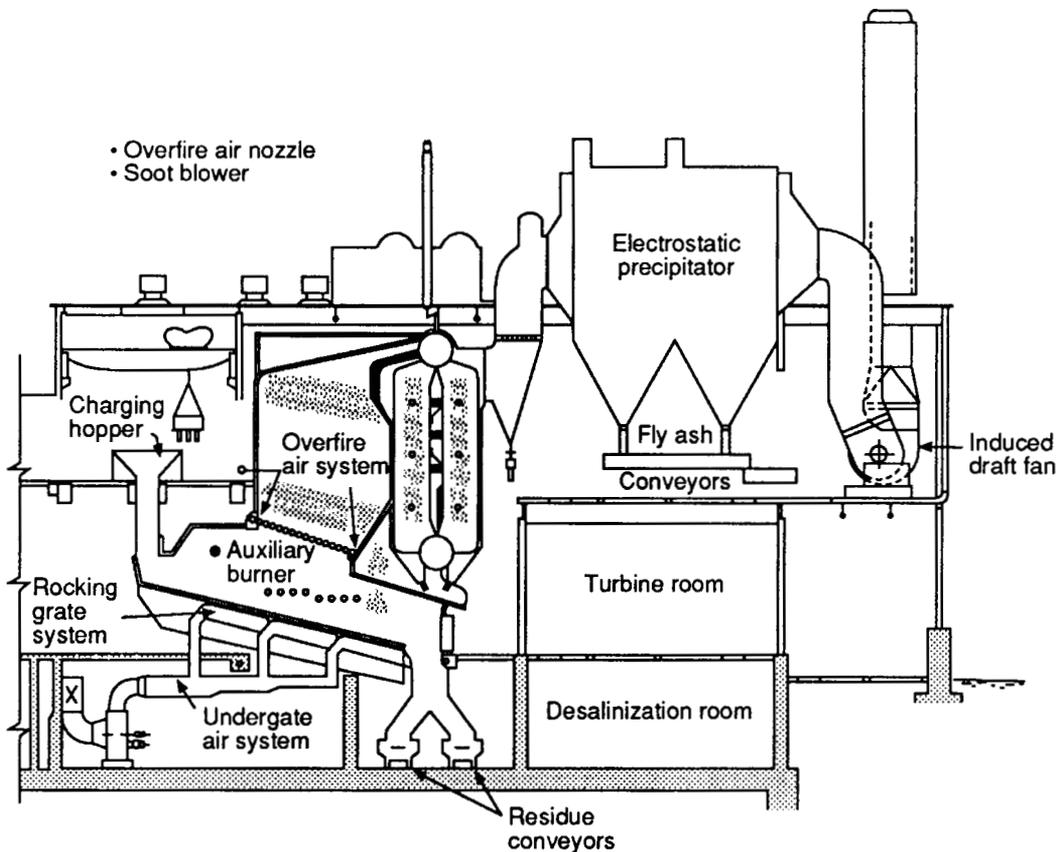


FIGURE 8.45 Refuse furnace, waste-heat boiler, and electrofilter sections (76).

Refractories. With present-day mass-fired waterwall furnaces, the use of refractories in furnace construction has been minimized but not eliminated. Refractory materials may be used to line charging chutes, provide a transition enclosure between the top of the grates and the bottom of the waterwalls, a protective coating on the waterwall tubes, and an insulating layer between the hot gases and the metal walls of flues downstream of the primary combustion chamber. Refractory brick used in a charging chute must be able to withstand abrasion and provide some insulation in the lower portions for protection from moderate temperatures. The construction above the grates must be able to withstand high temperatures, flame impingement, thermal shock, slagging, spalling, and abrasion. The protective coating on the waterwall tubes must be a relatively dense castable material with a relatively high heat conductivity (76). Insulating refractories used in flues downstream from the boilers, on the other hand, should have a low heat conductivity.

Refractories are generally classified according to their physical and chemical properties, such as resistance to chemical attack, hardness, strength, heat conductivity, porosity, and thermal expansion (77). The material may be cast in brick in a variety of shapes and laid up with air-setting or thermal-setting mortar, or may be used in a moldable or plastic form. Material used in incinerator construction includes “high-duty” and “superduty” fireclay brick, phosphate-bonded alumina material, and silicon carbide, among others. In selecting the proper materials for application in this type of service, because the selection of materials is so great and the conditions of service so varied and severe (78), advice of a recognized manufacturer should be sought.

Combustion Calculations. Factors directly affecting furnace design are moisture and combustible content of the solid waste being burned, the volatility of the material being burned, and the means for temperature control. The design of furnaces and boilers and sizing of flues and other plant elements should be based on design parameters that result in large sizes. Controls should provide satisfactory operation for loads below the maximum.

The combustion portion of MSW is composed largely of cellulose and similar materials originating from wood, mixed with appreciable amounts of fats, oils, waxes, rubbers, and plastics. The heat released by burning cellulose is approximately 8000 Btu/lb (18.6 MJ/kg) while that released by fats, oils, etc., is approximately 17,000 Btu/lb (39.5 MJ/kg). If cellulose and oil and fat exist in the ratio of 6:1 in MSW, the heat content of the combustible matter in MSW would be approximately 9290 Btu/lb (21.6 MJ/kg).

The heat released during combustion may be determined in a bomb calorimeter, a device with a metal container (bomb) immersed in a water jacket. The heat absorbed by the water is the heat of combustion. The heat of combustion of a number of materials is shown in Table 8.64.

Another method for determining the approximate heat value for solid fuels is to perform an ultimate analysis and then apply Dulong’s formula. This formula may be stated as

TABLE 8.64 Heat of Combustion

Material	Heat of combustion, Btu/lb
Carbon (to CO ₂)	14,093
Hydrogen	61,100
Sulfur	3,983
Methane	23,870
Ethylene	21,644
Oil (#3–#6)	17,500–19,000
Coal (bituminous)	12,000–14,500
MSW (as received)	3,500–5,500

Note: 1 Btu/lb = 0.43 kJ/kg.

$$\text{Btu/lb} = 14,544C + 62,028\left(\text{H}_2 - \frac{\text{O}_2}{8}\right) + 4050S$$

where C, H₂, O₂, and S represent the decimal proportionate parts by weight of carbon, hydrogen, oxygen, and sulfur in the fuel. The term O₂/8 is a correction used to account for hydrogen that is already combined with oxygen in the form of water.

For the purposes of illustrating the calculations required to determine refuse heat content using Dulong's formula, air requirements, and furnace temperature, the ultimate analysis of MSW in Table 8.65 is used. In the determination of air requirements, Table 8.66 combustion constants will be used.

With the ultimate analysis shown in Table 8.65 and the combustion constants shown in Table 8.64, a calculation can be made of input and output gas quantities for this given refuse composition. In the calculations, the following assumptions are made: base temperature = 80°F (27°C), 23.15% of air is O₂ and 76.85% is N₂ moisture in the air = 0.0132 lb/lb dry air; and unburned carbon in residue = 4% of carbon input. Gas quantities are tabulated on Tables 8.67 and 8.68 for two conditions: 140% excess air (EA) (refractory furnace where temperatures are controlled by adding excess air), and 80% excess air (waterwall furnace where heat is absorbed from combustion chamber by water circulating in the waterwall furnace enclosure).

Next a material balance can be calculated (see Table 8.69) for both excess air conditions. The following additional assumptions are made in performing these calculations: residue quench water evaporated = 0.03 lb/lb of MSW, and fly ash = 2% of MSW burned.

A heat balance and check on flue gas temperature assumptions (see Table 8.70) can now be performed by

TABLE 8.65 Ultimate Analysis

Component	Weight %, total	Weight, % combined
C	25.0	50.0
H ₂	4.2	8.4
O ₂	20.7	41.4
S	0.1	0.2
H ₂ O	28.0	—
Noncombustible		
N ₂	0.5	—
Ash	21.5	—
	100.0	100.0
Heat content, moisture and ash free		
$\text{Btu/lb} = 14,544 \times 0.5 + 62,028 \left(0.084 - \frac{0.414}{8}\right) + 4050 \times 0.002$		
$\text{Btu/lb} = 7272 + 62,028 \times 0.03225 + 8$		
$\text{Btu/lb} = 7272 + 2000 + 8$		
$\text{Btu/lb} = 9280 \text{ Btu/lb}$		
Heat content, as received (complete combustion)		
$\text{Btu/lb} = 9280 \times 0.5 = 4640 \text{ Btu/lb}$		

Note: 1 Btu/lb = 0.43 kJ/kg.

TABLE 8.66 Combustion Constants (79)

Substance*	Formula	Molecular Weight	lb/ ft ³	ft ³ /lb	Theoretical (stoichiometric) air, lb/lb combined					
					Required for combustion			Flue products		
					O ₂	N ₂	Air	CO ₂	H ₂ O	N ₂
Carbon	C	12.01	—	—	2.66	8.86	11.53	3.66	—	8.86
Hydrogen	H ₂	2.016	0.0053	187.723	7.94	26.41	34.34	—	8.94	26.41
Oxygen	O ₂	32.000	0.0846	11.819	—	—	—	—	—	—
Nitrogen (atm.)	N ₂	28.016	0.0744	13.443	—	—	—	—	—	—
Sulfur	S	32.06	—	—	1.00	3.29	4.29	2.00	—	3.29
Water vapor	H ₂ O	18.016	0.0476	21.017	—	—	—	—	—	—
Air	—	2.89	0.0766	13.063	—	—	—	—	—	—

* All gas volumes corrected to 60°F and 30 in Hg dry.

Note: 1 lb/ft³ = 0.06 kg/m³.

TABLE 8.67 Gas Quantity Calculations—Stoichiometric or Theoretical

Component	O ₂		Air		
	Fractional composition	Combustion constant	Q _{O₂} , lb/lb fuel	Combustion constant	Q _{air} , lb/lb fuel
Carbon					
Burned	0.240	2.66	0.638	11.53	2.767
Lost	0.010	—	—	—	—
Hydrogen	0.042	7.94	0.333	34.34	1.442
Oxygen	0.207	—	—	—	—
Nitrogen	0.005	—	—	—	—
Sulfur	0.001	1.00	0.001	4.29	0.004
Moisture	0.280	—	—	—	—
Ash	0.215	—	—	—	—
Total	1.000	—	0.972	—	4.213
Less O ₂ in fuel (deduct)	—	—	-0.207	(1/0.2315) ×	-0.894
Required at theoretical air	—	—	0.765	—	3.319
O ₂ and air @ 80% EA (1.8 × Q) =	—	—	1.377	—	5.974
EA = 5.974 - 3.319	—	—	—	—	2.655
E O ₂ = 1.377 - 0.765	—	—	0.612	—	—
O ₂ and air @ 150% EA (2.5 × Q) =	—	—	1.913	—	8.298
EA = 8.298 - 3.319	—	—	—	—	4.979
E O ₂ = 1.913 - 0.765	—	—	1.148	—	—

TABLE 8.69 Materials Balance

Input	@ 80% excess air		@ 150% excess air	
	Subtotal, lb/lb	Total lb/lb	Subtotal, lb/lb	Total lb/lb
Refuse				
Combustible material				
Complete combustion	0.49	0.49		
Unburned C	0.01	0.01		
Moisture	0.28		0.28	
Noncombustible	<u>0.22</u>	1.00	<u>0.22</u>	1.00
Total air				
O ₂	1.377		1.913	
N ₂	<u>4.591</u>	5.97	<u>6.377</u>	8.29
Moisture in air		0.08		0.11
Residue quench water		<u>0.03</u>		<u>0.03</u>
Total		7.08		9.43
CO ₂		0.88		0.88
Air				
O ₂	0.612		1.148	
N ₂	<u>4.591</u>	5.20	<u>6.377</u>	7.52
Moisture				
in MSW	0.280		0.280	
from combustion	0.375		0.375	
from combustion air	0.079		0.110	
from residue quench water	<u>0.030</u>	0.76	<u>0.030</u>	0.80
Noncombusted material				
Noncombustible	0.220		0.220	
Unburned C	<u>0.010</u>	0.23	<u>0.010</u>	0.23
Unaccounted for		<u>0.01</u>		<u>—</u>
Total		7.08		9.43

use of Figure 8.46 developed from similar figures in Ref. 78, and the following assumptions: the specific heat of both fly ash and residue = 0.25; and the temperature of the residue = 180°F (82°C).

Since the computation of heat input and heat output balances with minimal unaccounted for losses, the assumed temperatures are satisfactory. If the unaccounted for losses are greater than the minimum necessary to make the heat calculations balance, the assumed temperature should be adjusted upward or downward to bring the calculations more in balance.

The calculations for the 80% excess air calculations may be carried one step further to estimate steam-generating capability and anticipated boiler efficiency for different assumed conditions. Thus, assuming temperature of the gases leaving the boiler was 500°F (260°C), steam is generated at 125 lb/in² gage (860 kN/m²), and 400°F (205°C) [$h_f = 1221$ Btu/lb (2.8 MJ/kg)], and the heat content of the boiler feedwater at 181°F (83°C) was $h_s = 181$ Btu/lb (0.42 MJ/kg), the calculation in Table 8.71 would be made.

Detailed calculations for boiler design are beyond the scope of the material presented in this section. Sev-

TABLE 8.70 Heat Balance

	@ 80% excess air		@ 150% excess air	
	Calculation	Total, Btu/ lb/fuel	Calculation	Total, Btu/ lb/fuel
Heat input				
Refuse		4640		4640
Moisture in air	0.08×1.048	84	0.11×1.048	115
Total		4724		4755
Heat output				
Dry gas	Gas temp. = 1860°F $(0.88 + 5.20) \times 476$	2894	Gas temp. = 1460°F $(0.88 + 7.42) \times 360$	3024
Water vapor	0.76×2009 (steam tab.)	1527	0.80×1789 (steam tab.)	1431
Fly ash	$0.02 \times 0.25 \times (1860 - 80)$	9	$0.02 \times 0.25 \times (1460 - 80)$	7
Residue	$0.22 \times 0.25 \times (180 - 80)$	6	$0.22 \times 0.25 \times (180 - 80)$	6
Unburned carbon	$0.01 \times 14,093$	141	$0.01 \times 14,093$	141
Loss through furnace enclosure	0.03×4724	142	$0.03 \times 4,755$	143
Unaccounted for		5		3
Total		4724		4755

Note: 1 Btu/lb = 0.43 kJ/kg.

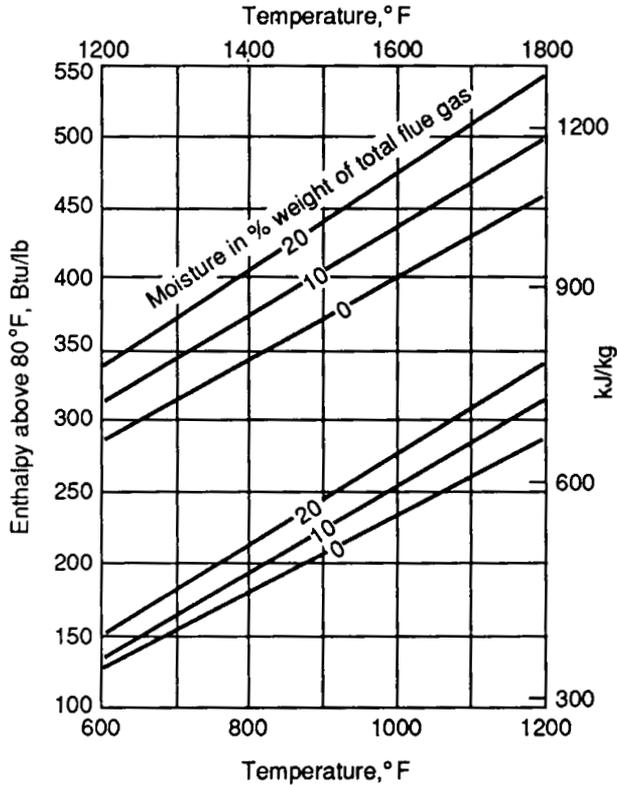


FIGURE 8.46 Enthalpy of flue gas (64).

eral of the major boiler manufacturers in this country have written comprehensive texts on this subject (79, 80) and the reader is referred to this material and the boiler manufacturers for detailed information on this subject.

Combustion Air Supply. As indicated in the section on combustion calculations, the combustion process requires oxygen to complete the reactions involved in the burning process. The air that must be delivered to the furnace to supply the exact amount of oxygen required for completion of combustion is called the stoichiometric air requirement. Additional air supplied to the furnace is called excess air and is usually expressed as a percentage of the stoichiometric requirements.

The total air supply capacity in an incinerator must be more than the stoichiometric requirement for combustion because of imperfect mixing and to assist in controlling temperatures, particularly with dry, high-heat-content refuse. The total combustion air requirements can range to 10 lb of air per pound of refuse for refractory wall furnaces and from 6 to 8 lb of air per pound of refuse for mass-fired waterwall furnaces.

In the modern mechanical grate furnace chamber, at least two blower systems should be provided to supply combustion air to the furnace—one for underfire or undergrate air and the other for overfire air. Underfire air, admitted to the furnace from under the grates and through the fuel bed, is used to supply primary air to the combustion process and, secondarily, to cool the grates.

Overfire air may be introduced in two levels. Air introduced at the first level, immediately above the fuel bed, is used to promote turbulence and mixing and to complete the combustion of volatile gases driven off

TABLE 8.71 Boiler Calculations

Useful heat output

Dry gas	2894
Water vapor	<u>1527</u>
Total	4421 Btu/lb fuel

Heat in gases at boiler outlet at 500°F (260°C)

Dry gas	$6.08 \times 103 = 626$
Water vapor	$0.76 \times 1287 = \underline{978}$
Total	1604 Btu/lb fuel

Heat required for 1 lb steam

$$1221 - 181 = 1040 \text{ Btu/lb steam}$$

Steam produced per pound of fuel

$$\frac{(4421 - 1604)}{1040} = \frac{2817}{1040} = 2.71$$

Boiler efficiency

$$\frac{(4421 - 1604)}{4724} \times 100 = \frac{2817}{4724} \times 100 = 59.6\%$$

Note: 1 Btu/lb = 0.43 kJ/kg; 1 lb = 0.454 kg.

the bed of burning solid waste. The second row of nozzles, which are higher in the furnace wall, allows secondary overfire air to be introduced into the furnace to promote additional mixing of the gases and for temperature control.

Blower capacities should be divided so that the underfire blower is capable of furnishing half or more of the total calculated combustion air requirements, while the overfire blower would have a capacity of somewhat less than half of the total calculated air requirements. Setting these capacities requires some judgment related to assessing how great a variation is anticipated in refuse heat contents during plant operation. Dampers should be provided on fan inlets and on air distribution ducts for control purposes.

Pressures on underfire air systems for most U.S. types of grates will normally range from 2 to 5 in (5 to 12.7 cm) of water. European grate systems frequently require a higher pressure. The pressure on the overfire air should be high enough so that the air, when introduced into the furnace, produces adequate turbulence without impinging on the opposite wall. This is normally accomplished by the use of numerous relatively small 1.5 to 3 in (3.8 to 7.6 cm) diameter nozzles at pressures of 20 in (51 cm) of water and higher.

Boilers. Substantial quantities of heat energy may be recovered during the thermal destruction of the combustible portions of MSW. Systems that have been successfully used to recover this energy include mass-fired refractory combustion chambers followed by a convection boiler section; an RDF semisuspension fired spreader-stoker-boiler unit; and an RDF suspension-burning utility type of boiler. Each system has apparent advantages and disadvantages.

Mass-Fired. In a refractory furnace waste-heat boiler unit, energy extraction efficiencies are generally lower, assuming the same boiler outlet temperatures, than with the other systems. Approximately 50 to 60%

of the heat generated in the combustion process may be recovered with such systems. These units can produce approximately 2 lb steam per pound of normal MSW [heat content = 4500 Btu/lb (10.5 MJ/kg)], versus 3 lb/lb MSW in the other units described above. This lower efficiency of steam generation is caused by larger heat losses due to higher combustion air quantities needed with such units to control furnace temperatures so that furnace refractories are not damaged. However, the boilers in such units, if properly designed and operated, generally are less susceptible to boiler tube metal wastage problems than the other systems listed above.

Mass-fired waterwall units are perhaps the most widely utilized type of heat recovery unit in the field today. In this type of unit, the primary combustion chamber is fabricated from closely spaced steel tubes through which water circulates. This waterwall-lined primary combustion chamber is followed by a convection type of boiler surface. It has been found desirable in these plants to coat a substantial height of the primary combustion chamber, subject to higher temperatures and flame impingement, with a thin coating of a silicon carbide type of refractory material, and to limit average gas velocities to under 15 ft/s (4.5 m/s). Gas velocities entering the boiler convection bank should be less than 30 ft/s (9.0 m/s) (76). Efficiency of heat recovery in such units has been found to range generally from 65 to 70%, with steam production usually about 3 lb of steam per pound of normal MSW. Water table studies have been found to be very useful in the larger units to check on combinations of furnace configuration and location of overfire combustion air nozzles.

Semi-Suspension-Fired. In an RDF-fired spreader-stoker type of unit, the combustible material is generally introduced through several air-swept spouts in the front waterwall, is partially burned in suspension, and then falls onto a grate on which combustion is completed as the partially burned material is conveyed to the residue discharge under the front waterwall face of the furnace. These units can generally handle a coarser RDF than the so-called full-suspension burning units. Densified RDF can also be burned in such units. The RDF can furnish all the combustible input to the system, or it can be cofired with a fossil fuel, generally coal. While it was originally felt that such units could achieve more controllable combustion conditions than the mass-fired units, experience to date has not proven this concept.

Suspension-Fired. The so-called full-suspension combustion concept was originally proposed so that finely shredded combustible material from MSW could be burned in existing utility boilers. In this way, the expense of constructing a boiler would be mitigated and 10 to 15% of the fossil fuel normally consumed by the utility would be displaced (saved) by burning the RDF. This has been the least successful of the system types due to problems related to additional handling and greater power requirements to achieve a finer shred. Also, some utility boilers seemed to experience a greater tendency for slag formation in the boiler. While the concept initially anticipated that the RDF would completely burn in suspension, experience to date indicates that this does not occur. Accordingly, dump grates are now considered a necessity in such boiler units to allow for completion of combustion prior to water quenching of the residue.

Efficiencies of both RDF-fired types of boiler units will generally range from 65 to 75%. Steam production would normally be expected to be somewhat greater than 3 lb steam per pound of RDF.

Limitations. If the energy recovered from the combustion of as-received MSW or RDF is to be used to produce electricity, some superheat is at least desirable, if not necessary. Since boiler tube metal wastage in these plants is, at least partially, a function of tube metal temperature (83), and steam is a less efficient cooling medium than water, superheater surface is more prone to metal wastage problems than other areas of boiler tubing. Tube metal temperatures, above which metal wastage can be a significant operational problem, are generally thought to range from 650 to 750°F (395 to 450°C). These temperatures are lower than desirable for maximum efficiency of electrical generation by steam-driven turbines. However, this limitation does not rule out consideration of this form of energy utilization.

Air Pollution Control. Potential emissions from the burning of MSW may be broadly classified into particulates, gaseous emissions, incompletely burned products (primarily hydrocarbon) from the combustion process, and trace emissions. Particulates have been a matter of concern, and regulatory agency attention,

for some time. The initial concern was from the standpoint of reducing gross emissions that were both an aesthetic and a potential public health problem. Current interest and concern, since the initial problem has largely been solved, is directed toward better control of submicron-size particles (63).

Gaseous emissions, such as CO_2 , SO_2 , and NO_x , are not generally felt to be a major problem in an incineration plant. However, control measures may be required at a specific site if it is located in an area designated by EPA as nonattainment for some or all of the above pollutants. Incompletely burned products from the combustion process, such as CO and hydrocarbons, can be a problem if emission levels are not closely controlled. Thus, CO levels at poorly controlled MSW incinerators have been observed at well over 1000 ppm, an indication of the potential presence of unburned hydrocarbons in the stack gases and much higher than allowed in those jurisdictions that have established emission limits. Unburned hydrocarbons can cause odor problems, and, depending on the specific compounds, can be toxic. Of course, the most direct method of control for both CO and hydrocarbons is to achieve better control of the combustion process.

The existence, identification, and quantification of trace metal and gaseous (particularly chlorinated hydrocarbon) substances has been of increasing interest and concern over the last 15 to 20 years as increasingly sophisticated and complicated analytical equipment and procedures have been developed. Trace metal emissions can generally be controlled by better particulate control (84). Some extremely toxic gaseous materials have been identified in the emissions from MSW incineration plants over the past 15 years in the parts per billion and parts per trillion level. However, while the substances are extremely toxic, leading to great concern on the part of the public, sampling and analysis methods are still under development, raising questions as to the actual emission levels reported. Emission levels are usually so low that sampling and laboratory analysis procedures are frequently brought into question; the source or mechanism of formation of the toxins is still the subject of investigation, and projections of health risks are very approximate. Until better scientific investigations into this problem produce more rigorous information, concern for potential health problems from these emissions will continue.

Regulatory Requirements. Emission standards issued by the U.S. government in 1971, to date, are based on provisions of the Clean Air Act Amendments of 1970 and as subsequently amended. Different levels of allowable emissions of particulates and gases have been established for different processes and for the ambient atmosphere. The EPA has also established standard methods for testing and analyzing for these pollutants.

Over the past 15 years, allowable emission levels for particulates have been reduced significantly. Figure 8.47 illustrates the rather dramatic reduction in allowable particulate emission levels. Current permit levels usually are one-tenth the 1971 Federal New Source Performance Standards. Figure 8.48 illustrates the types of different air pollution control equipment available and its collection efficiency relative to particle size. The relatively rapid decrease in allowable emissions, and the current concern for submicron-size particles, has caused an evolution in applied control technology from settling chambers, to wetted-wall collectors, to wet scrubbers (tray type, venturi, etc.), to electrostatic precipitators, to baghouse collectors.

Flue Gas Tempering. Flue gases exiting the primary combustion chamber generally will range in temperature from 1500 to 1800°F (815 to 982°C) and will contain appreciable quantities of fly ash (approximately 15 to 20 lb per 1000 lb) (85). The fly ash loading must be reduced by over 99% to meet most current air pollution code requirements. This in turn indicates that equipment, such as electrostatic precipitators or baghouses, must be used for particulate control. Such equipment requires that entering gas temperatures be held to 500 to 600°F (260 to 315°C). Thus, flue gas temperatures must be reduced by approximately 1000°F (555°C). This may be accomplished either by use of boilers or by evaporation of water directly by the flue gases.

Calculations for absorption of heat by boilers have been covered earlier. Table 8.72 gives the requisite heat balance calculations for a spray chamber where flue gas (entering at 1680°F (915°C) and at 847 lb/h (385 kg/h) flow rate for dry gas + 73.1 lb/h (33 kg/h) moisture is cooled to 600°F (315°C) by water evaporating from sprays. The calculation results are expressed per 100 lb of 5000-Btu/lb MSW. There will be some heat loss through the furnace walls, estimated at 3% of the heat input. Air leakage into the furnace and

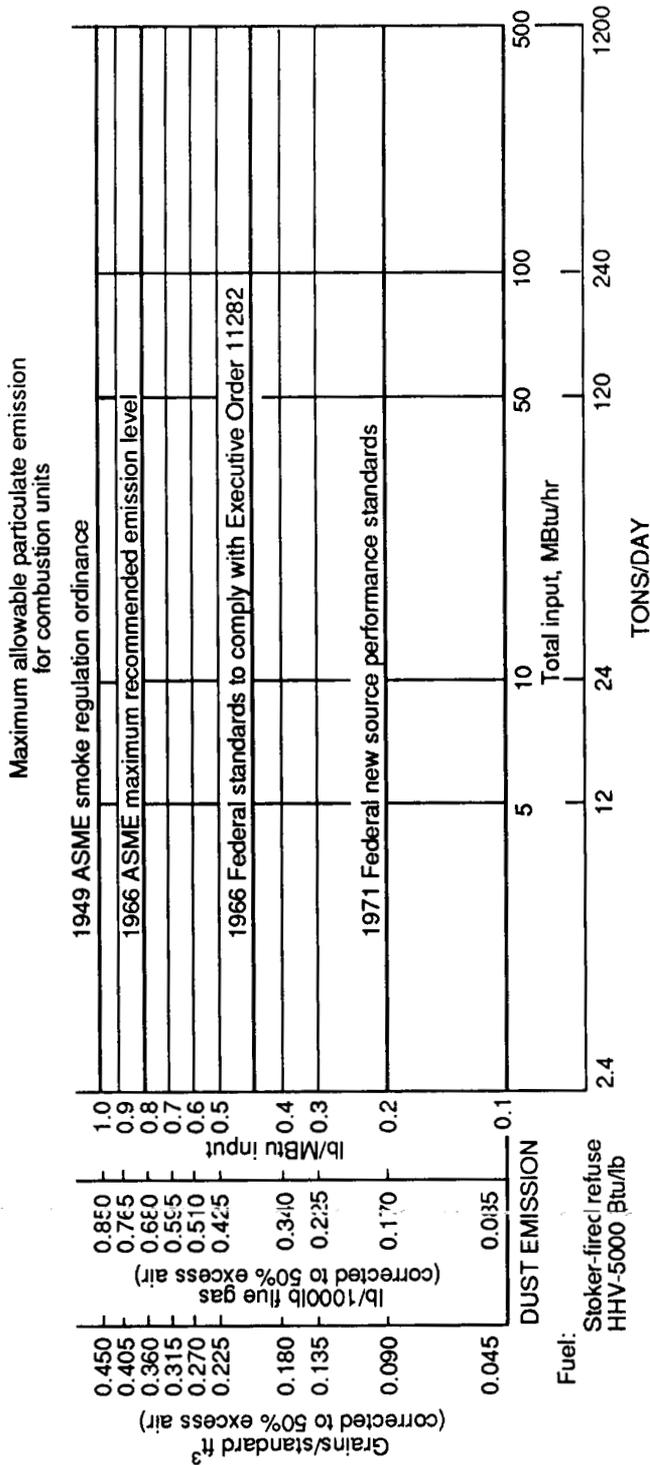
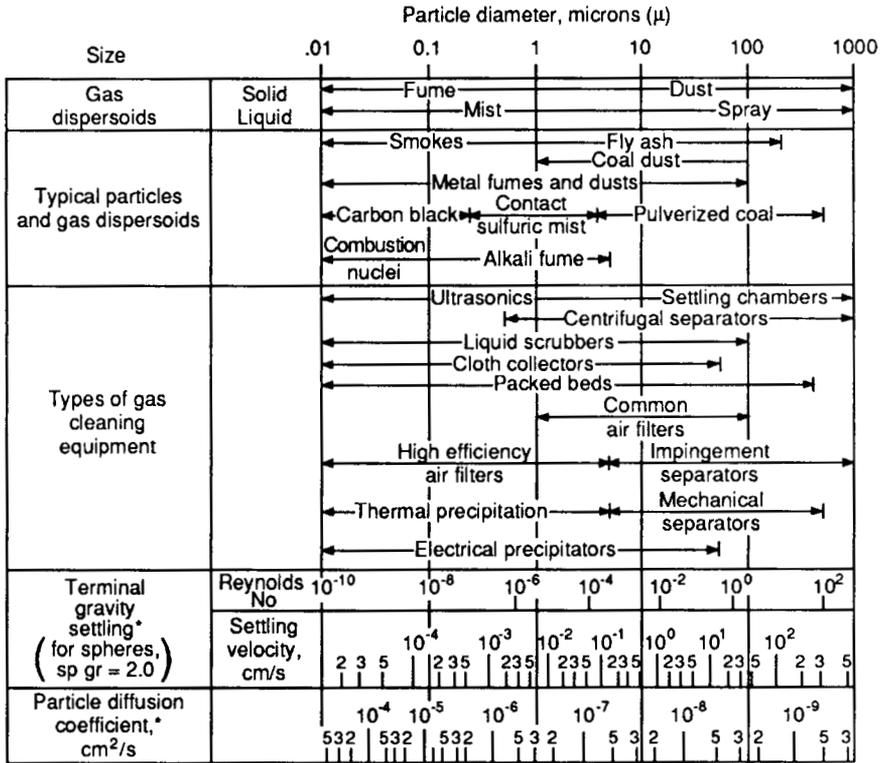


FIGURE 8.47 Emission control standards (85).



* In air at 25°C, 1 atm; Stokes-Cunningham factor included

FIGURE 8.48 Particle classification chart ((70).

flues is estimated as 10% of the dry flue gas, or 85 lbs (38.6 kg) of dry air. With this air, there will be 1.12 lb (0.5 kg) of moisture ($85 \times 0.0132 \text{ lb H}_2\text{O}/\text{lb air}$). Some minor losses will also occur in sluicing the fly ash, estimated at 1% of the heat in the dry gas.

The computation in Table 8.72 shows that there will be total heat energy of 358,929 Btu that must be absorbed by moisture in the flue gas leaving the furnace and total moisture = $358,929 / (1335 - 48) = 278.9 \text{ lb}$. Since the quantity of moisture in the flue gas was 73.1 lb, the theoretical spray water required would be $278.9 - 73.1 = 205.8 \text{ lb}$, or $205.8 / 8.34 = 24.7 \text{ gal}$. In practice, the actual quantity of spray water required will depend on the manner of introducing the water into the gas stream (fine or coarse sprays, evaporation from a wetted surface, etc.). An evaporation efficiency of 50% or less is conservative, or a requirement of at least 50 gal per 100 lb of MSW for this particular analysis.

Control Devices. Over the past 20 to 30 years, a number of different approaches have been used to control particulate emissions from incinerators. Settling chambers were probably the earliest means used to abate the pollution impact of gross particulates from incinerators. These chambers, which depended on gravity settling of fly ash were essentially ineffective in removing particles smaller than 10 μm in size. Overall collection efficiencies were on the order of 10%. Cyclones, which depend on centrifugal force to remove the particulate matter, were also used in some of the earlier plants. While these devices were much

TABLE 8.72 Heat Balance for Spray Chamber (in Btu)

Input at 1680°F	
Heat of dry gas: 847×424	= 359,128
Heat in water vapor: $73.1 (1900 - 48)$	= 135,381
Heat in fly ash carryover (assume 2.0 lb/100 lb MSW) and specified heat of 0.25): $2.0 \times 0.25 \times (1680 - 80)$	= 800
Heat unaccounted for	= 400
Total	495,709
Output at 600°F	
Heat in dry gas: $(847 + 85) \times 128$	= 119,296
Heat in air leakage moisture: $1.12 \times (1335 - 48)$	= 1,441
Heat loss through walls @ 3% input	= 14,850
Minor losses from sluicing	= 1,193
Heat in vapor from furnace and spray water	= 358,929
Total	495,709

Note: 1055 Btu = 1 J; °C = (5/9) (°F - 32)

more efficient than settling chambers, reaching efficiencies of 50 to 70%, they are not capable of meeting current emission standards. Small-diameter [less than 9 in (23 cm)] cyclones arranged in banks of multiple cyclones can achieve higher efficiencies of removal. Experience has shown that there are serious operational problems related to plugging of these small-diameter cyclones due to characteristics of the particulate and moisture in the flue gases. These problems indicate that this equipment is not suitable for application to this field.

Wet scrubbers, particularly venturi scrubbers and tray-type scrubbers, have also been used to control particulate emissions from incinerators. Although medium- to high-energy scrubbers [pressure drop of 7 in (18 cm) of water and higher, three-tray scrubbers or venturi scrubbers] should theoretically be able to meet standards, experience to date indicates this is not the case. As a matter of fact, it may be seen in Table 8.73 that there is relatively little improvement in emission control with significantly increasing energy inputs using wet collection equipment.

Several other disadvantages, in addition to the problem of low removal efficiencies noted above, that are inherent in application of wet scrubbers have essentially eliminated this equipment from consideration for application to this field. Thus, these units require the use of large quantities of water resulting in problems of water cleanup and significantly increasing the probability of corrosion problems (87). Further, stack exhausts from such plants either exhibit a white steam plume or the exhaust gases must be reheated, at additional expense, so that the plume separates from the stack.

TABLE 8.73 Range in Test Results Using Wet Collection (86)

Type of facility	Range in pressure drop, in wet collection	Corrected emissions, g/ft ³ @ 12% CO ₂
Water sprays	less than ¼	0.19–0.72
Wetted baffle walls	¼–4	0.10–0.9
Tray scrubbers	3–11	0.025–0.6
Venturi scrubbers	8–20	0.01–0.61

Electrostatic precipitators have been a widely used particulate collection device in incinerator plants because they have given consistently successful results. All of the more than 50 units that have been installed at plants on this continent since the early 1970s have successfully passed their acceptance tests, while many have achieved emission test results well below the Federal New Source Performance Standards. They are also very effective in collecting submicron-size particles.

A basic electrostatic precipitator consists of a negatively charged discharge electrode that places a charge on the particulate matter in the gas stream, and a series of collecting electrodes, generally grounded plates, that provide the surface on which the particulate matter collects and to which it adheres. The collection efficiency is dependent on a number of factors, including the strength of the electric field, the gas temperature [usually around 500°F (260°C) for this service], the moisture content of the gas, the resistivity of the dust itself, the effectiveness of cleaning the electrodes and collection plates, and proper gas flow distribution (88). Achievement of acceptable collection efficiencies requires that the particulate matter have a resistivity of 1×10^5 to 2×10^{10} ohm-cm. Other factors that impact collection efficiency are the collecting surface area provided, the velocity of gases in the precipitator [generally 2 to 4 ft/s (0.6 to 1.2 m/s)], and the retention time in the precipitator (generally from 5 to 10 seconds to meet current enforcement code levels).

Precipitators are generally cleaned by vibrating or rapping the collecting plates. One key to maximum precipitator collection efficiency is proper rapping or cleaning of the plates. If the rapping is too violent or is not sequenced properly, the collected dust will be disturbed, reentrained, and carried out of the precipitator rather than sliding down the plate into the collection hopper below.

Fabric filters are currently the preferred method of particulate control in incinerator plants because they are known to be high-efficiency particulate collection devices. They are particularly effective in controlling the emission of small submicron-size particles. The filtering process, occurring as the gases pass through bags, similar to a household vacuum cleaner, is affected by interception, impingement, and agglomeration.

The choice of woven fabric material for use in a baghouse is based on the required efficiency of particulate removal, the pressure drop across the unit, the allowable gas throughput, and the gas temperature (89). Normally, the efficiency of removal and the pressure drop across the bags are closely related because they are both a function of the tightness of the weave or permeability of the filter. The usual values of permeability for bag materials for this service range from 10 to 70 ft³/min/ft² (3 to 21 m³/min/m²). Some filter fabrics that may have application in this service, with some of their characteristics, are listed in Table 8.74. Newer fabrics reportedly have been developed that can withstand temperatures in excess of 750°F (400°C), which is higher than the temperature one would normally expect in the gases exiting the boiler in an energy-from-waste plant.

With the increasing concern about trace chlorinated hydrocarbon, heavy metal, and acid gas emissions from incineration plants, most modern plants utilize scrubbers and baghouses or dry scrubbers and electrostatic precipitators to meet permit requirements for particulate and acid gas control. In this type of gas cleanup system, pioneered in Europe, a lime slurry is introduced into the gas stream prior to the particulate collection device (baghouse or electrostatic precipitator). The lime dosage is set at a multiple of the stoichiometric ratio required for complete reaction with the acid component of the effluent gases. After evaporating the slurry to dryness, the gas-lime dust mixture is passed through the particulate collection device.

Data on performance testing of such units indicate that such systems, if properly controlled and operated, effectively remove over 90% of HCl, up to 85% of SO₂, over 99% of particulates and most of the heavy metals because of chemical reactions in the gas stream and the generally higher collection efficiencies required to remove the lime reagent added to the gas stream. The additional capital and operating costs of such systems are substantial.

Operational Experience. Literally hundreds of waste-to-energy plants have been constructed around the world since the development of the current basic technologies in the early to mid-1960s. In the process, a number of operational problems have surfaced, most of which have been solved. While there is room for further refinement and development of the mass-fired technologies, primarily to improve efficiencies and in-

TABLE 8.74 Properties of Fiber Materials (89)

Fiber	Physical characteristics				Relative resistance to attack by			
	Relative strength	Specific gravity	Normal moisture content, %	Maximum usable temperature, °F (°C)	Acid	Base	Organic solvent	Other attribute
Polyester (Dacron)	Strong	1.4	0.4	280 (138)	Good	Medium	Good*	—
Acrylonitrile (Orlon)	Medium	1.2	1	250 (121)	Good	Medium	Good	—
Polyethylene	Strong	1.0	0	250 (121)	Medium	Medium	Medium	—
Tetrafluoroethylene	Medium	2.3	0	500 (260)	Good	Good	Good	Expensive
Polyvinyl acetate	Strong	1.3	5	250 (121)	Medium	Good	Poor	—
Glass	Strong	2.5	0	550 (288)	Medium	Medium	Good	Poor resistance to abrasion
Graphitized fiber	Weak	2.0	10	500 (260)	Medium	Good	Good	Expensive
Asbestos	Weak	3.0	1	500 (260)	Medium*	Medium	Good	—
“Nomex” nylon	Strong	1.4	5	450 (232)	Good	Medium	Good	Poor resistance to moisture

*Except phenol.

†Except heated acetone.

‡Except SO₂.

TABLE 8.75 Status of U.S. Waste-to-Energy Facilities in 1984 (69)

Status	Number of facilities	Rated daily capacity, tons/day
Operational	52	23,450
Under construction	14	9,300
Closed	20	12,350

Note: 1 ton = 0.9 t.

crease plant availability and reliability, this technology is proven and available for application. RDF processing and combustion technologies are not as far along in development but, if conservatively designed, this type of plant also has a place for application in this field. Other technologies that have been tried, in this country and overseas, are not at a stage of development such that they are ready for commercial application at this time (90).

A summary of the experience in this country is presented in the following tables. Table 8.75 summarizes the status of energy-from-waste projects in the United States as of the end of 1984.

Table 8.76 presents a summary of operational MSW processing and resource recovery facility capacities in plant sizes larger than laboratory or bench-scale in 1984 and in 1991. Several additional comments are in order to further qualify the information summarized in this table. Of the 29 operating modular plants in 1984, 11 were of 100 ton/day (91-t/day) capacity and larger while the other 18 averaged 40-ton/day (36.4-t/day) capacity. Thus, while there were a great number of plants in this category, they were of relatively small size. In the RDF technology, many of the 10 operating plants in 1984 were in startup for extended periods (years) prior to achieving operational status. In many instances, the plants were substantially modified, and in several instances they were derated. Most, if not all, of the eight RDF plants that were shut down by 1984 were closed without achieving full-capacity operation. This underscores the fact that utilization of this technology should be done very carefully.

As the solid waste disposal crisis deepens and this nation's energy needs become more critical, in the future, the available energy in MSW will be tapped more frequently. The technology is available now for successful application of these techniques if provision is made for adequate funding to purchase high-quality equipment suitable for the intended service, and to hire properly trained operating staffs.

TABLE 8.76 Types of Technology at U.S. Waste-to-Energy Facilities

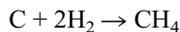
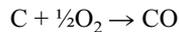
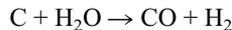
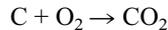
Technology	Operational 1984		Operational 1991	
	Rated capacity, tons/day	Percent	Rated capacity, tons/day	Percent
Mass burn	11,000	46.9	69,3300	75.1
Modular systems	3,125	13.3	2,700	2.9
RDF	9,325	39.8	20,245	22.0
Pyrolysis	0	0	0	0
Total	23,450	100	92,275	100

Pyrolysis and Gasification Processes

Incineration is the dominant method for the thermal destruction of municipal solid waste (MSW) and is treated separately in this handbook. This section deals with other processes that use high temperatures to alter the physical and chemical character of MSW. The thermal processes discussed provide for the conversion of MSW to a variety of alternate fuels that can be substituted for fossil fuels in a wide variety of combustion systems.

Two processes, pyrolysis and gasification, are presented. Pyrolysis is an old process. Prebiblical Egyptians prepared embalming fluid by pyrolysis. Pyrolysis of wood to produce chemicals (acetic acid, acetone, methanol) was widely practiced through World War II. Pyrolysis is the process of heating an organic material in the absence of oxygen. No other reacting material is introduced into the reactor system. Large organic molecules, as a result of the high temperature, break into smaller and simpler molecules.

Gasification processes introduce reacting gases into the reactor to encourage the formation of gaseous products and/or release heat within the reactor. Some of the common gases introduced include oxygen, steam, or hydrogen that cause reactions such as



Gasification reactions usually take place at higher temperatures than required for pyrolysis. For complete gasification, all of the solid is converted to a gas.

In many discussions, the terms pyrolysis and gasification are used interchangeably. There is no universally accepted definitions for these terms in modern literature, which has led to confusion. For this discussion, whenever a gas is added to the reactor to promote the gas yield, the system is classified as a gasification process.

The discussion is limited to the organic portion of MSW, referred to as refuse-derived waste (RDF). RDF consists largely of cellulose based materials (primarily paper) and behaves similarly to wood. The large body of knowledge developed for wood is helpful in the prediction of the behavior of RDF.

Cellulose represents a polymeric material as shown in Figure 8.49. This is a polymer material, and when

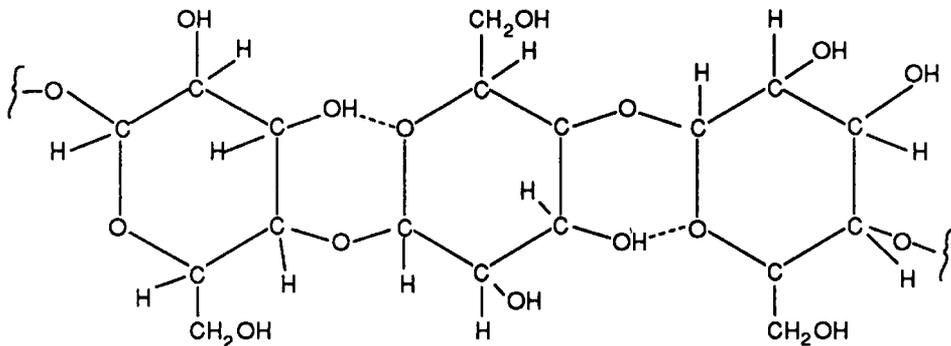


FIGURE 8.49 The cellulose molecule.

subject to thermal stress (elevated temperature), it breaks down. The pyrolysis process may be selective or nonselective. In the selective process, a given chemical bond is broken; this may provide a high yield of a single product. The pyrolysis of RDF is a highly nonselective process. The feed itself is heterogeneous in size, chemical composition, and structure, and there is little chance to select a particular bond to be broken. Raising the temperature of pyrolysis has a major effect on the destruction of the RDF.

Figure 8.50 shows general reaction scheme for cellulose. When cellulose is heated very slowly, the polymer is slowly broken down, with the weaker bonds being broken first. The products are largely noncombustible gases, a large fraction of nonreactive char, and a wide variety of medium- to high-molecular-weight hydrocarbons. Typical yields from destructive distillation, wherein the wood is heated for many hours at a moderate temperature are

Gas	14–17 wt %
Methanol	1.5–2.5 wt %
Acetic acid	3.5–8 wt %
Tar	12–16 wt %
Char	31–37 wt %

Most of the heating value is retained in the char, which may be stored and used as a fuel.

When the cellulose is heated rapidly to a high temperature, the cellulose molecule is shattered into small molecules having a small tar, liquid, and char yield. Most of the newer thermal systems for treatment of MSW to produce a fuel product use high temperatures to provide high gas yields. In these applications, since the original structure of the feed material is almost completely destroyed by the high temperature (thermal hammer), the products are largely independent on the physical nature of the feed. The effects of temperature on product yield are shown in Figure 8.51.

The primary difference between RDF and wood when pyrolyzed to a high temperature is that RDF has a

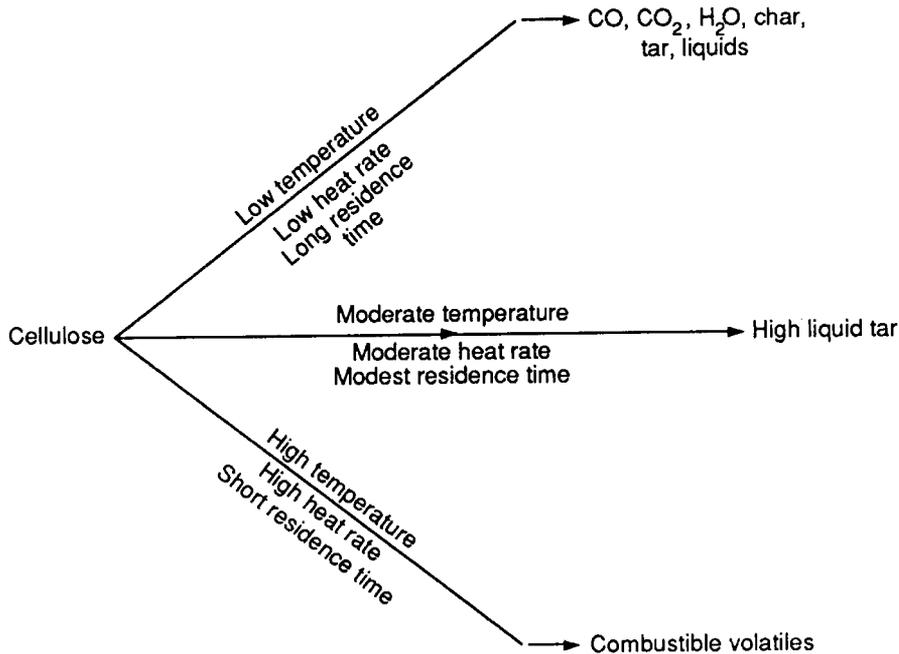


FIGURE 8.50 Breakdown of cellulose with heat.

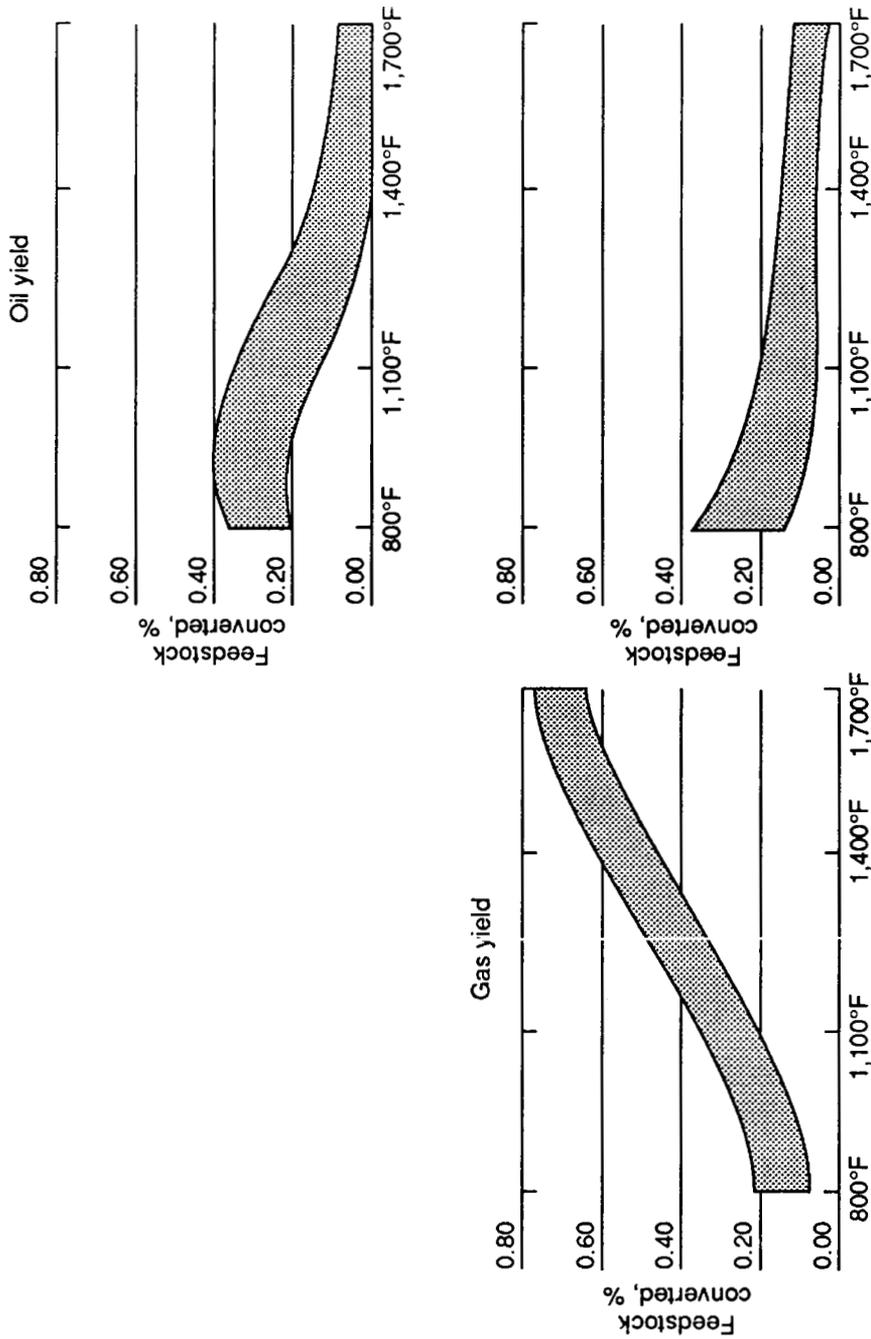


FIGURE 8.51 Yield of gas, char, and tar at various temperatures.

8.170

lower char yield. The plastic materials in RDF provide little char and a gas with a higher heating value. The impurities found in RDF apparently have a catalytic effect that leads to lower char fraction.

The products from pyrolysis include a tar, liquid, and char fraction along with gas. In gasification systems, a reactive gas is introduced to react with these products and increase gaseous products. To achieve high gas conversion rates, high temperatures are required in gasification systems. The reactive gases most often used are oxygen, steam, and sometimes hydrogen. Even when there are no reactive gases added to the system, the pyrolysis gases produced will react with char and tar to produce additional gas if the temperature is high.

Energy/Fuel Recovery. Figure 8.52 shows how energy is recovered (chemical, sensible, or heat) from various thermal treatment systems. The values shown in Figure 8.52 would be similar to those obtained from the organic portion of MSW when processed in a thermal process at 1500°F (815°C). In order to maintain the process at 1500°F, heat (represented by q) may be added to or removed from the process.

At point 1 in Figure 8.52, no air (oxygen) or other reactive gas is introduced. To obtain 1500°F, temperature heat (q_1) is required. This represents pyrolysis. Most of the energy leaves the system as chemical energy associated with the pyrolysis products (represented by ΔH_i) and as sensible heat of the pyrolysis products

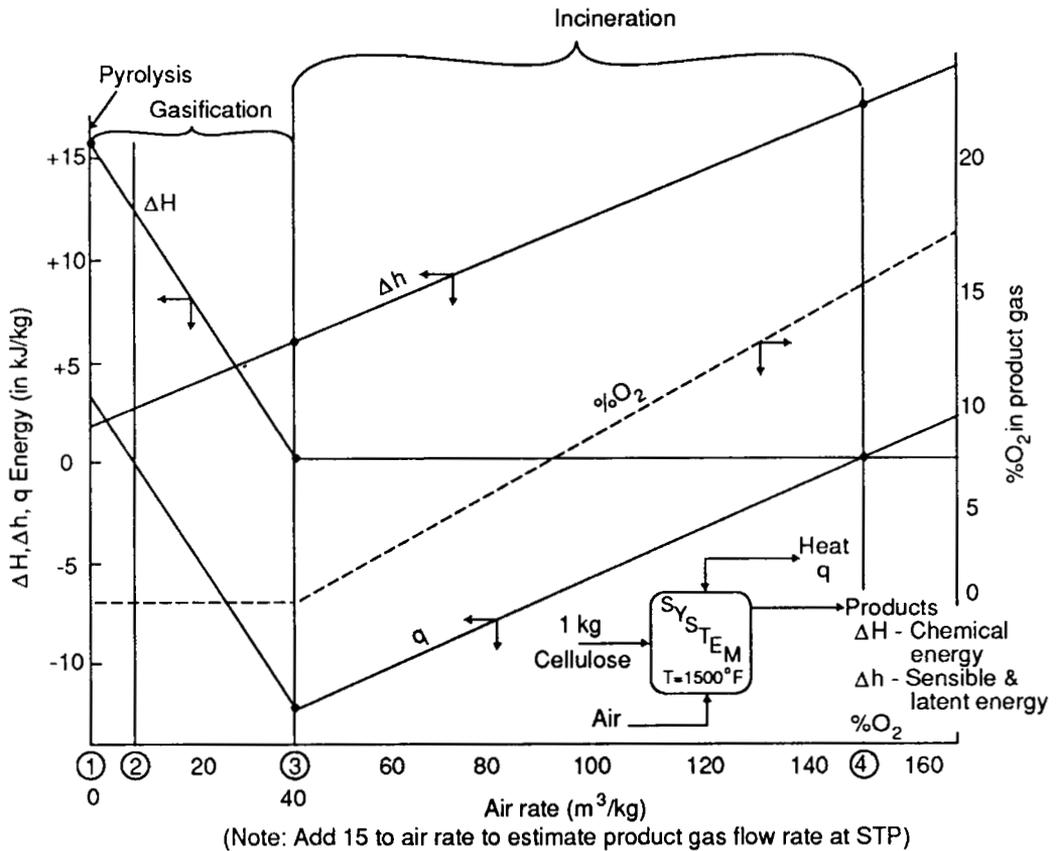


FIGURE 8.52 Generalized energy relationships for thermal treatment of cellulose.

(represented by ΔH_1). Most of the chemical energy available in the MSW has been converted to chemical energy in the pyrolysis products.

To the right of point 1 in Figure 8.52, air has been introduced into the system. The oxygen reacts and releases heat. This reduces the chemical energy available in the products and reduces the amount of heat added. At point 2, sufficient heat is released within the reactor to eliminate the need to add heat. This condition is identified as gasification, since a reactive gas (oxygen) was introduced. It is often referred to as partial combustion, starved air combustion, or low-Btu pyrolysis. Much of the energy available in the MSW is removed as chemical energy, ΔH_2 of the products formed. A major portion of the energy available in the MSW continues to leave as chemical energy. A larger portion of the energy is carried away as sensible heat. If pure oxygen replaces air, the fraction of energy carried away as sensible heat is much less. This is the result of reducing the nitrogen heated to 1500°F (815°C).

To the right of point 2, additional air (oxygen) is introduced, resulting in more combustion and more heat released in the reactor. Provisions must be made to remove this energy from the reactor. At point 3, sufficient air (oxygen) is introduced to combust all of the MSW. There is no chemical energy in the reactor products. If the heat removed, q_3 , is used to generate steam, this represents the maximum amount of steam that can be generated. This is a theoretical condition of zero percent excess air. Practical systems require a significant amount of excess air to obtain complete combustion.

To the right of point 3, excess air is introduced, passes through the reactor unreacted, and carries out energy as sensible heat, reducing the heat to be removed from the reactor. At point 4, no heat is added to or removed from the process (adiabatic system). All of the energy leaves the process as sensible heat and may be recovered in a waste heat recovery system.

The regions to the left of point 3 serve only as suggestive values because the products produced are not known and depend upon many factors, including the type of reactor selected for the process as well as physical characteristics of the feed.

The major difference between operations to the right of point 3 and those to the left is that those to the left remove a portion of the energy as chemical energy (fuels). Those fuels may be removed from the process and transported to a user where it may be burned in the user's combustion equipment. It is this potential to recover useful fuel that may replace an existing fossil fuel that makes alternate thermal treatment systems other than incineration attractive.

The region to the right of point 3 is termed combustion, which is a special case of gasification where all carbon is oxidized to CO_2 and all hydrogen to H_2O . As the products are known, precise energy and material balances can be made if the composition and heating value of the fuel are known. Water-walled incinerators fall in region 3–4, and most other incinerators fall at point 4.

The volume of gas leaving the reactor system is also shown in Figure 8.52 for the case where the major portion of the products are gaseous. This is important when it is necessary to clean the gases to protect the environment. Cost of gas cleanup is strongly dependent upon the volume of gas and concentration of pollutant. The ratios of gas volumes for pyrolysis, adiabatic gasification, stoichiometric combustion, and adiabatic combustion are 1/2/4/11.

In comparing combustion systems to other thermal destruction processes (OTP), combustion results in high gas volumes while OTP results in energy recovered as chemical energy and lowered gas volumes.

Reactor Systems. Systems used for thermal destruction of MSW may be classified according to the mechanisms of solids flow and the manner in which heat is provided.

1. *Gravity:* Solids flow under the force of gravity. Shaft furnaces (moving beds) belong to the class.
2. *Mechanical:* Solids flow as a result of mechanical force. Multihearth furnaces, rotary kilns, and auger kilns belong to this class.
3. *Drag:* Solids flow as a result of drag force resulting from gas flow past the solid. Fluidized beds and entrained beds belong to this class.
4. Combinations of above.

The published literature on pyrolysis and gasification of MSW and biomass reveals that Europe stresses shaft kilns (gravity flow), the Japanese emphasize fluidized-bed systems (drag flow), and none of the groups dominate in the United States.

Heat may be added or removed from the reactor by three mechanisms:

1. *Indirect*: Heat is added to the vessel through the vessel walls. Combustion occurs external to the reactor.
2. *Direct*: Heat is released within the reacting vessel by gasification reactions (usually combustion).
3. *Carrier*: Energy is carried into reacting vessel as sensible heat of a nonreacting material. Carrier system material may be a solid, gas, or liquid.

Representative examples for each type of reactor system will be discussed below. The systems identified do not provide a comprehensive list but include most of the major systems that have reached commercial or semicommercial status.

Gravity Flow Systems. The shaft furnace has been the dominant reactor system considered for the gasification of MSW. Figure 8.53 shows its important features. Air or oxygen is introduced through the bottom and the solid through the top where it descends toward the bottom through several zones. At the top the solid is dried. The dry solid passes into a higher-temperature zone where pyrolysis occurs (no oxygen zone). Leaving the pyrolysis zone is a solid char that passes into a gasification region where some of the char reacts with H_2O (or CO_2) to produce H_2 and CO . The remaining char moves down into a zone containing oxygen, where combustion occurs and oxygen is consumed.

The shaft furnace is almost always refractory-lined and is a simple reactor system. The major differences between systems is found in the manner that the solid residue is removed and the auxiliary units, such as those used to preheat the entering gas, clean the product gas, and utilize the product gas. Only a small amount of pretreatment of MSW, such as the removal of oversized objects, is required.

The Andco-Torrax system is an application of the shaft kiln. In this system, the air to the gasifier is preheated to provide a sufficiently high temperature in the bottom of the kiln to slag the inorganic material present. The gas leaving is not cleaned but fired directly in a close-coupled combustion, steam generation, and heat-recovery unit. The gas composition from a typical application is shown in Table 8.77. There are several commercial units operating overseas.

Union Carbide developed the Purox system in a 200-ton/day (180-t/day) pilot plant in South Charleston, West Virginia. Instead of using air, this unit uses oxygen. The product gas is treated to remove liquids and tars that are recycled to the high-temperature region of the furnace. The clean product gas composition is shown in Table 8.77. It differs from the Anco-Torrax gas, which contains over 50% N_2 .

Figure 8.54 shows the major features of a vertical retort. The solid is fed to the top, and char is removed from the bottom. It is a pyrolysis system, and the heat is supplied through the walls. A portion of the gas product is used as fuel to heat the walls. The heat transfer rate is slow, and the solids residence time is several hours. When these systems were used for the thermal destruction of wood at low temperatures, a typical product showed:

Gas	14–17%
Methanol	1.5–2%
Acetic acid	3.5–8%
Tar	12–16%
Char	31–37%

The Destrugas system represents a European-developed vertical retort operated in the pyrolysis mode. It is based upon coke oven techniques. The furnace is externally heated to temperatures of 1650 to 1830°F (900 to 1000°C). Residence time of solid in the reactor is about 20 h. Typical values obtained from pilot units showed:

Gas	36 wt.%
Residue	28 wt.%

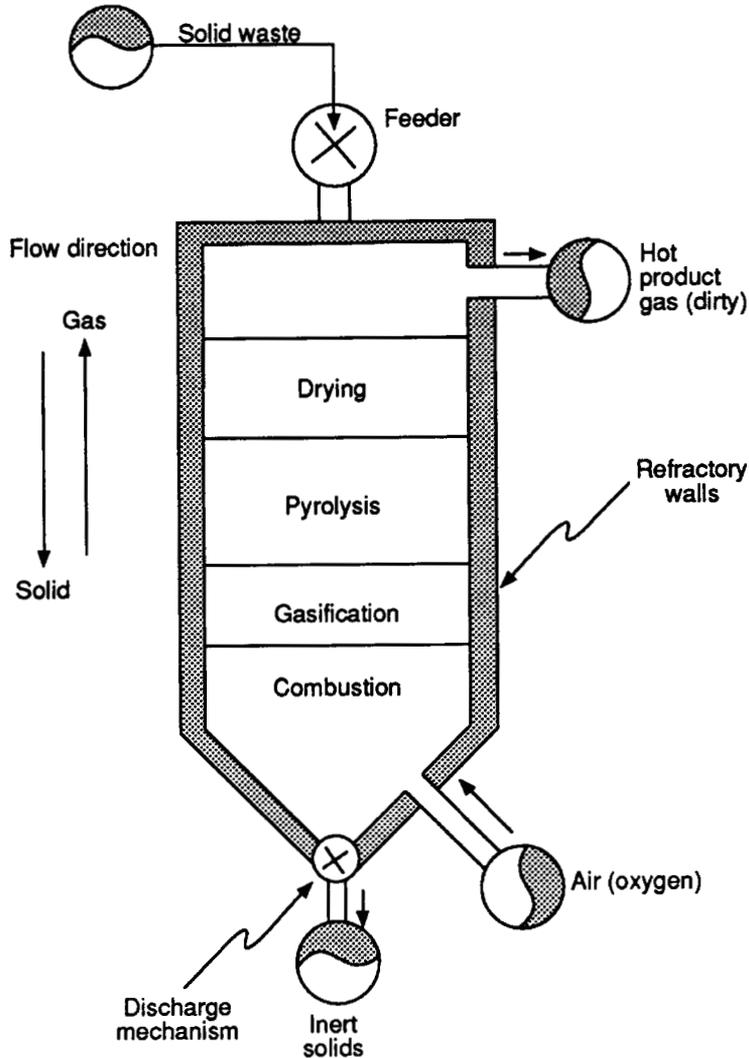


FIGURE 8.53 Features of shaft kiln.

Tar	4wt.%
Water	32 wt.%

Pilot and demonstration plants have been run at sizes up to 18 tons/day (16 t/day). These systems are not practical at a large size because of the volume to surface ratio limits, the area of heat transfer through the walls, and the increase in thickness of material to be heated.

Mechanical Flow Systems. The most used reactor in this class is the rotary kiln or retort. The rotary kiln is a cylindrical vessel set at an angle of a few degrees from horizontal. This vessel is rotated slowly. The solid material enters the elevated end and is carried up the walls for some distance before it tumbles back to-

TABLE 8.77 Gas Compositions and Characteristics for Representative Systems, Percent by Volume

Component	Ando-Torraz	Purox	Landgard	Babcock	High-temp. fluid bed	Ebara
CO	10.3	47	6.6	8-15	8.9	35
H ₂	11.2	33	6.4	15-25	13.8	23
CH ₄	1.9	4	2.6	3-8	2.4	15
C ₂ H ₄	0.8	1	1.8		2.7	7
O ₂	3.0	—	1.6			
CO ₂	10.5	14	11.4	15-12	17.6	19
N ₂	62.3	1	69.7	45-65	55.2	1
Process						
Pyrolysis				X		
Gasification	X	X	X	X	X	
Gravity flow	X	X				
Mechanical flow			X	X		
Drag flow					X	X
Heating						
Indirect				X		
Direct	X	X	X			
Carrier						X

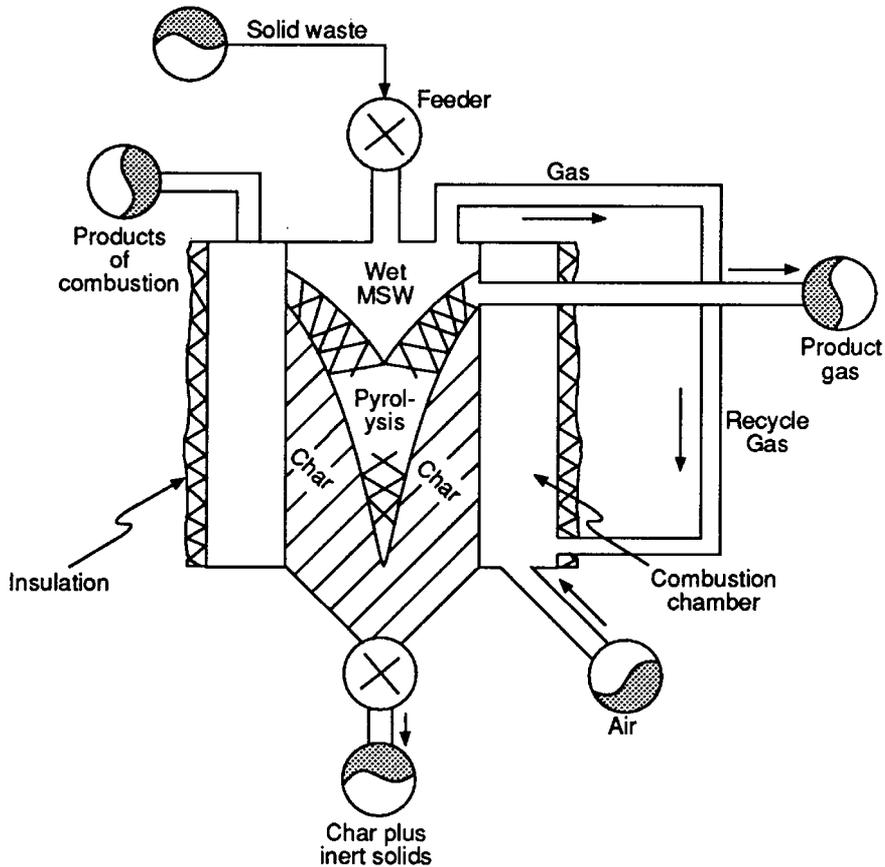


FIGURE 8.54 Features of a vertical, continuous-feed retort.

ward the bottom of the kiln (as well as toward the lower end of the inclined vessel where it is discharged). Flights (see Figure 8.55) may be added to the vessel walls to help carry the solid close to the top of the kiln before it falls away from the wall.

The rotating kiln serves to agitate the solids and move the solid through the system. Only a small fraction of the total volume is occupied by the solid; most of the volume is occupied by the gas. The gas-solid contact is poor. The solid tumbles through the gas stream to the wall where it remains with little contact with the gas until it is ready to drop back through the gas stream. The rotary kiln will accept a wide variety of materials. Not all shapes and materials move through the system with the same speed. Some materials “roll” through the kiln rapidly and some light materials are caught in the gas and move through the system rapidly.

The greatest problems associated with any mechanical system result from the protection of rotating parts at high temperatures and the seals between the rotating members and rest of the process.

The Monsanto Landgard System is a directly heated rotary kiln gasifier. In this application, fuel oil is burned with a portion of the air to the kiln to provide the temperatures required for the thermal destruction of the MSW. The reported gas compositions are given in Table 8.77.

A plant to treat 900 tons/day (810 t/d) of MSW was built in Baltimore, Maryland, but never performed as

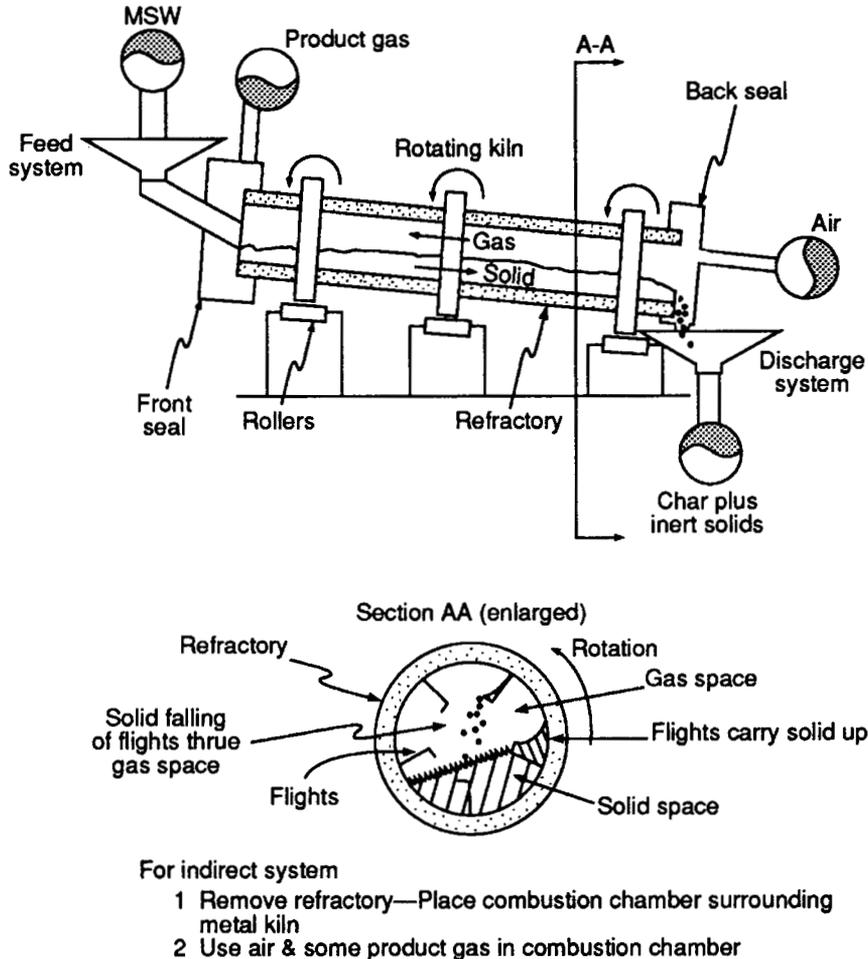


FIGURE 8.55 Features of a rotary kiln gasifier.

predicted from data obtained in a (30-ton/day) (27-t/d) pilot plant unit. Monsanto has withdrawn from the project, and the plant is not being operated.

The Pyreval process of Babcock-Krauss-Jueffer represents an indirectly heated pyrolysis process. The external heating chamber is divided into several zones. Low pyrolysis temperatures between 750 to 1025°F (400 to 550°C) are used. This results in high liquid tar yields. These are sent to an external thermal cracking unit held at 1830°F (1000°C) where they are broken down into low-molecular-weight gases and char. The solid residence time is between 30 and 60 min. About 30 weight percent of the product is gas and 30 weight percent char. The gas composition is shown in Table 8.77. A 2 to 3 ton/day (1.8 to 2.7-t/day) pilot plant is being built.

Drag Flow Systems. The solids move through the system as the result of drag forces resulting from the flow of a fluid. This includes molten salt, fluidized bed, and entrained-bed systems. In the fluidized-bed systems (shown in Figure 8.56), the solid particles are inert and are suspended by the upward flow of fluid.

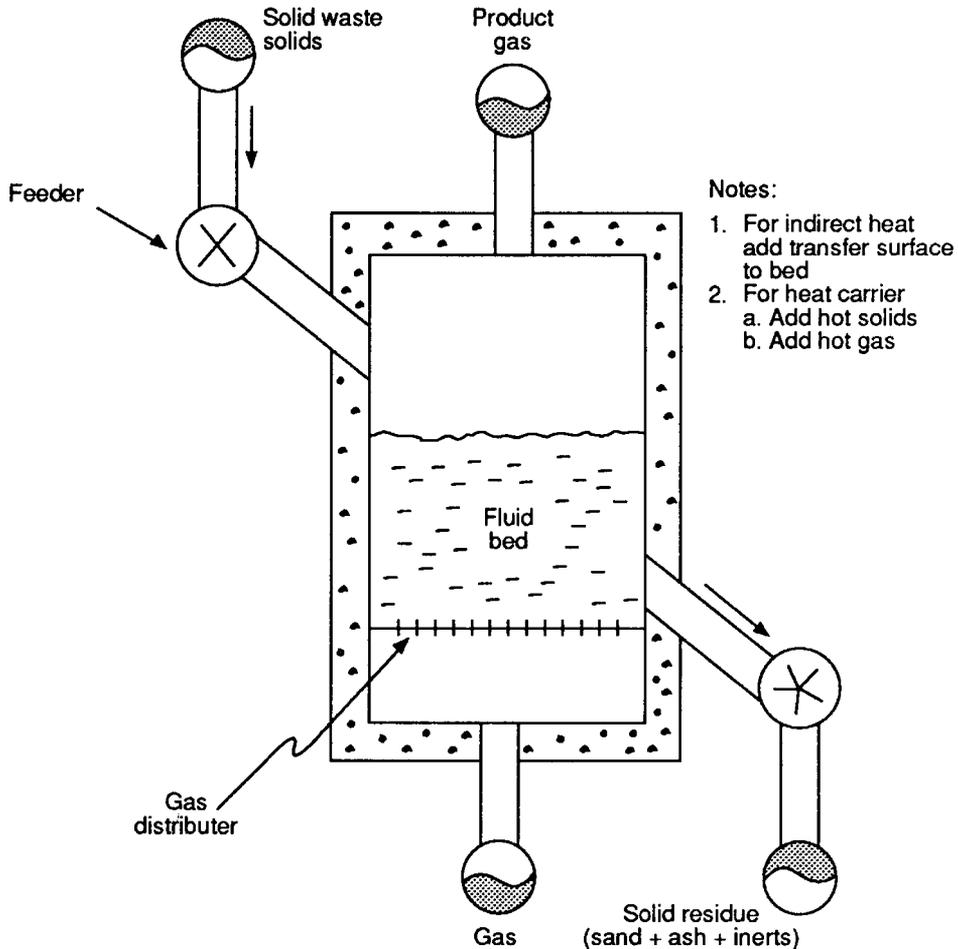


FIGURE 8.56 Features of a fluid-bed system.

Each particle is surrounded by a gas. There is an upper interface between the top of the fluid bed (solid-gas mixture) and the gas. The volume of the bed is only about 30% greater than would be occupied by the solid alone.

The particles move about rapidly. This rapid mixing results in isothermal operations. Heat transfer to any solid added to the bed is rapid.

In the fluid bed, the solid particles are not carried out of the reactor system by the gas stream. In the entrained bed, however, the gas velocity is large enough to carry the solids out of the reactor along with the gas. The entrained-bed reactor system may best be described as a pneumatic solids transfer system where chemical reactions occur. In this system, the solids are not mixed and the temperature is not constant throughout the reactor.

The Hitachi process is a low-temperature fluid-bed gasifier. Air is pressed upward to provide fluidization. The fluid bed is made up of inert sand operating at a low temperature [840 to 1025°F (450 to 550°C)]. The

bed is fed shredded MSW. This low temperature produces a high liquid and char fraction. A typical yield shows

Gas	30 wt. %
Water	37 wt. %
Char	13 wt. %
Oil	17 wt. %
Ash	3wt.%

The high liquid fraction comes mostly from decomposition of the plastic in the waste (12%) and less from decomposition of the cellulose (5%). It is noted that Japanese MSW has a much larger plastic fraction than does United States MSW. While the Hitachi process has been classified as gasification because reactive gas (air) is introduced, the low temperatures reached result in almost no reaction of char with these gases, and the product yield is more closely related to the pyrolysis yield.

If the temperatures of the fluid bed are raised to 1500°F (815°C), the char and tar fraction are reduced. The gas composition for this high-temperature system is shown in Table 8.77.

Ebara has developed a pyrolysis system using a fluid bed where heat is carried into the bed by a high-temperature inert solid. The fluid bed is composed of sand, which is continuously drawn from the bed, while hot sand is continuously added at the same rate. This hot sand rapidly loses all sensible heat above the bed temperature to the bed. The system is fluidized by recycling pyrolysis gas. The sand drawn off goes to a sand heater that is a second fluid bed where the char and liquids separated from the product gas stream are burned. The gas product does not contain nitrogen, since no air is introduced into the reactor. The product gas composition is shown in Table 8.77. This system has been demonstrated in Yokohama, Japan, at 30-ton/day (27-t/day) capacity. A similar plant has recently started up in Funabachi City, Japan, using three 150-ton/day (135-t/day) trains.

Occidental Research developed an entrained-bed pyrolysis system. Recycle gas was used to entrain hot char at about 1380°F (750°C). Finely divided RDF is introduced at the bottom. The temperature drops to 750°F (400°C) at the top. By controlling the gas velocity and the reactor height, the solid residence time is controlled (a few seconds). Controlling the temperature to modest levels and providing short residence times leads to high liquid yields. Typical values are

Gas	27 wt. %
Water	13 wt. %
Oil/tar	40 wt. %
Char	20 wt. %

The liquid product (“garboil”) was tested as a substitute for Bunker C fuel oil. It had several rather obnoxious qualities that suggest it is a poor substitute. It was corrosive, requiring special storage facilities and fuel nozzles, etc. It was more difficult to pump and smelled bad. These qualities largely resulted from the wide range of highly oxygenated organics (including acids).

A 100-ton/day (90-t/day) demonstration plant was built in California but never ran successfully and was shut down.

There are many advantages of the fluid-bed and entrained-bed systems. They have a high throughput. In the case of the fluid bed, it is an isothermal system, and the temperature is easy to control. It also produces high gas yields. There are also some major disadvantages. The most important is the requirement that MSW be pretreated to reduce the size to less than 1½ in (3.8 cm) and materials that cause slagging in the bed, such as glass and aluminum, must be controlled to a low value. It requires at least a poor-quality RDF as feed. In the case of the entrained bed, the feed must be ground to the consistency of flour or fluff. Significant energy may be expended in terms of blowers to provide for suspension and movement of particles in these systems.

The emphasis in pyrolysis and gasification has moved in the direction of producing a gaseous fuel for

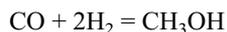
several reasons. There are over 15,000 boilers in the United States firing natural gas or oil with a capacity between 50,000 and 250,000 lb/hr [150 to 750 ton/day (135 to 675 t/day) RDF]. In many cases, these could be used to fire low-Btu gas from a close-coupled gasifier. The gasifier would be located adjacent to the boiler and the necessary breeching provided to move the hot gas to the boiler. Burners and controls would require replacement. Any char and liquid formed should burn along with the gas in the existing combustion chamber. Overall thermal efficiencies of 85% or more would be expected.

If it is necessary to cool and clean the gas prior to combustion in order to protect the environment, the boiler, or because there is not sufficient time to burn tar and char, the efficiency drops to about 55%. The sensible heat in the pyrolysis gas as well as the energy of the char, tar, and liquids are lost.

Figure 8.57 shows the effect of boiler efficiency gas and air volumes for fuel gas of varying energy values. It shows the efficiency to drop rapidly for heating values less than 200 Btu/ft³ (7450 KJ/m³) indicating that a boiler firing a low-Btu gas would have to be derated. This also shows that the amount of flue gas produced for a given boiler rating rises rapidly for heating values less than 400 Btu/ft³ (14,900 KJ/m³). For boilers designed with critical gas velocities (more likely in American design than European), the low-Btu gas would again require derating.

For cases of pyrolysis gas, a medium gas is produced, and no derating would be required.

The nitrogen-free pyrolysis gas may be used as a process synthesis gas for the production of organic chemicals. The city of Seattle, Washington, performed an extensive feasibility study to consider producing either methanol or ammonia from MSW. While the study came to a positive conclusion, the program stalled before detailed design was carried out. The preliminary plans called for the Purox process to produce a medium-Btu gas. To produce methanol, this synthesis gas is cleaned; reformed (to convert hydrocarbons to H₂ and CO); shifted (to provide an H₂/CO ratio of 2/1 needed for methanol production); and scrubbed to remove CO₂. This gas then goes to a catalytic reactor where methanol is produced according to the reaction



Methanol can be used in an internal combustion engine either as an additive to gasoline where it acts as an octane enhancer or as straight methanol (which requires engine modification).

Advantages over Direct Incineration. There are numerous advantages to using pyrolysis and gasification systems for treatment of MSW as a replacement for incineration. Some of these are listed below. In this list, a system that produces a fuel gas by either pyrolysis or gasification followed by a combustion system for this fuel gas is compared to direct incineration.

1. There is less gas to clean in order to protect the environment. The volume of air used to incinerate waste is 5 to 10 times that used to pyrolyze the same waste. The cost of gas cleaning is much less.
2. There is less gas to clean for environmental protection even in the case of close-coupled systems where the fuel gas products are burned prior to being cleaned. Large excess air quantities are required to burn the waste directly, whereas the gaseous fuel products require far less excess air.
3. Pyrolysis systems allow for the energy user to be located some distance from the waste processing plant. For cases where char and liquid are the products, they may be stored until needed. User demands need not match the incinerator output.
4. Investment and operating costs are higher for incinerator. In most cases, the pyrolysis and gasification systems are much simpler than the mass burn incinerators where heat is effectively recovered. Table 8.78 points out factors relative to these costs.
5. Pyrolysis and gasification systems can handle a variety of wastes, such as rubbers and plastics, that cause problems in most incinerators as a result of high temperatures resulting from these high-heating-value feeds.
6. There is a much wider range of uses for the energy products from pyrolysis and gasification than from in-

8.180

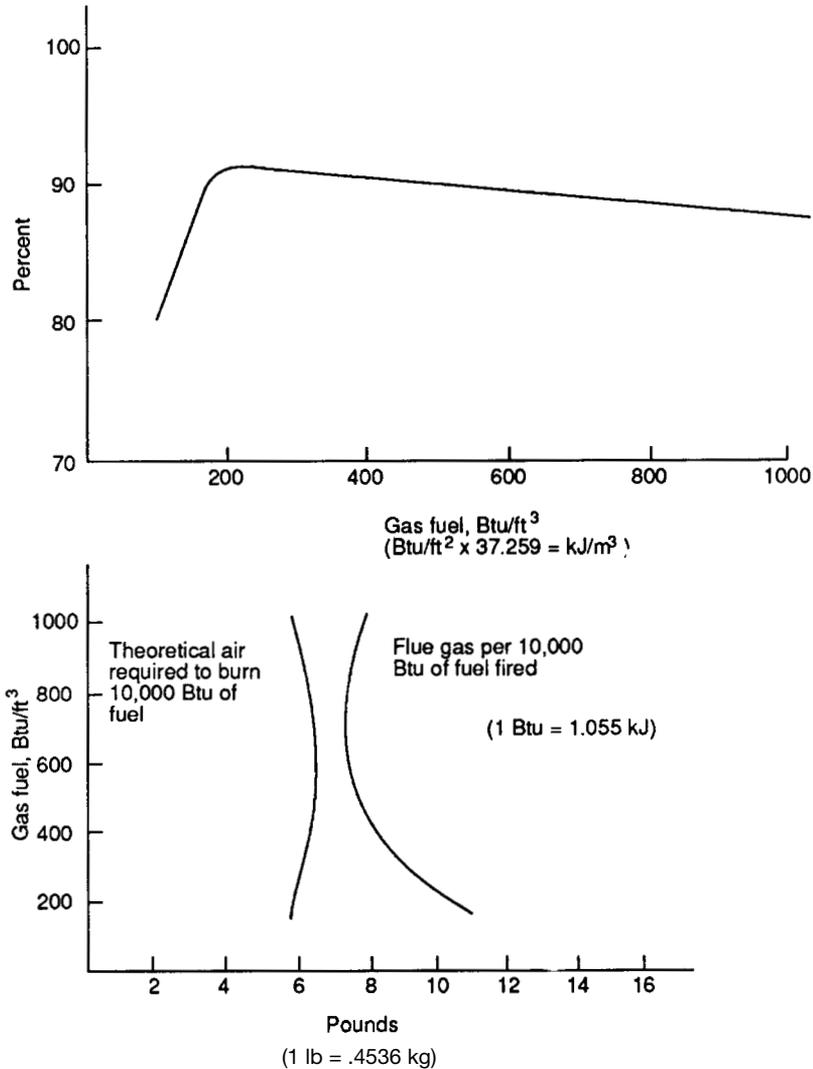


FIGURE 8.57 Combustion efficiency and theoretical air and flue gas volume as a function of fuel gas heating volume.

cineration (steam). The fuel products can be used in boilers, engines, turbines, etc., as well as a chemical feedstock.

7. Low-cost packaged boilers or a wide variety of existing equipment may be used to burn the fuel gas.

Pyrolysis is an old technology that is not widely used today. While there are apparent advantages over incineration, it is seldom recognized as an alternative to incineration. Although incineration has had major problems in the United States, it still remains conditionally acceptable. Unfortunately, there is not sufficient experience in operating commercial pyrolysis and gasification systems to establish the validity of many of the advantages claimed.

TABLE 8.78 Comparison of Pyrolysis and Combustion Furnaces

Aspects	Pyrolysis	Combustion
Atmosphere	Reducing	Oxidizing, sometimes alternately reducing and oxidizing
Temperature	Mostly lower	Especially local high temperatures are possible (on grate)
Construction	In most cases only few moving parts	Mostly mechanically moved grates
Corrosion during heat recovery	Small in case of pure pyrolysis (gas cleaning before energy recovery)	Deposition of fly ash; corrosion of the tubes of steam boiler (gas cleaning after energy recovery)

Wet Oxidation

The wet oxidation process is a type of incineration that uses oxidation in the presence of water in a closed reactor at moderately high temperatures and at a wide range of pressures. Since solid wastes are generally generated and collected in relatively dry form, the wet oxidation process is not currently common to solid waste processing but is used for the treatment of wastewater sludges and concentrated wastewater streams.

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CHAPTER 9

HAZARDOUS WASTE

Robert A. Corbitt, P.E.*

Hazardous wastes are present throughout the world as by-products of growth and development and include pesticides from agricultural lands, gasoline leakage from service stations, heavy metals from plating solutions, medical wastes from hospitals, and radioactive wastes from nuclear power plants. These waste products pose unreasonable risks to human health and safety, property values and utilization, and all other components of the environment. The U.S. Environmental Protection Agency regulations define hazardous wastes by inclusion of specific lists and by characteristics of ignitability, corrosivity, reactivity, and/or EP toxicity. Household wastes, domestic sewage, and certain other wastes are excluded from these regulations.

The technology for treatment and disposal of hazardous wastes is the most rapidly developing area of environmental engineering. A significant portion of this technology is refinement and/or adaptation of proven practices in air quality control, wastewater treatment, and solid waste management. Also, the environmental engineer must learn more about hydrogeology to assess the subsurface disposition of hazardous wastes.

Hazardous waste treatment is characterized as liquid waste treatment, solid waste treatment, solidification and stabilization, thermal destruction, and land disposal. Remedial action is distinguished as surface water control, air pollution control, or in situ treatment.

At a hazardous waste treatment or disposal site, the environmental engineer must obstruct the routes of the waste into the environment. Principal routes include surface water contamination from runoff or overflows; groundwater contamination from leaks or leachate; air contamination from open burning, evaporation, or fugitive dusts; fire and/or explosion; and health risks from human contact.

In the United States, hazardous wastes regulations are very stringent. They include requirements for a manifest system for tracking wastes from generation through storage, transport, treatment, and disposal. In addition to treatment and disposal, the environmental engineer helps management address means to satisfy these regulatory controls as well as achieving pollution prevention by reducing waste and through process changes, including recovery and reuse.

HAZARDOUS WASTE: SOURCES AND EFFECTS

Hazardous waste is used as a broad term to denote industrial by-products and waste materials discarded from homes, commercial establishments, and institutions that pose an unreasonable risk to human health and safety, property, and the environment. Most notable is the waste's potential cause of or contribution to increased mortality or an increase in serious irreversible or incapacitating, reversible illness.

This section deals with the liquid, gaseous, and solid hazardous wastes included in the Resource Conser-

*This chapter was prepared in association with Paul T. Dickman and Terrence J. McManus, P.E.

vation and Recovery Act of 1976 (RCRA), as amended. Radioactive wastes are controlled by the Atomic Energy Act of 1954, as amended, and are discussed in the last section of this chapter.

Pursuant to RCRA, the U.S. Environmental Protection Agency (EPA) published regulations defining hazardous wastes by inclusion in specific lists and by characteristics of ignitability, corrosivity, reactivity, and/or EP toxicity (1). Certain wastes are not subject to RCRA control; these include (1, 2):

- Domestic sewage
- Point-source industrial wastewater discharges
- Irrigation return flows
- Nuclear and radioactive wastes
- Materials subjected to in situ mining techniques
- Reclaimed and reused secondary materials from production processes
- Household wastes
- Utility industry wastes
- Oil- and gas-drilling fluids

Sources of Hazardous Wastes

The major source of hazardous waste is the generator. Other sources are transporters, treaters, and disposers of wastes deemed hazardous by the generator. A manifest system is required to track the hazardous waste from its generation to its final disposal.

Generation. Since hazardous wastes are the result of satisfying society's needs and conveniences, the primary waste generator is industry. However, relatively minor volumes of hazardous wastes may come from a very large number of commercial or institutional sources. Commercial sources include dry cleaners, gasoline and service stations, and printers. Institutional sources include hospitals, laboratories, and military installations.

During 1980, the EPA estimated that U.S. industry generated some 52 tons (57 million metric tons) (wet) of hazardous wastes, with about 31 tons (34 million metric tons) coming from the chemicals and allied products industry. Table 9.1 illustrates the distribution among six major industrial generators. Furthermore, 10 states were identified as contributing 60% of the waste; in descending order by volume, these states are New Jersey, Illinois, Ohio, California, Pennsylvania, Texas, New York, Michigan, Tennessee, and Indiana (3).

The regulations issued under RCRA (4, 5) require that a generator of hazardous waste

- Determine if a waste is hazardous
- Obtain an identification number

TABLE 9.1 Relative Distribution of Hazardous Waste Generation by U.S. Industry (3)

Industrial generator	Percent
Chemical and allied products	60
Machinery (except electrical)	10
Primary metals	8
Paper and allied products	6
Fabricated metal products	4
Stone, clay, and glass products	3
All others	9

- Obtain a facility permit, if accumulating wastes on site for more than 90 days
- Use appropriate containers and labeling for shipment
- Prepare a manifest system for tracking the waste
- Submit an annual activities summary

If a generator stores, treats, or disposes of wastes on site, additional requirements apply.

Transportation. A transporter or carrier operates equipment to load hazardous waste at the point of generation and must ensure the safe and proper delivery of the waste to a permitted storage, treatment, and/or disposal site.

The RCRA regulations for transportation of hazardous wastes require the transporter to (6, 7)

- Obtain an identification number
- Comply with the manifest for tracking the waste
- Retain a copy of the manifest for three years
- Deliver the entire quantity of waste to the selected designation
- Comply with applicable U.S. Department of Transportation regulations
- Clean up any hazardous wastes discharged during transportation

For transporters, a discharge is defined as the accidental or intentional spilling, leaking, or in any way emptying of hazardous waste onto the land or into surface waters. Shipments may be made via road, rail, or water. The greatest potential for accidental discharge lies in the transfer of wastes between containment facilities.

Treatment and Disposal. The treaters and disposers of hazardous waste complete the chain of responsibility under the manifest system. They must comply with RCRA regulations on facility standards and apply for a permit (8–10). The focus of this chapter is the technology available to owners and operators of these facilities to comply with RCRA requirements.

The source of hazardous waste at a treatment and/or disposal facility is the individual waste(s) being handled. The routes for environmental damage at these sites are (3)

- Groundwater contamination via leaks or leachate
- Surface water contamination via runoff or overflows
- Air contamination via open burning, evaporation, and wind erosion
- Fire and explosion
- Poisoning via the food chain
- Human contact

Effects of Hazardous Wastes

The effects of hazardous wastes vary considerably with respect to human health and safety, property, and the environment. Sources of wastes are so numerous that site and waste characteristics are needed to define the timing and severity of local impacts.

Health. Hazardous wastes may cause health effects by entering the body through inhalation, skin absorption, ingestion, or puncture wounds. Temporary effects include dizziness, headaches, and nausea; permanent effects include cancer, disability, and death. These effects may be evident immediately, or they may not be apparent for months or even years. The impact on health is also dependent on the amount and duration of hazardous waste exposure.

Safety. Knowledge of proper safety attire and procedures is essential for personnel involved in cleanups and day-to-day hazardous waste handling activities. A site safety plan is needed to establish policies and procedures to protect workers and the public from potential hazards. Table 9.2 includes minimum contents of a site safety plan; the plan should be updated as needed to reflect new conditions at either a cleanup site or handling facility.

Property. Damage to private and public property and the physical environment is most likely to result from fires and explosions at hazardous waste sites. In addition to creating hazards through intense heat and smoke, open flames, and flying debris, fires and explosions may emit hazardous substances to the atmosphere. Most often, fires and explosions result from investigative or remedial activities, such as accidentally mixing incompatible contents of drums, or from introduction of an ignition source, such as sparks from equipment.

Environment. The potential adverse effects on the air, water, and land environment are as varied as the numbers of hazardous wastes. In addition to environmental consequences of improper handling of hazardous waste, there is almost always an associated risk to human health.

Guidelines for hazardous waste handling facility performance with respect to human health and the air, water, and land environment are presented in Table 9.3. These guidelines were developed from specific standards published by the EPA for new landfills, surface impoundments, land treatment facilities, and underground injection wells.

The risks of environmental damage from hazardous wastes obviously can be lessened by the reduction or elimination of waste generation. Waste reduction is a critical and promising technological challenge to develop practices of product substitution, process modification, source segregation, and recycle, recovery, and reuse. In some cases, government action is used to stop the production of particularly hazardous substances.

Waste Minimization

Waste minimization (pollution prevention) practices are not only intended to protect and improve human health and the environment but also are means to reduce costs, build community relations, demonstrate environmental leadership, and reduce the impacts of state and federal requirements. Waste minimization consists of source reduction and recycling, which consist of a number of practices and approaches, as illustrated in Figure 9.1. Source reduction usually is preferable from an environmental protection perspective (119–120).

TABLE 9.2 Site Safety Plans (11)

-
- Name key personnel and alternates responsible for site safety.
 - Describe the risks associated with each operation conducted.
 - Confirm that personnel are adequately trained to perform their job responsibilities and to handle the specific hazardous situations they may encounter.
 - Describe the protective clothing and equipment to be worn by personnel during various site operations.
 - Describe any site-specific medical surveillance requirements.
 - Describe the program for periodic air monitoring, personnel monitoring, and environmental sampling, if needed.
 - Describe the actions to be taken to mitigate existing hazards (e.g., containment of contaminated materials) to make the work environment less hazardous.
 - Define site control measures and include a site map.
 - Establish decontamination procedures for personnel and equipment.
 - Set forth the site's standard operating procedures for those activities that can be standardized, and where a checklist can be used.
 - Set forth a contingency plan for safe and effective response to emergencies.
-

TABLE 9.3 Environmental Performance Guidelines (12)

Prevention of adverse effects on air quality considering

1. The volume and physical and chemical characteristics of the waste in the facility, including its potential for volatilization and wind dispersal
2. The existing quality of the air, including other sources of contamination and their cumulative impact on the air
3. The potential for health risks caused by human exposure to waste constituents
4. The potential damage to wildlife, crops, vegetation, and physical structures caused by exposure to waste constituents
5. The persistence and permanence of the potential adverse effects

Prevention of adverse effects on surface water quality considering

1. The volume and physical and chemical characteristics of the waste in the facility
2. The hydrogeological characteristics of the facility and surrounding land, including the topography of the area around the facility
3. The quantity, quality, and directions of groundwater flow
4. The patterns of rainfall in the region
5. The proximity of the facility to surface waters
6. The uses of nearby surface waters and any water quality standards established for those surface waters
7. The existing quality of surface water, including other sources of contamination and their cumulative impact on surface water
8. The potential for health risks caused by human exposure to waste constituents
9. The potential damage to wildlife, crops, vegetation, and physical structures caused by exposure to waste constituents
10. The persistence and permanence of the potential adverse effects

Prevention of adverse effects on groundwater quality considering

1. The volume and physical and chemical characteristics of the waste in the facility, including its potential for migration through soil or through synthetic liner materials
2. The geologic characteristics of the facility and surrounding land
3. The patterns of land use in the region
4. The potential for migration of waste constituents into subsurface physical structures
5. The potential for migration of waste constituents into the root zone of food-chain crops and other vegetation
6. The potential for health risks caused by human exposure to waste constituents
7. The potential damage to wildlife, crops, vegetation, and physical structures caused by exposure to waste constituents
8. The persistence and permanence of the potential adverse effects

The EPA's Waste Minimization National Plan outlines a long-term national effort to reduce the quantity and toxicity of hazardous wastes. The plan proposes mechanisms and implementation methods for affecting waste minimization. Key components of the plan include establishing goals, setting priorities for waste source reduction and recycling efforts, identifying and evaluating source reduction and recycling opportunities, arraying mechanisms for affecting source reduction and recycling, implementing mechanisms, and measuring progress (121–123).

The goals of the Waste Minimization National Plan are to:

- Reduce by 50% the most persistent, bioaccumulative, and toxic (PBT) chemicals in the nation's hazardous waste by the year 2005, as compared with the baseline year of 1991.
- Emphasize source reduction (reducing waste at its source, before it is even generated) and environmentally sound recycling, over waste treatment and disposal.
- Prevent transfers of chemical releases from one medium (air, water, or land) to another.

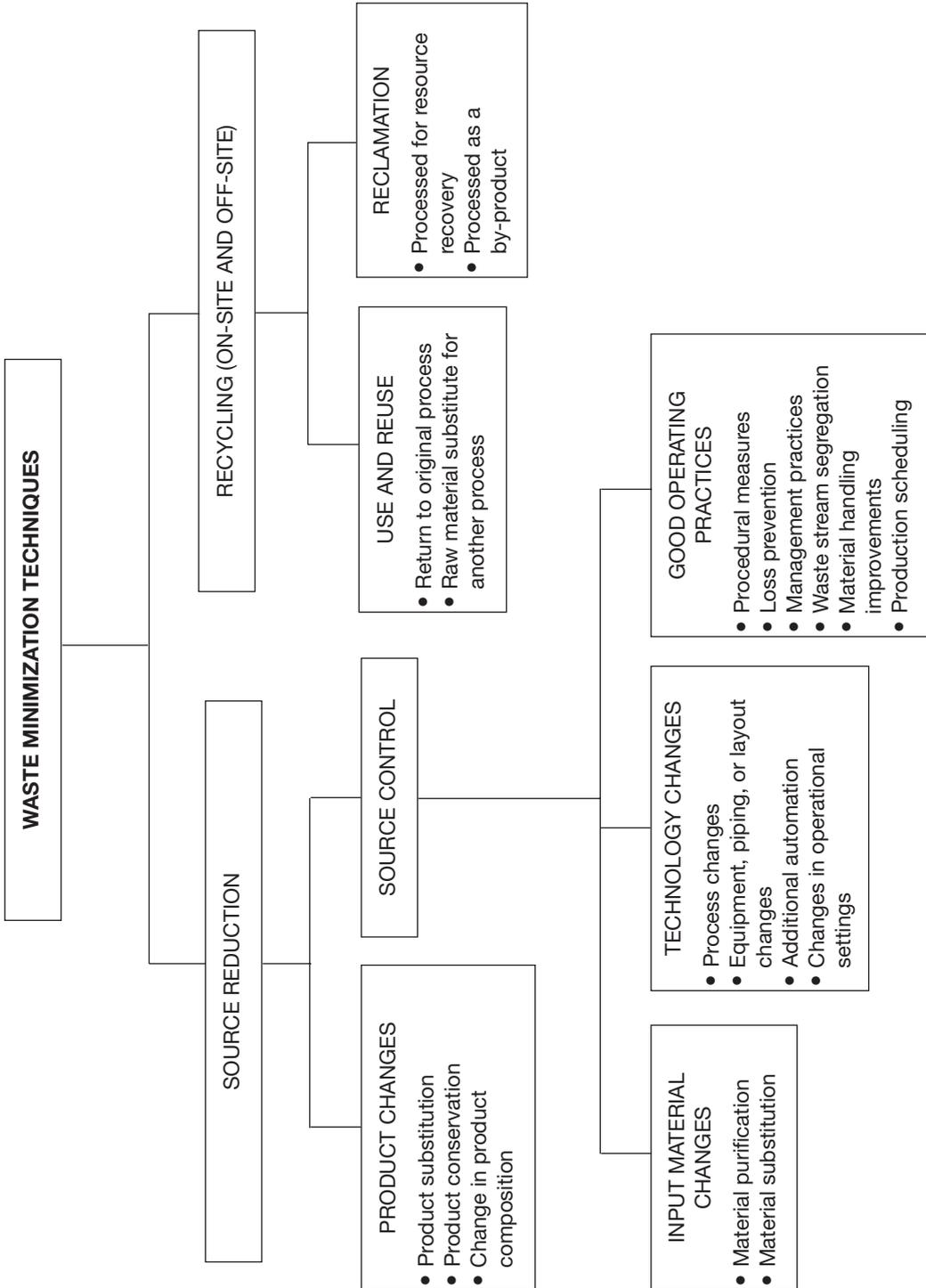


FIGURE 9.1 Waste minimization techniques (119).

In particular, the reduction in the presence of PBT chemicals in waste will reduce long-term threats to human health and the environment because:

- Persistent (P) chemicals generally do not break down in the environment.
- Bioaccumulative (B) chemicals tend to concentrate in animal and plant tissue.
- Highly Toxic (T) chemicals can cause cancer or other health effects in humans, and greatly endanger the environment.

The waste minimization assessment procedure, illustrated in Figure 9.2, is a means for waste generators to support the success of the national plan. First, generators establish waste minimization goals at their facilities and then develop and carry out a program to achieve or exceed the goals. Table 9.4 (page 9.9) identifies some of the facility information that may be needed or acquired. Often, the assessment phase begins with the identification of PBT chemicals on site and determination of the processes producing PBT waste. Finally, source reduction and recycling alternatives are evaluated, selected, and implemented to eliminate or reduce PBT waste.

CHARACTERIZATION

Hazardous wastes can be solid, liquid, or gaseous. The characterization of a hazardous wastes can range from a simple review of a list published by the EPA to a detailed chemical analysis. Reasons for characterizing hazardous wastes are to

1. Classify the waste (i.e., hazardous or nonhazardous)
2. Define the nature of the waste to determine the best method for handling, treatment, storage, and/or disposal
3. Determine the nature of the waste to assess compatibility with other wastes
4. Assess potential environmental and/or public health impacts of the waste
5. Delist a hazardous waste

Characteristics

A simplified logic diagram showing the steps necessary to determine whether a waste is hazardous is presented in Figure 9.3 (page 9.10). The key to applying the diagram is knowing the source and nature of the waste material. The EPA has listed specific hazardous wastes according to the following criteria (*I*):

1. *F Series*: Hazardous waste from nonspecific sources (e.g., halogenated solvents used in degreasing and spent electroplating bath solutions)
2. *K Series*: Hazardous waste from specific sources (e.g., API separator sludge and tank bottoms from leaded petroleum refining)
3. *P Series*: Acutely hazardous waste of specific commercial chemical products, including discarded products, off-specification products, containers, and spill residuals
4. *U Series*: Toxic hazardous wastes that are commercial chemical products, including discarded products, off-specification products, containers, and spill residuals

The initial list of hazardous wastes was published in the Federal Register on May 19, 1980 (*I*). This list is periodically amended, and the most recent list should be used for the determination of hazardous waste.

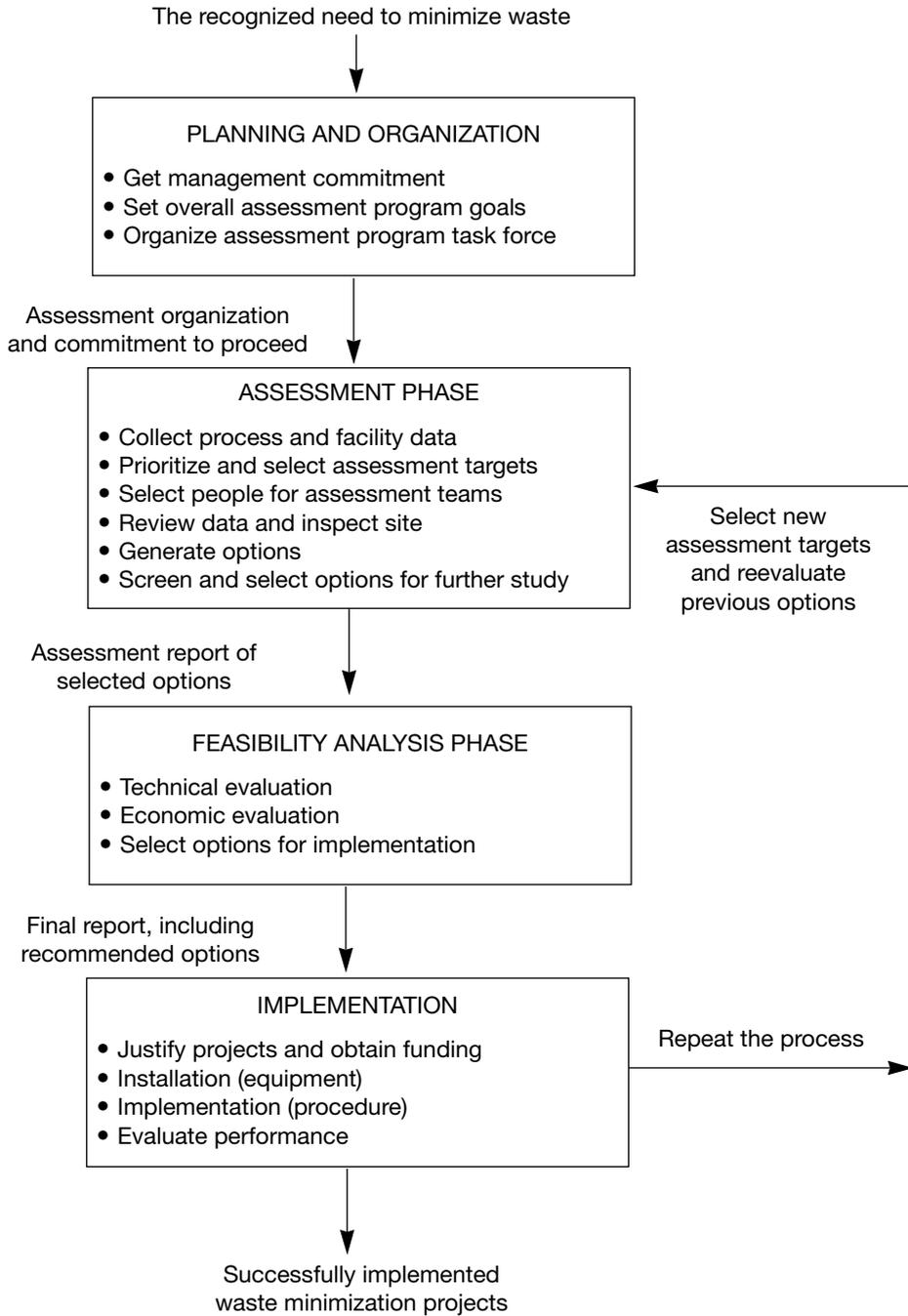


FIGURE 9.2 Waste minimization assessment procedures (119).

TABLE 9.4 Facility Information for Waste Minimization Assessments (119)*Design Information*

- Process flow diagrams
- Material and heat balances (both design balances and actual balances) for production processes
- Pollution control processes
- Operating manuals and process descriptions
- Equipment lists
- Equipment specifications and data sheets
- Piping and instrument diagrams
- Plot and elevation plans
- Equipment layouts and work flow diagrams

Environmental Information

- Hazardous waste manifests
- Emission inventories
- Biennial hazardous waste reports
- Waste analyses
- Environmental audit reports
- Permits and/or permit applications

Raw Material/Production Information

- Product composition and batch sheets
- Material application diagrams
- Material safety data sheets
- Product and raw material inventory records
- Operator data logs
- Operating procedures
- Production schedules

Economic Information

- Waste treatment and disposal costs
- Product, utility, and raw material costs
- Operating and maintenance costs
- Departmental cost accounting reports

Other Information

- Company environmental policy statements
- Standard procedures
- Organization, charts

Also, lists prepared by the applicable state regulatory agency should be reviewed for materials are not on the federal list.

If the source and nature of the waste material are known, however, and the source material is not a listed hazardous waste, then a determination must be made as to whether the material exhibits hazardous waste characteristics. Table 9.5 presents hazardous waste characteristics and the criteria used for evaluation. It should be noted that many state environmental regulatory agencies have additional criteria for characterizing a waste as hazardous beyond the federal list presented in Table 9.5. The appropriate agency should be contacted to determine what additional testing may be necessary.

The complexity of characterizing a waste to determine whether it is hazardous increases significantly when the source and/or nature is unknown. This is a common occurrence at an abandoned waste site. The only methods available are to sample and perform chemical analysis to determine the nature of the material.

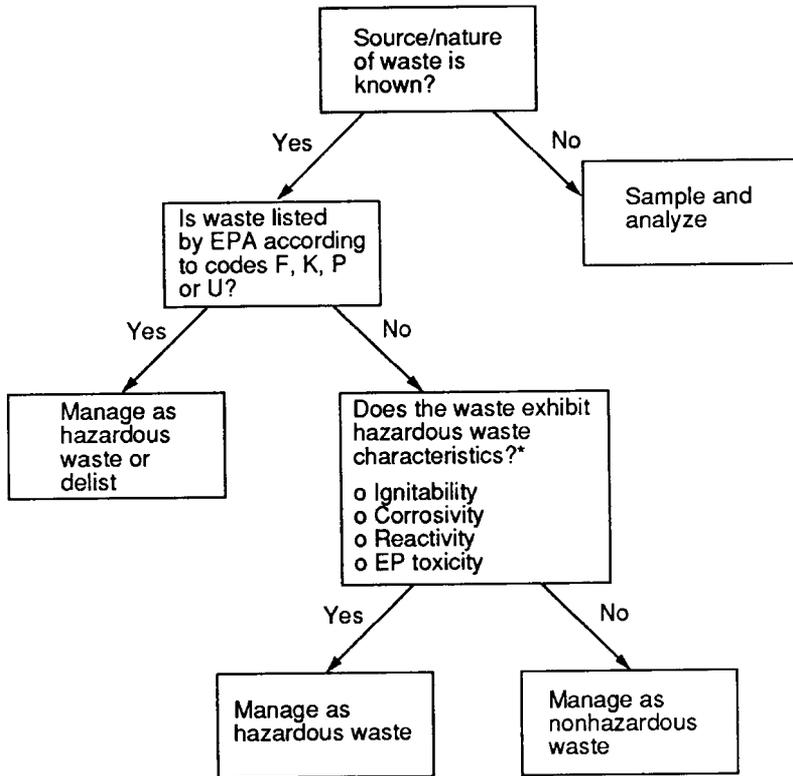


FIGURE 9.3 Simplified hazardous waste characterization logic diagram.

Properties

Once a material has been deemed a hazardous waste, the properties of this waste must be determined. This information can be used to evaluate the health effects; to determine the best method of handling; and to evaluate methods for storage, treatment, or disposal. Key items of interest include

- Physical characteristics
- Synonyms
- Permissible exposure limits (PELs) in the air
- Toxicity in water
- Incompatibilities
- Toxicology
- Safety precautions
- Disposal methods

There are a number of references that can be consulted to define properties of hazardous waste (15–20). It should be noted that these sources define the properties of specific compounds; currently, no source is known that defines the impacts of hazardous waste mixtures.

TABLE 9.5 Hazardous Waste Characteristics (*I, 13, 14*)

Characteristic	EPA HZ Number	Criteria	Applicable test procedure																												
Ignitability	D001	<ol style="list-style-type: none"> 1. Liquid that is less than 24% alcohol by volume 2. Liquid that exhibits a flash point less than 140°F (60°C) 3. Nonliquid that can cause a fire 	<ul style="list-style-type: none"> • Pensky-Martens Closed Cup Tester (ASTM Standard D-93-79 or D-93-80)¹ • Setaflash Closed Cup Tester (ASTM Standard D-3278-78) • For a compressed gas, criteria defined in 49 CFR 173.300 • For an oxidizer, criteria defined in 49 CFR 173.151 																												
Corrosivity	D002	<ol style="list-style-type: none"> 1. Aqueous and exhibits a pH less than 2 or greater than 12.5 2. Liquid that corrodes steel (SAE 1020) at a rate greater than 0.25 in (6.35 mm) per year at 130°F (55°C) 	<ul style="list-style-type: none"> • USEPA Method 5.2 (see USEPA. <i>Test Methods for Evaluation of Solid Waste, Physical/Chemical Methods</i>) • NACE Standard TM-01-69² 																												
Reactivity	D003	<ol style="list-style-type: none"> 1. Normally unstable 2. Reacts violently with water 3. Forms potentially explosive mixtures with water 4. When mixed with water, generates toxic gases, vapors, or fumes in quantity sufficient to present a danger to human health or the environment 5. Cyanide or sulfide, which will produce toxic gases between pH 2 and 12.5 6. Can detonate or capable of explosive reaction or 7. Forbidden explosive (49 CFR 173.51) Class A explosive (49 CFR 173.53) Class B explosive (49 CFR 173.88) 																													
EP toxicity	D004 D005 D006 D007 D008 D009 D010 D011 D012 D013 D014 D015 D016 D017	<table> <tbody> <tr> <td>Arsenic</td> <td>5.0 mg/L</td> </tr> <tr> <td>Barium</td> <td>100.0 mg/L</td> </tr> <tr> <td>Cadmium</td> <td>1.0 mg/L</td> </tr> <tr> <td>Chromium</td> <td>5.0 mg/L</td> </tr> <tr> <td>Lead</td> <td>5.0 mg/L</td> </tr> <tr> <td>Mercury</td> <td>0.2 mg/L</td> </tr> <tr> <td>Selenium</td> <td>1.0 mg/L</td> </tr> <tr> <td>Silver</td> <td>5.0mg/L</td> </tr> <tr> <td>Endrin</td> <td>0.02 mg/L</td> </tr> <tr> <td>Lindane</td> <td>0.4 mg/L</td> </tr> <tr> <td>Methoxychlor</td> <td>10.0 mg/L</td> </tr> <tr> <td>Toxaphene</td> <td>0.5 mg/L</td> </tr> <tr> <td>2,4-D</td> <td>10.0 mg/L</td> </tr> <tr> <td>2,4,5-TP Silvex</td> <td>1.0 mg/L</td> </tr> </tbody> </table>	Arsenic	5.0 mg/L	Barium	100.0 mg/L	Cadmium	1.0 mg/L	Chromium	5.0 mg/L	Lead	5.0 mg/L	Mercury	0.2 mg/L	Selenium	1.0 mg/L	Silver	5.0mg/L	Endrin	0.02 mg/L	Lindane	0.4 mg/L	Methoxychlor	10.0 mg/L	Toxaphene	0.5 mg/L	2,4-D	10.0 mg/L	2,4,5-TP Silvex	1.0 mg/L	<ul style="list-style-type: none"> • EP toxic procedure set forth in the Code of Federal Regulations, 40 CFR Part 261 Appendix II
Arsenic	5.0 mg/L																														
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Cadmium	1.0 mg/L																														
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Toxaphene	0.5 mg/L																														
2,4-D	10.0 mg/L																														
2,4,5-TP Silvex	1.0 mg/L																														

Delisting

The EPA has provided a mechanism for delisting of a listed hazardous waste (22). This process covers hazardous waste from nonspecific sources (F Series, 40 CFR 261.31) and from specific sources (K Series, 40 CFR 261.32) (1). The generator of the listed hazardous waste must submit a formal petition to the EPA or the applicable state regulatory agency to receive a delisting. Table 9.6 presents the elements that must be included in a delisting application.

Sampling, Preservation, and Handling

The two major concerns when sampling hazardous waste are representativeness and safety. If the source and nature of the hazardous waste are known, then the personnel performing the sampling should research the properties of the material to determine necessary safety precautions, such as

- Protective clothing
- Equipment for protection from inhalation of fumes or dust
- Special handling precautions

In a case where the nature of the hazardous waste is unknown, such as at an abandoned waste disposal site, then the sampling personnel should take additional precautions to protect themselves from direct contact with the waste material. References should be reviewed and understood prior to proceeding with sampling of closed drums with unknown contents and/or materials buried at an abandoned waste site (11, 23–27).

TABLE 9.6 Elements of a Delisting Application (1, 22)

General information

1. Petitioner's name and address
2. Statement of petitioner's interest and proposed action
3. Description of proposed action
4. Justification for proposed action, including supporting information

Specific information

1. Name and address of laboratory facility performing sampling and testing of waste
 2. Name and qualifications of person sampling and testing the waste
 3. Dates of sampling and testing
 4. Location of the generating facility
 5. Description of manufacturing processes or other operations and feed materials producing the waste, and an assessment of whether such processes, operations, or feed materials can or might produce a waste that is not covered by the test demonstration
 6. Description of the waste and estimate of the average and maximum monthly and annual quantities of waste generated
 7. Pertinent data on and discussion of the factors delineated in respective criteria for listing the hazardous waste [presented in Section 261.11(a)(3)]
 8. Description of the methodologies and the equipment used to obtain representative samples
 9. Description of the sample handling and preservation techniques, including techniques used for extraction, containerization, and preservation of the samples
 10. Description of test performed and presentation of test results
 11. Name and model number of the instruments used to perform the test
 12. Certification statement by authorized representative of the petitioner
-

To determine the nature of a waste material, it is important that representative samples be collected and analyzed. If the waste is in liquid form in drums, it should be completely mixed prior to sampling. Note, however, that precautions must be taken to ensure that the nature of the material is such that mixing will not result in an unsafe environment for the person sampling. If the material can be mixed, it should be done, and an aliquot taken from each separate container of liquid. If there is a group of drums of similar wastes, then random sampling should be performed on a representative number of the drums. Typically, 20% sampling is sufficient to characterize drums of similar waste. In other words, if there are 20 drums, 4 drums would be sampled. If the sampler is not sure of the contents of each drum, then all unknown drums require sampling and analysis.

Generally, if the source of the waste is a solid from a manufacturing or waste treatment process, then composite sampling and analysis are performed. In such cases, an aliquot is periodically collected and composited, and the composite sample is submitted for analysis. If the solid waste is present in a lagoon, abandoned disposal facility, tank, or similar facility, then three-dimensional sampling usually is recommended. In some instances, samples collected three-dimensionally are composited; however, in most cases, they are analyzed individually. The objective is to characterize the solid waste to determine whether or not the entire quantity of material is hazardous.

If the source and nature of the material are known, then the sampling and analysis are limited to the parameters expected to be present and those mandated by the regulating agency. In a situation where the waste is an unknown, a full spectrum of analyses often is required, consisting of analysis for hazardous substances. The elements and compounds designated as hazardous substances by the EPA are presented in Table 9.7. The designation includes any isomers and hydrates, as well as any solutions and mixtures containing these substances. The hazardous substances listed in Table 9.7 are identified by common name, Chemical Abstract System (CAS) number, and synonym(s). In cases of disparity, the common names are considered the designated substance.

Once the suite of needed laboratory analyses is determined, the method for collecting representative samples is developed. A summary of recommended sample container, preservation requirements, and minimum holding times are presented in Table 9.8 (page 9.22) for soil/sediment and waste. Those procedures applicable to liquids are presented in Chapter 6.

Since there is always the possibility that the results of hazardous waste sampling and analysis may be presented in a court case, it is recommended that all handling of samples be performed under chain-of-custody procedures. This requires that documentation be maintained regarding all personnel involved in handling, storage, and analysis of the samples from the point where the empty sample containers are prepared through final disposal of the sample, after analysis. The samples must be maintained under lock and key at all times or signed out to the specific individual performing a laboratory test. Figure 9.4 (page 9.23) presents a typical chain-of-custody sheet, which is to be maintained with the samples at all times.

An important component of the sampling program is the quality assurance and quality control (QA/QC) plan, the objective of which is to ensure that there has been no external contamination of the samples from initial sampling through final analysis. Generally, duplicate, field blank, and travel blank samples should be prepared and subjected to laboratory analysis. Typically, 10% of the samples should be collected in duplicate. A travel blank is a contaminant-free sample prepared in the laboratory that travels with the empty bottles to the sampling site and returns to the laboratory with the samples. Typically, two travel blanks are prepared and shipped. Field blanks are empty sample bottles that are prepared in the field using contaminant-free water following the general sampling procedures used in the field for collection of all waste samples. These field blanks are returned to the laboratory for analysis. Travel blanks identify if there is a problem of contamination in the preparation of the sample containers and shipping procedures, whereas field blanks address contamination issues with the field sampling procedures. Duplicate sampling requires splitting one field sample into two aliquots for laboratory analysis. Duplicates demonstrate the reproducibility of the sampling procedure. In the case of liquid samples, all three types of QA/QC samples are prepared. For soil, semisolids, sludges, and solids, only duplicate samples are typically prepared.

TABLE 9.7 List of Hazardous Substances (28)

Common name	CAS Number*	Synonyms
Acetaldehyde	75070	Ethanal, ethyl aldehyde, acetic aldehyde
Acetic acid	64197	Glacial acetic acid, vinegar acid
Acetic anhydride	108247	Acetic oxide, acetyl oxide
Acetone cyanohydrin	75865	2-methylactonitrile, alpha-hydroxyisobutyronitrile
Acetyl bromide	506967	
Acetyl chloride	79367	
Acrolein	107028	2-propenal, acrylic aldehyde, acrylaldehyde, acraldehyde
Acrylonitrile	107131	Cyanoethylene, Fumigrain, Ventox, propeneitrile, vinyl cyanide
Adipic acid	124049	Hexanedioic acid
Aldrin	309002	Octalene, HHDN
Allyl alcohol	107186	2-propen-1-ol, 1-propenol-3, vinyl carbinol
Allyl chloride	107051	3-chloropropene, 3-chloropropylene, Chlorallylene
Aluminum sulfate	10043013	Alum
Ammonia	7664417	
Ammonium acetate	631618	Acetic acid ammonium, salt
Ammonium benzoate	1863634	
Ammonium bicarbonate	1066337	Acid ammonium carbonate, ammonium hydrogen carbonate
Ammonium bichromate	7789095	
Ammonium bifluoride	1341497	Acid ammonium fluoride, ammonium hydrogen fluoride
Ammonium bisulfite	10192300	
Ammonium carbamate	1111780	Ammonium aminofornate
Ammonium carbonate	506876	
Ammonium chloride	12125029	Ammonium muriate, sal ammoniac, salmiac, Amchlor
Ammonium chromate	7788989	
Ammonium citrate dibasic	3012655	Diammonium citrate, citric acid diammonium salt
Ammonium fluoborate	13826830	Ammonium fluoroborate, ammonium borofluoride
Ammonium fluoride	12125018	Neutral ammonium fluoride
Ammonium hydroxide	1336216	
Ammonium oxalate	6009707	
	5972736	
	14258492	
Ammonium silicofluoride	16919190	Ammonium fluosilicate
Ammonium sulfamate	7773060	Ammate, AMS, ammonium amidosulfate
Ammonium sulfide	12135761	
Ammonium sulfite	10196040	
	10192300	
Ammonium tartrate	3164292	Tartaric acid ammonium salt
	14307438	
Ammonium thiocyanate	1762954	Ammonium rhodanide, ammonium sulfocyanate, ammonium sulfocyanide
Amyl acetate	628637	Amylacetic ester, pear oil, banana oil
Aniline	62533	Aniline oil, phenylamine, aminobenzene, aminophen, kyanol
Antimony pentachloride	7647189	
Antimony potassium tartrate	28300745	Tartar emetic, tartrated antimony, tartarized antimony, potassium antimonyltartrate.
Antimony tribromide	7789619	
Antimony trichloride	10025919	Butter of antimony
Antimony trifluoride	7783564	Antimony fluoride
Antimony trioxide	1309644	Diantimony trioxide, flowers of antimony
Arsenic disulfide	1303328	Red arsenic sulfide

TABLE 9.7 List of Hazardous Substances (28) (*continued*)

Common name	CAS Number*	Synonyms
Arsenic pentoxide	1303282	Arsenic acid anhydride, arsenic oxide
Arsenic trichloride	7784341	Arsenic chloride, arsenious chloride, arsenous chloride, butter of arsenic
Arsenic trioxide	1327533	Arsenious acid, arsenious oxide, white arsenic
Arsenic trisulfide	1303339	Arsenious sulfide, yellow arsenic sulfide
Barium cyanide	542621	
Benzene	71432	Cyclohexatriene, benzol
Benzoic acid	65850	Benzenecarboxylic acid, phenylformic acid, dracylic acid
Benzonitrile	100470	Phenyl cyanide, cyanobenzene
Benzoyl chloride	98884	Benzenecarbonyl chloride
Benzyl chloride	100447	
Beryllium chloride	7787475	
Beryllium fluoride	7787497	
Beryllium nitrate	7787555	
	13597994	
Butyl acetate	123864	Acetic acid butyl ester
Butylamine	109739	1-aminobutane
<i>n</i> -butyl phthalate	84742	1,2-benzenedicarboxylic acid, dibutyl ester, dibutyl phthalate
Butyric acid	107926	Butanoic acid, ethylacetic acid
Cadmium acetate	543908	
Cadmium bromide	7789426	
Cadmium chloride	10108642	
Calcium arsenate	7778441	Tricalcium orthoarsenate
Calcium arsenite	52740166	
Calcium carbide	75207	Carbide, acetylenogen
Calcium chromate	13765190	Calcium chrome yellow, geblin, yellow ultramarine.
Calcium cyanide	592018	
Calcium dodecylbenzenesulfonate	26264062	
Calcium hypochlorite	7778543	
Captan	133062	Orthocide-406, SR-406, Vancide-89
Carbaryl	63252	Sevin
Carbofuran	1563662	Furadan
Carbon disulfide	75150	Carbon bisulfide, dithiocarbonic anhydride
Carbon tetrachloride	56235	Tetrachloromethane Perchloromethane
Chlordane	57749	Toxichlor, chlordan
Chlorine	75003	
Chlorobenzene	108907	Monochlorobenzene, benzene chloride
Chloroform	67663	Trichloromethane
Chlorpyrifos	2921882	Dursban
Chlorosulfonic acid	7790945	Sulfuric chlorohydrin
Chromic acetate	1066304	
Chromic acid	11115745	Chromic anhydride, chromium trioxide
Chromic sulfate	10101538	
Chromous chloride	10049055	
Cobaltous bromide	7789437	Cobalt bromide
Cobaltous formate	544183	Cobalt formate
Cobaltous sulfamate	14017415	Cobalt sulfamate
Coumaphos	56724	Co-Ral
Cresol	1319773	Cresylic acid, hydroxytoluene

(continues)

TABLE 9.7 List of Hazardous Substances (28) (*continued*)

Common name	CAS Number*	Synonyms
Crotonaldehyde	4170303	2-butenal propylene aldehyde
Cupric acetate	142712	Copper acetate, crystallized verdigris
Cupric acetoarsenite	12002038	Copper acetoarsenite, copper acetate arsenite, Paris green
Cupric chloride	7447394	Copper chloride
Cupric nitrate	3251238	Copper nitrate
Cupric oxalate	5893663	Copper oxalate
Cupric sulfate	7758987	Copper sulfate
Cupric sulfate, ammoniated	10380297	Ammoniated copper sulfate
Cupric tartrate	815827	Copper tartrate
Cyanogen chloride	506774	
Cyclohexane	110827	Hexahydrobenzene, hexamethylene, hexanaphthene
2,4-D acid	94757	2,4-dichlorophenoxyacetic acid
2,4-D ester	94111	2,4-dichlorophenoxyacetic acid ester
	94791	
	94804	
	1320189	
	1928387	
	1928616	
	1929733	
	2971382	
	25168267	
	53467111	
DOT	50293	<i>p,p'</i> -DDT
Diazinon	333415	Dipofene, Diazitol, Basudin, Spectracide
Dicamba	1918009	2-methoxy-3,6-dichlorobenzoic acid
Dichlobenil	1194656	2,6-dichlorobenzonitrile, 2,6-DBN
Dichlone	117806	Phygon, dichloronaphthoquinone
Dichlorobenzene	25321226	Di-chloricide Paramoth (Para)
Dichloropropane	26638197	Propylene dichloride
Dichloropropene	26952238	
Dichloropropene– dichloropropane (mixture)	8003198	D-D mixture, Vidden D
2,2-Dichloropropionic acid	75990	Dalapon
Dichlorvos	62737	2,2-dichlorovinyl dimethyl phosphate, Vapona
Dicofol	115322	Di(<i>p</i> -chlorophenyl)-trichloromethylcarbinol, DTMC, dicofol
Dieldrin	60571	Alvit
Diethylamine	109897	
Dimethylamine	124403	
Dinitrobenzene (mixed)	25154545	Dinitrobenzol
Dinitrophenol	51285	Aldifen
Diquat	85007	Aquacide
	2764729	Dextrone, Reglone, Diquat dibromide
Disulfoton	298044	Di-syston
Diuron	330541	DCMU, DMU
Dodecylbenzenesulfonic acid	27176870	
Endosulfan	115297	Thiodan
Endrin	72208	Mendrin, Compound 269
Epichlorohydrin	106898	-chloropropylene oxide
Ethion	563122	Nialate, ethyl methylene, phosphorodithioate

TABLE 9.7 List of Hazardous Substances (28) (*continued*)

Common name	CAS Number*	Synonyms
Ethylbenzene	100414	Phenylethane
Ethylenediamine	107153	1,2-diaminoethane
Ethylenediamine-tetraacetic acid (EDTA)	60004	Edetic acid, Havidote, (ethylenedinitrilo)-tetraacetic acid
Ethylene dibromide	106934	1,2-dibromoethane acetylene dibromide sym-dibromoethylene
Ethylene dichloride	107062	1,2-dichloroethane sym-bichloroethane
Ferric ammonium citrate	1185575	Ammonium ferric citrate
Ferric ammonium oxalate	2944674	Ammonium ferric oxalate
	55488874	
Ferric chloride	7705080	Flores martis, iron trichloride
Ferric fluoride	7783508	
Ferric nitrate	10421484	Iron nitrate
Ferric sulfate	10028225	Ferric persulfate, ferric sesquisulfate, ferric tersulfate
Ferrous ammonium sulfate	10045893	Mohr's salt, iron ammonium sulfate
Ferrous chloride	7758943	Iron chloride, iron dichloride, iron protochloride
Ferrous sulfate	7720787	Green vitriol
	7782630	Iron vitriol, Iron sulfate, iron protosulfate
Formaldehyde	50000	Methyl aldehyde, methanal, formalin
Formic acid	64186	Methanoic acid
Fumaric acid	110178	Trans-butenedioic acid, trans-1,2-ethylenedicarboxylic acid, boletic acid, allomaleic acid
Furfural	98011	2-furaldehyde, pyromucic aldehyde
Guthion	86500	Gusathion, azinphos-methyl
Heptachlor	76448	Velsicol-104, Drinox, Heptagran
Hexachlorocyclopentadiene	77474	Perchlorocyclopentadiene
Hydrochloric acid	7647010	Hydrogen chloride, muriatic acid
Hydrofluoric acid	7664393	Fluohydric acid
Hydrogen cyanide	74908	Hydrocyanic acid
Hydrogen sulfide	7783064	Hydrosulfuric acid sulfur hydride
Isoprene	78795	2-methyl-1,3-butadiene
Isopropanolamine dodecylbenzenesulfonate	42504461	
Kepone	143500	Chlordecone 1,1a,3,3a,4,5,5a,5b,6-decachlorooctahydro-1,3,4-metheno-2H-cyclobuta(cd)pentalen-2-one
Lead acetate	301042	Sugar of lead
Lead arsenate	7784409	
	7645252	
	10102484	
Lead chloride	7758954	
Lead fluoborate	13814965	Lead fluoroborate
Lead fluoride	7783462	Lead difluoride, plumbous fluoride
Lead iodide	10101630	
Lead nitrate	10099748	
Lead stearate	7428480	Stearic acid lead salt
	1072351	
	52652592	
Lead sulfate	7446142	
Lead sulfide	1314870	Galena
Lead thiocyanate	592870	Lead sulfocyanate
Lindane	58899	Gamma-BHC, gamma-benzene hexachloride

(continues)

TABLE 9.7 List of Hazardous Substances (28) (*continued*)

Common name	CAS Number*	Synonyms
Lithium chromate	14307358	
Malathion	121755	Phosphothion
Maleic acid	110167	Cis-butenedioic acid, <i>cis</i> -1,2-ethylenedicarboxylic acid, toxilic acid
Maleic anhydride	108316	2,5-furandione, <i>cis</i> -butenedioic anhydride, toxilic anhydride
Mercaptodimethur	203657	Mesurool
Mercuric cyanide	592041	Mercury cyanide
Mercuric nitrate	10045940	Mercury nitrate, mercury pernitrate
Mercuric sulfate	7783359	Mercury sulfate, mercury persulfate
Mercuric thiocyanate	592858	Mercury thiocyanate, mercuric sulfocyanate, mercuric sulfocyanide
Mercurous nitrate	7782867	
	10415755	Mercury protonitrate
Methoxychlor	72435	DMDT, methoxy-DDT
Methyl mercaptan	74931	Methanethiol, mercaptomethane, methyl sulfhydrate, thiomethyl alcohol
Methyl methacrylate	80626	Methacrylic acid methyl ester, methyl-2-methyl-2-propenoate
Methyl parathion	298000	Nitrox-80
Mevinphos	7786347	Phosdrin
Mexacarbate	315184	Zectran
Monoethylamine	75047	Ethylamine, aminoethane
Monomethylamine	74895	Methylamine, aminomethane
Naled	300765	Dibrom
Naphthalene	91203	White tar, tar camphor, naphthalin
Naphthenic acid	1338245	Cyclohexanecarboxylic acid, hexahydrobenzoic acid.
Nickel ammonium sulfate	15699180	Ammonium nickel sulfate
Nickel chloride	37211055	Nickelous chloride
	7718549	
Nickel hydroxide	12054487	Nickelous hydroxide
Nickel nitrate	14216752	
Nickel sulfate	7786814	Nickelous sulfate
Nitric acid	7697372	Aqua fortis
Nitrobenzene	98953	Nitrobenzol, oil of mirbane
Nitrogen dioxide	10102440	Nitrogen tetraoxide
Nitrophenol (mixed)	25154556	Mononitrophenol
Nitrotoluene	1321126	
Paraformaldehyde	30525894	Paraform, Formagene, Triformol, polymerized formaldehyde, polyoxymethylene
Parathion	56382	DNTP, Niran
Pentachlorophenol	87865	PCP, Penta
Phenol	108952	Carbolic acid, phenyl hydroxide, hydroxybenzene, oxybenzene
Phosgene	75445	Diphosgene, carbonyl chloride, chloroformyl chloride
Phosphoric acid	7664382	Orthophosphoric acid
Phosphorus	7723140	Black phosphorus, red phosphorus, white phosphorus, yellow phosphorus
Phosphorus oxychloride	10025873	Phosphoryl chloride, phosphorus chloride
Phosphorus pentasulfide	1314803	Phosphoric sulfide, thiophosphoric anhydride, phosphorus persulfide
Phosphorus trichloride	7719122	Phosphorous chloride
Polychlorinated biphenyls	1336363	PCB, Aroclor, polychlorinated diphenyls

TABLE 9.7 List of Hazardous Substances (28) (*continued*)

Common name	CAS Number*	Synonyms
Potassium arsenate	7784410	
Potassium arsenite	10124502	Potassium metaarsenite
Potassium bichromate	7778509	Potassium dichromate
Potassium chromate	7789006	
Potassium cyanide	151508	
Potassium hydroxide	1310583	Potassium hydrate, caustic potash, potassa
Potassium permanganate	7722647	Chameleon mineral
Propargite	2312358	Omite
Propionic acid	79094	Propanoic acid, methylacetic acid, ethylformic acid
Propionic anhydride	123626	Propanoic anhydride, methylacetic anhydride
Propylene oxide	75569	Propene oxide
Pyrethrins	121299	Pyrethrin I
	121211	Pyrethrin II
Quinoline	91225	1-benzazine, benzo(b)pyridine, leucoline, chinoleine, leucol
Resorcinol	108463	Resorcin, 1,3-benzenediol, metadihydroxybenzene.
Selenium oxide	7446084	Selenium dioxide
Silver nitrate	7761888	Nitric acid silver (1+) salt lunar caustic
Sodium	7440235	Natrium
Sodium arsenate	7631892	Disodium arsenate
Sodium arsenite	7784465	Sodium metaarsenite
Sodium bichromate	10588019	Sodium dichromate
Sodium bifluoride	1333831	
Sodium bisulfite	7631905	Sodium acid sulfite, sodium hydrogen sulfite
Sodium chromate	7775113	
Sodium cyanide	143339	
Sodium dodecylbenzene-sulfonate	25155300	
Sodium fluoride	7681494	Villiumite
Sodium hydrosulfide	16721805	Sodium hydrogen sulfide
Sodium hydroxide	1310732	Caustic soda, soda lye, sodium hydrate
Sodium hypochlorite	7681529	Bleach
	10022705	
Sodium methylate	124414	Sodium methoxide
Sodium nitrite	7632000	
Sodium phosphate, dibasic	7558794	
	10039324	
	10140655	
Sodium phosphate, tribasic	7785844	
	7601549	
	10101890	
	10361894	
	7758294	
	10124568	
Sodium selenite	10102188	
	7782823	
Strontium chromate	7789062	
Strychnine	57249	
Styrene	100425	Vinylbenzene, phenylethylene, styrol, styrolene, cinnamene, cinnamol.
Sulfuric acid	7664939	Oil of vitriol, oleum
Sulfur monochloride	12771083	Sulfur chloride

(continues)

TABLE 9.7 List of Hazardous Substances (28) (*continued*)

Common name	CAS Number*	Synonyms
2,4,5-T acid	93765	2,4,5-trichlorophenoxyacetic acid
2,4,5-T amines	6369966	Acetic acid (2,4,5-trichlorophenoxy)-compound with N,N-dimethylmethanamine (1:1)
	6369977	Acetic acid (2,4,5-trichlorophenoxy)-compound with N-methylmethanamine (1:1)
	1319728	Acetic acid (2,4,5-trichlorophenoxy)-compound with 1-amino-2-propanol (1:1)
	3813147	Acetic acid (2,4,5-trichlorophenoxy)-compound with 2,2'-nitrioltris [ethanol] (1:1)
2,4,5-T esters	2545597	2,4,5-trichlorophenoxyacetic esters
	93798	
	61792072	
	1928478	
	25168154	
2,4,5-T salts	13560991	Acetic acid (2,4,5-trichlorophenoxy)-SOdiUm salt
TDE	72548	DDD
2,4,5-TP acid	93721	Propanoic acid 2-(2,4,5-trichlorophenoxy)
2,4,5-TP esters	32534955	Propanoic acid, 2-(2,4,5-trichlorophenoxy)-isooctyl ester.
Tetraethyl lead	78002	Lead tetraethyl, TEL
Tetraethyl pyrophosphate	107493	TEPP
Thallium sulfate	10031591	
	7446186	
Toluene	108883	Toluol, methylbenzene, phenylmethane, Methacide
Toxaphene	8001352	Camphechlor
Trichlorfon	52686	Dipterex Dylox
Trichlorethylene	79016	Ethylene trichloride
Trichlorophenol	25167822	Collunosol, Dovicide 2 or 2S, Omal, Phenachlor
Triethanolamine	27323417	
dodecylbenzenesulfonate		
Triethylamine	121448	
Trimethylamine	75503	TMA
Uranyl acetate	541093	
Uranyl nitrate	10102064	
	36478769	
Vanadium pentoxide	1314621	Vanadic anhydride, vanadic acid anhydride
Vanadyl sulfate	27774136	Vanadic sulfate, vanadium sulfate
Vinyl acetate	108054	Acetic acid ethylene ether
Vinylidene chloride	75354	1,1-dichlorethylene 1,1-dichloroethene
Xylene (mixed)	1330207	Dimethylbenzene Xylol
Xylenol	1300716	Dimethylphenol, hydroxydimethylbenzene
Zinc acetate	557346	
Zinc ammonium chloride	14639975	
	14639986	
	52628258	
Zinc borate	1332076	
Zinc bromide	7699458	
Zinc carbonate	3486359	

TABLE 9.7 List of Hazardous Substances (28) (*continued*)

Common name	CAS Number*	Synonyms
Zinc chloride	7646857	Butter of zinc
Zinc cyanide	557211	
Zinc fluoride.	7783495	
Zinc formate	557415	
Zinc hydrosulfite	7779864	
Zinc nitrate	7779886	
Zinc phenolsulfonate	127822	Zinc sulfocarbolate
Zinc phosphide	1314847	
Zinc silicofluoride	16871719	Zinc fluosilicate
Zinc sulfate	7733020	White vitriol, zinc vitriol, white copperas
Zirconium nitrate	13746899	
Zirconium potassium fluoride	16923958	
Zirconium sulfate	14644612	Disulfatozirconic acid
Zirconium tetrachloride	10026116	

*Chemical Abstract System number.

Analysis

Analysis of hazardous waste or potentially hazardous materials can be extremely complicated and costly. If the source and nature of the waste are known, all that is required is a determination of whether the material exhibits hazardous waste characteristics—a relatively straightforward process. However, when the source of the waste is unknown, testing the waste for the full spectrum of the 129 priority pollutants may be necessary. If there are a number of samples involved, the analytical costs can be substantial. The typical cost for the full spectrum of priority pollutant analyses ranges from \$500 to \$1500 per sample. Table 9.9 (page 9.24) presents the recommended analytical procedures for various categories of contaminants. Typically, organics analysis is performed via gas chromatography and mass spectrometry (GC/MS). This method can identify organics within the particular spectrum being scanned. Typical sensitivity is on the order of 1 to 100 parts per billion (ppb), depending on the specific organic compound and the concentration of similar compounds which may interfere with the analysis. This procedure provides good quantification and excellent qualification regarding the nature of the organics.

Additional procedures are available using GC methods only, which will increase sensitivity; however, identification of the spectrum of organic compounds is not practical. Once the nature of the contaminant is defined, GC procedures can be used, in many cases, to provide better quantification at a lower cost. There are a number of references which should be consulted when determining the analytical protocol for the samples (29–34).

Due to the high costs associated with analysis of hazardous waste samples, one of several available analytical screening procedures should be used. It may be beneficial to prepare several representative composites and subject them to screening procedures. Based on the data obtained, the analytical program then can be directed to focus only on the major chemical constituents of concern, resulting in significant cost savings. Recommended screening tests include

- pH
- Conductivity
- Total organic carbon

TABLE 9.8 Recommended Containers, Preservation, and Holding Times for Soil/Sediment and Waste Samples (124)

Analytical group	Soil/Sediment			Waste		
	Container ^a	Preservation ^b	Holding time (days) ^c	Container ^a	Preservation ^b	Holding time (days) ^c
Inorganics						
pH ¹	8G	NA	—	8G	NA	N
Dermal corrosion	—	—	—	8G	NA	N
Flashpoint	—	—	—	8G	NA	N
Btu content	—	—	—	8G	NA	N
Ash content	—	—	—	8G	NA	N
Nutrients (N, P)	8G	I	NS	—	—	—
Ortho-P	8G	I	NS	—	—	—
Dissolved P	—	—	—	—	—	—
COD	8G	I	NS	—	—	—
Metals	8G	I	180	8G	NA	180
Mercury	8G	I	180	8G	NA	180
Metals—TCLP	8G	I	360 ⁴	8G	NA	360 ⁴
Metals—EP	8G	I	360 ⁴	8G	NA	360 ⁴
Cyanide	—	—	—	8G	NA	14
<i>Organics</i>						
VOCs ¹	2G	I	14	8G	NA	14
VOCs—TCLP ¹	2G	I	28 ⁵	8G	NA	28 ⁵
Extractables ²	8G	I	54 ⁶	8G	NA	54 ⁶
Extractables—TCLP	8G	I	61 ⁷	8G	NA	61 ⁷
Dioxins ³	A	I	75 ⁸	A	I	75 ²
Percent alcohol	8G	I	NS	8G	NA	NS
Organic halide (TOX)	8G	I	28	—	—	—

^aContainers: 8G—8 oz. widemouth glass (Teflon lid); 2G—2 oz. widemouth glass (Teflon septum lid); A—500 ml wide-mouth amber glass (Teflon lid).

^bPreservatives: I—ice (4°C); NA—not applicable.

^cHolding times (in days unless noted otherwise): NS—not Specified; N—indefinite; 4—360 days (180 days to extraction plus 180 days to analysis); 5—28 days (14 days to TCLP extraction plus 14 days to analysis (7 days if not preserved following extraction)); 6—54 days (14 days to extraction, 40 days to analysis); 7—61 days (14 days to TCLP extraction, 7 days to solvent extraction, 40 days to analysis); 8—Method 8290 specifies 30 days to extraction plus 45 days to analysis.

¹Grab sample only, unless indicated a grab or composite is acceptable.

²Including pesticides, herbicides and PCBs.

³Consult local laboratory for most recent dioxin container and preservation requirements.

Shipping Note: When samples are to be shipped by common carrier or sent through the United States mail, the shipper must comply with the Department of Transportation Hazardous Materials Regulations (49 CFR 172). The person offering such material for transportation is responsible for ensuring such compliance.

HAZARDOUS WASTE

HAZARDOUS WASTE

9.23

SURVEY DESCRIPTION & LOCATION				1 EMPTIES PREPARED FOR SHIPMENT (Signature) _____ DATE _____ TIME _____					
2 LABORATORY DISPATCH OF EMPTIES (Signature)(Seal Attest-If Approp.) _____ NO. _____ TYPE _____ DATE _____ TIME _____				3 EMPTIES RECEIVED BY COURIER (Signature) _____ DATE _____ TIME _____					
5 EMPTIES RECEIVED (IF BY OTHER THAN SAMPLER) (Signature) _____ DATE _____ TIME _____				4 EMPTIES RELINQUISHED BY COURIER (Signature) _____ DATE _____ TIME _____					
6 EMPTIES RECEIVED BY SAMPLER (Signature) _____ DATE _____ TIME _____				7 EMPTIES RECEIVED BY SAMPLER(S) (Signature)(Seal Attest-If Approp.) _____ DATE _____ TIME _____					
METHOD OF SHIPMENT		SAMPLING AND ANALYSES AUTHORIZED BY (Name) _____ DATE _____ TIME _____			SAMPLING SURVEY DESCRIPTION				
TO:		DATE _____ TIME _____			SAMPLING SURVEY DESCRIPTION				
FROM:		DATE _____ TIME _____			SAMPLING SURVEY DESCRIPTION				
SAMPLE STATION NO.	SAMPLE LOCATION	DATE	TIME	SAMPLE TYPE			NO. OF CONTAINERS	ANALYSES REQUIRED	
				WATER	AIR	SOLID			
				C	G				
SAMPLES RELINQUISHED BY: (Signature)(Seal attest-If Approp.) _____				SAMPLES RECEIVED BY: (Signature) _____			DATE _____	TIME _____	
SAMPLES RELINQUISHED BY: (Signature) _____				SAMPLES RECEIVED AT LAB BY: (Signature) (Seal Attest) (Custodian) _____			DATE _____	TIME _____	
SAMPLES RELINQUISHED FOR ANALYSES: (Signature) _____				SAMPLES RECEIVED FOR ANALYSES ANALYST: (Signature) _____			DATE _____	TIME _____	
SAMPLES RELINQUISHED AFTER ANALYSES ANALYST: (Signature) _____				ANALYZED SAMPLES RECEIVED & STORED (Signature)(Custodian) _____			DATE _____	TIME _____	
ANALYZED SAMPLES DESTROYED (Signature) (Custodian) _____			DATE _____	TIME _____	CUSTODY RECORD VERIFICATION			DATE _____	TIME _____

DISTRIBUTION: ORIGINAL ACCOMPANIES EMPTIES AND SAMPLES

FIGURE 9.4 Typical chain of custody record.

TABLE 9.9 Recommended Method for Analysis

Analytical category	Recommended method for analysis*
Volatile organics	GC/MS (USEPA Method 624)
Acid-extractable organics	GC/MS (USEPA Method 625)
Base and neutral organics	GC/MS (USEPA Method 625)
TCDD (dioxin)	GC/MS (USEPA Method 608)
Pesticides and PCBs	GCIMS (USEPA Method 625)
Metals	Atomic absorption (flame or graphite) [†]
Mercury	Cold vapor atomic absorption spectroscopy
Cyanide	EPA colorimetric method
Asbestos	Fibrous asbestos method
Anions (SO ₄ ²⁻ , F ⁻ , Cl ⁻)	Ion chromatography
Oil and grease	Freon extraction and gravimetric measurement
Purgeable halocarbons	GC (USEPA Method 601)
Purgeable aromatics	GC (USEPA Method 602)
Acrolein and acrylonitrile	GC (USEPA Method 603)
Phenols	GC (USEPA Method 604)
Benzidine	GC (USEPA Method 605)
Pthalate esters	GC (USEPA Method 606)
Nitrosamines	GC (USEPA Method 607)
Pesticides and PCBs	GC (USEPA Method 608)
Nitroaromatics and isophorone	GC (USEPA Method 609)
Polynuclear aromatic hydrocarbons	GC (USEPA Method 610)
Chlorinated hydrocarbons	GC (USEPA Method 611)
TCDD (dioxin screening)	GC (USEPA Method 612)

*GC/MS = gas chromatography/mass spectrometry; GC = gas chromatography.

[†]Graphite furnace is a more sensitive technique.

- Total phenols
- Organic scan (FID)
- Halogenated (ECD)
- Volatile organic scan
- Nitrogen–phosphorus organic scan
- Metals (via inductively coupled plasma atomic admission spectroscopy)

Compatibility

In many instances, it is necessary to transport hazardous waste from point of generation to the point of treatment and/or disposal. To facilitate transportation and reduce cost, it is beneficial to consolidate wasteloads. Information is currently available to enable a determination of whether specific wastes can be mixed. Figure 9.5 (pages 9.26 and 9.27) presents a hazardous waste compatibility chart, developed by ASTM Committee D-34. Reaction codes included are (36)

- Heat generation
- Fire
- Innocuous and nonflammable gas generation
- Gas generation
- Toxic gas generation

- Flammable gas generation
- Explosion
- Violent polymerization
- Solubilization of toxic substances

In a case of an abandoned site where the nature of the hazardous waste is unknown, the EPA's Environmental Response Team (ERT) in Edison, New Jersey, has developed preliminary procedures for evaluation compatibility, based on field testing procedures (37). The characteristics evaluated include

- Flammability
- Radiation
- pH
- Water reactivity
- Oxidation/reduction potential (redox)
- Volatile gas or vapor generation

These field procedures should be used with extreme care. A number of synergistic reactions could occur when two waste materials are mixed.

Manifest

In order to transport hazardous waste off a generator's site, the material must be manifested. Figure 9.6 (page 9.28) presents a copy of the universal manifest developed by the EPA, which also incorporates requirements of the state of California. The engineer should check with the appropriate environmental agency of the state which is the point of generation and initial transportation to determine the proper manifest form. This manifest must follow the waste material from point of generation through to the point of ultimate disposal. It is a criminal offense to transport hazardous waste without a manifest. Once the waste is received at the disposal site, a copy of the manifest needs to be returned to the generator to notify that the material has been received at the disposal site.

If a signed copy of the manifest is not returned to the generator within 35 days of the date the waste was shipped, the generator must contact the transporter(s) and/or the designated facility to which the waste is being transported to determine the status. If the signed copy of the manifest is not received by the generator within 45 days of the date of shipment, the generator must file an exception report with the regional administrator of the EPA and/or the appropriate state administrator. The exception report must include:

1. A legible copy of manifest
2. A cover letter from the generator of the waste which explains the efforts taken to locate the hazardous waste and the results of these efforts

NATIONAL PRIORITIES LIST

The Comprehensive Environmental Response, Compensation, and Liability Act of 1980 (CERCLA) was amended in 1986 by the Superfund Amendments and Reauthorization Act (SARA). This legislation requires that the National Oil and Hazardous Substances Pollution Contingency Plan (NCP) include a list of national priorities. The National Priorities List (NPL) includes the known releases or threatened releases of hazardous substances, pollutants, or contaminants throughout the United States (125).

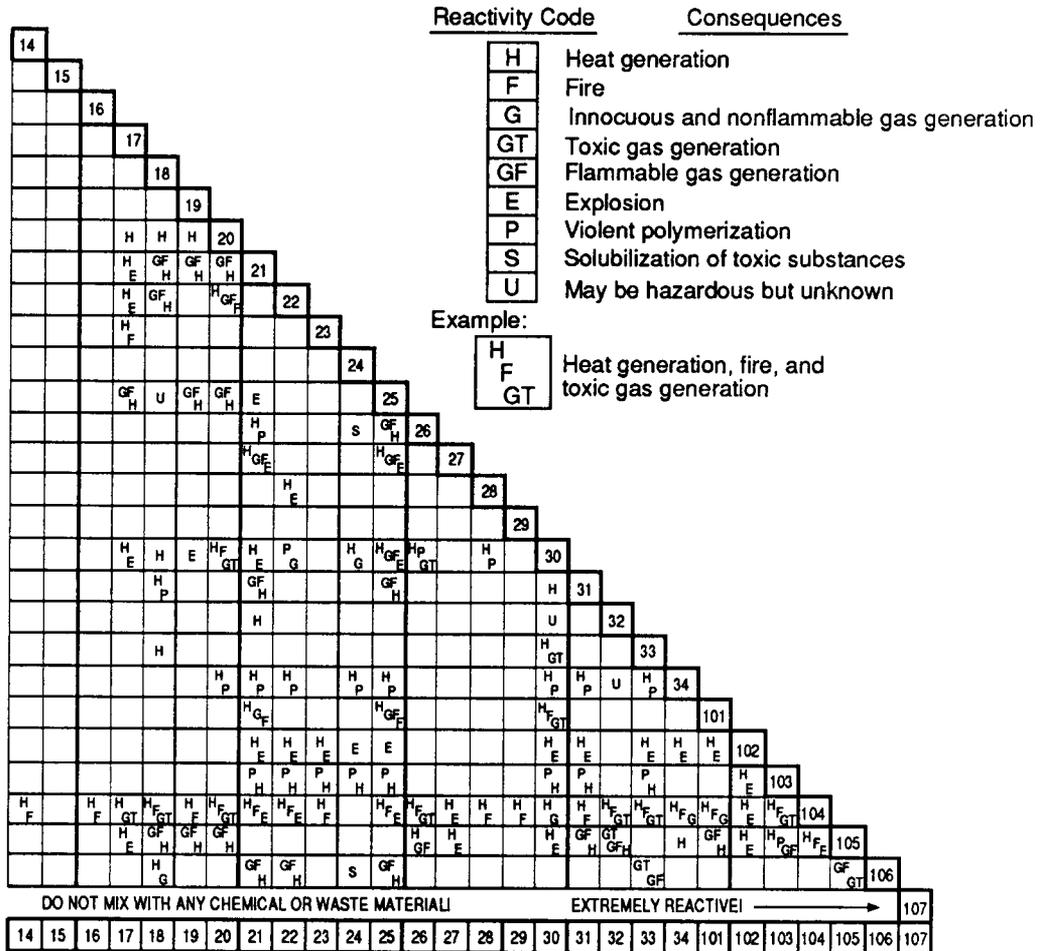


FIGURE 9.5 Hazardous waste compatibility chart (35).

Reactivity group no.	Reactivity group name													
1	Acids, mineral, nonoxidizing	1												
2	Acids, mineral, oxidizing		2											
3	Acids, organic		H	3										
4	Alcohols and glycols	H	H	H	4									
5	Aldehydes	H	H	H	H	5								
6	Amides	H	H	6										
7	Amines, aliphatic and aromatic	H	H	H	H	-								
8	Azo compounds, diazo compounds, and hydrazines	H	H	H	H	H	8							
9	Carbamates	H	H	H	H	H	H	9						
10	Caustics	H	H	H	H	H	H	H	10					
11	Cyanides	GT	GT	GT	GF					11				
12	Dithiocarbamates	H	H	H	H	H	H	H	H	H	12			
13	Esters	H	H	H	H	H	H	H	H	H	H	13		
14	Ethers	H	H											
15	Fluorides, inorganic	GT	GT	GT										
16	Hydrocarbons, aromatic		H	F										
17	Halogenated organics	H	H	H	H	H	H	H	H	H	H			
18	Isocyanates	H	H	H	H	H	H	H	H	H	H	H		
19	Ketones	H	H	H	H	H	H	H	H	H	H			
20	Mercaptans and other organic sulfides	GT	GF	H	H	H	H	H	H	H	H			
21	Metals, alkali and alkaline earth, elemental	GF	GF	GF	GF	GF	GF	GF	GF	GF	GF	GF	GF	
22	Metals, other elemental & alloys as powders, vapors, or sponges	GF	GF	GF										
23	Metals, other elemental & alloys as sheets, rods, drops, moldings, etc.	GF	GF											
24	Metals and metal compounds, toxic	S	S	S	S	S	S	S	S	S	S			
25	Nitrides	GF	H	H	H	H	H	H	H	H	H	H	H	
26	Nitriles	H	H	H	H	H	H	H	H	H	H	H	H	
27	Nitro compounds, organic		H	H	H	H	H	H	H	H	H	H	H	
28	Hydrocarbons, aliphatic, unsaturated	H	H	H	H	H	H	H	H	H	H	H	H	
29	Hydrocarbons, aliphatic, saturated		H	F										
30	Peroxides and hydroperoxides, organic	H	H	H	H	H	H	H	H	H	H	H	H	
31	Phenols and cresols	H	H	H	H	H	H	H	H	H	H	H	H	
32	Organophosphates, phosphothioates, phosphodithioates	H	H	H	H	H	H	H	H	H	H	H	H	
33	Sulfides, inorganic	GT	H	H	H	H	H	H	H	H	H	H	H	
34	Epoxides	H	H	H	H	H	H	H	H	H	H	H	H	
101	Combustible and flammable materials, miscellaneous	H	H	H	H	H	H	H	H	H	H	H	H	
102	Explosives	H	H	H	H	H	H	H	H	H	H	H	H	
103	Polymerizable compounds	H	H	H	H	H	H	H	H	H	H	H	H	
104	Oxidizing agents, strong	H	H	H	H	H	H	H	H	H	H	H	H	
105	Reducing agents, strong	H	H	H	H	H	H	H	H	H	H	H	H	
106	Water and mixtures containing water	H	H											
107	Water reactive substances	← EXTREMELY REACTIVE!												
		1	2	3	4	5	6	7	8	9	10	11	12	13

FIGURE 9.5 (continued)

HAZARDOUS WASTE

9.28

CHAPTER NINE

State of California—Health and Welfare Agency

Department of Health Services
Toxic Substances Control Division
Sacramento, California

Please print or type. (Form designed for use on elite (12-pitch) typewriter.)

UNIFORM HAZARDOUS WASTE MANIFEST		1. Generator's US EPA ID No.	Manifest Document No.	2. Page 1 of	Information in the shaded areas is not required by Federal law.	
3. Generator's Name and Mailing Address				A. State Manifest Document Number 86416635		
4. Generator's Phone ()				B. State Generator's ID		
5. Transporter 1 Company Name		6. US EPA ID Number		C. State Transporter's ID		
7. Transporter 2 Company Name		8. US EPA ID Number		D. Transporter's Phone		
9. Designated Facility Name and Site Address		10. US EPA ID Number		E. State Transporter's ID		
				F. Transporter's Phone		
				G. State Facility's ID		
				H. Facility's ID		
GENERATOR	11. US DOT Description (Including Proper Shipping Name, Hazard Class, and ID Number)		12. Containers No.	13. Total Quantity	14. Unit Wt/vol	15. Waste No.
	a.					
	b.					
	c.					
	d.					
J. Additional Descriptions for Materials Listed Above			K. Handling Codes for Wastes Listed Above			
15. Special Handling Instructions and Additional Information						
16. GENERATOR'S CERTIFICATION: I hereby declare that the contents of this consignment are fully and accurately described above by proper shipping name and are classified, packed, marked, and labeled, and are in all respects in proper condition for transport by highway according to applicable international and national government regulations. Unless I am a small quantity generator who has been exempted by statute or regulation from the duty to make a waste minimization certification under Section 3002(b) of RCRA, I also certify that I have a program in place to reduce the volume and toxicity of waste generated to the degree I have determined to be economically practicable and I have selected the method of treatment, storage, or disposal currently available to me which minimizes the present and future threat to human health and the environment.						
Printed/Typed Name			Signature		Month Day Year	
TRANSPORTER	17. Transporter 1 Acknowledgement of Receipt of Materials					
	Printed/Typed Name			Signature		Month Day Year
	18. Transporter 2 Acknowledgement of Receipt of Materials					
Printed/Typed Name			Signature		Month Day Year	
FACILITY	19. Discrepancy Indication Space					
	20. Facility Owner or Operator: Certification of receipt of hazardous materials covered by this manifest except as noted in item 19.					
Printed/Typed Name			Signature		Month Day Year	

86416635

DHS 8022 A (11/85)
(EPA 8700-22)

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To P.O. Box 3000, Sacramento CA 95812

FIGURE 9.6 Hazardous waste manifest (4).

The NPL is a guide to the EPA in determining which sites warrant further investigation to assess the nature and extent of public health and environmental risks and to decide what CERCLA-financed remedial action(s), if any, may be appropriate. The NPL list of releases and highest priority facilities is revised at least annually.

Two sections make up the NPL: the General Superfund Section covers sites that are evaluated and cleaned up by EPA, and the Federal Facilities Section addresses sites that generally are evaluated and cleaned up by other federal agencies. Each federal agency is responsible for carrying out most response actions at facilities under its own jurisdiction, custody, or control, although the EPA is responsible for preparing a Hazard Ranking System (HRS) score and determining whether the facility is placed on the NPL.

Three mechanisms are used for placing sites on the NPL for possible remedial action:

1. A site may be included on the NPL if it scores sufficiently high on the HRS. The HRS serves as a screening device to evaluate the relative potential of substances posing a threat to human health or the environment. Four pathways—groundwater, surface water, soil exposure, and air—are evaluated.
2. Each state may designate a single site as its top priority to be listed on the NPL, regardless of the HRS score. This mechanism requires that, to the extent practicable, the NPL include within the 100 highest priorities one facility designated by each state representing the greatest danger to public health, welfare, or the environment among known facilities in the state.
3. The third mechanism for listing allows certain sites to be listed, regardless of their HRS score, if all of the following conditions are met:
 - The Agency for Toxic Substances and Disease Registry (ATSDR) of the U.S. Public Health Service has issued a health advisory that recommends dissociation of individuals from the release.
 - The EPA determines that the release poses a significant threat to public health.
 - The EPA anticipates that it will be more cost-effective to use its remedial authority than to use its removal authority to respond to the release.

A site may undergo remedial action financed by the Trust Fund, commonly referred to as the Superfund, established under CERCLA, only after it is placed on the NPL. However, placing a site on the NPL does not imply that monies will be expended. The EPA may pursue other appropriate authorities to remedy the releases, including enforcement action under CERCLA and other laws.

The NPL does not describe releases or facilities in precise geographical terms. Although a CERCLA facility is broadly defined to include any area where a hazardous substance release has come to be found, the listing process itself is not intended to define or reflect the boundaries of such facilities or releases. If the HRS is used to list a site, the data will, to some extent, describe the release(s) at issue.

When a site is listed, the approach generally used is to delineate a geographical area and to identify the site by reference to that area. The site is not coextensive with that area, as the site boundary encompasses all contaminated areas within the area used to identify the site, and any other location in which that contamination has come to be found, or from which that contamination has migrated.

As more information is developed on site contamination, the extent of the threat presented by a release will be determined by a Remedial Investigation/Feasibility Study (RI/FS). During the RI/FS process, more is learned about the source(s) and the migration of the contamination. Generally, it is impossible to discover the full extent of where the contamination has come to be located before all necessary studies and remedial work are completed. In fact, the boundaries of the contamination can often be expected to change over time. For these reasons, the NPL need not be amended as further research reveals more information about the location of the contamination or release.

Where no further response is appropriate under Superfund, the EPA may delete sites from the NPL. Regulations provide that the EPA will consult with states on proposed deletions and will consider whether any of the following criteria have been met:

- Responsible parties or other persons have implemented all appropriate response actions required,
- All appropriate Superfund-financed response has been implemented and no further response action is required; or
- The remedial investigation has shown the release poses no significant threat to public health or the environment, and taking of remedial measures is not appropriate.

Since a total site cleanup may take many years, while portions of the site may have been cleaned up and made available for productive use, the EPA has authority to delete portions of a NPL site. The NPL Construction Completion List (CCL) was developed by the EPA to simplify its system of categorizing sites and to better communicate the successful completion of cleanup activities. Inclusion of a site on the CCL has no legal significance. Sites qualify for the CCL when:

- Any necessary physical construction is complete, whether or not final cleanup levels or other requirements have been achieved;
- The EPA has determined that the response action should be limited to measures that do not involve construction (e.g., institutional controls); or
- The site qualifies for deletion from the NPL.

WASTE TREATMENT

Many of the treatment processes discussed in previous chapters are applicable to treatment of liquid, gaseous, and solid hazardous wastes. This section focuses on the applications and limitations of these processes to hazardous waste treatment. Land disposal, solidification, and stabilization for liquid and solid wastes and thermal treatment for all types of wastes are presented separately.

Liquid Waste Treatment

Since liquid hazardous wastes vary widely with respect to type and concentration, source (e.g., process and site cleanup), volume, and solids content, numerous treatment processes have application. Normally, multiple processes are used in combination for pretreatment and/or final treatment. The four general approaches to liquid treatment are

1. On-site treatment system
2. On-site pretreatment with discharge to existing wastewater treatment facility
3. On-site treatment with a mobile unit (applies to site cleanup)
4. Off-site treatment via waste hauling (limited by volume)

The liquid waste treatment methods most commonly applied and discussed in this section are

Activated carbon	Ion exchange
Air stripping	Neutralization
Biological processes	Oxidation–reduction
Chemical precipitation	Reverse osmosis
Filtration	Sorption resins
Gravity separation	

Activated Carbon. Using the technology developed for water and wastewater treatment (38), activated carbon is well suited for removal of hazardous organics, such as aromatics, halocarbons, pesticides, phenols, and

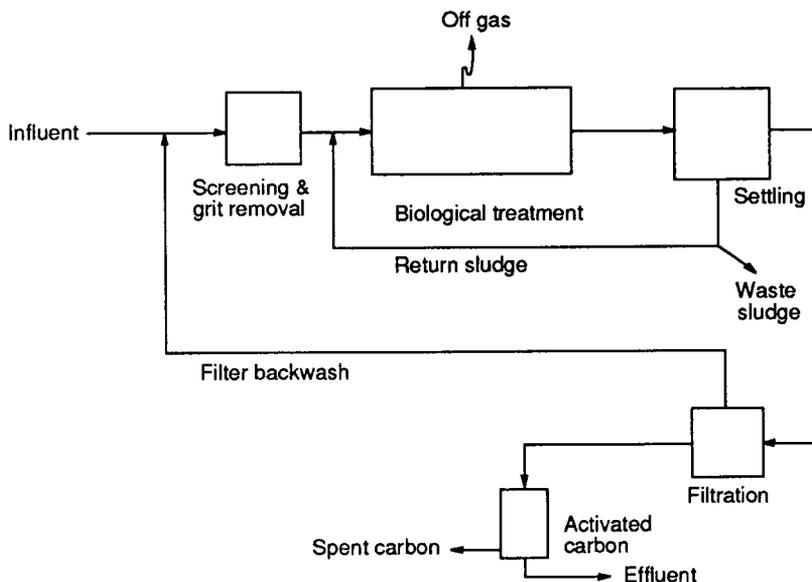


FIGURE 9.7 Typical activated-carbon process application.

phthalates. Also, some inorganics have shown good to excellent adsorption potential; these include antimony, arsenic, bismuth, bromine, chlorine, chromium, cobalt, iodine, mercury, tin, silver, and zirconium (38).

Pretreatment is necessary to secure carbon column effectiveness and to minimize regeneration time and expense. Common methods include air stripping or biological treatment for gross reduction of organics, filtration for suspended solids control, and gravity separation for protection from oils and grease and/or suspended solids. Figure 9.7 illustrates the process flow diagram of a facility using biological treatment and filtration before an activated carbon system.

A typical activated-carbon system for hazardous waste will use granular activated carbon in a series of downflow reactor vessels. Figure 9.8 shows a two-vessel system; on-site or mobile systems are also used. A hydraulic loading rate of 200 gal/mm (12.5 L/s) with a contact time of 27 mm for a three-column system has proven adequate for many waste streams (38).

Spent carbon may be regenerated and reused until performance declines suggest the need for some blow-down and replenishment with fresh carbon. Air pollution controls are generally needed in association with spent-carbon handling and thermal regeneration.

Air Stripping. In an air stripping system, a contaminated water stream is mixed with clean air, and intimate contact causes a mass transfer process by which volatile contaminants are transferred to gas. Air stripping is categorized as putting air through water or putting water through air. In either case, treatability studies are advised for developing design criteria.

Aeration devices that put air through water are illustrated in Figure 9.9 (page 9.33). The mass transfer occurs at the bubble surface, and the practical upper limit for the air-to-water (volume) ratio is about 20 to 1 (40). It can be expected that this category of air stripping will be a secondary benefit from aeration systems at new or existing wastewater treatment works.

A designated air stripping system is most frequently a water-through-air system; for removal of volatile organics, countercurrent packed towers are most common. These systems create water droplets or a water film to facilitate mass transfer, using devices like those illustrated in Figure 9.10. Although air-to-water

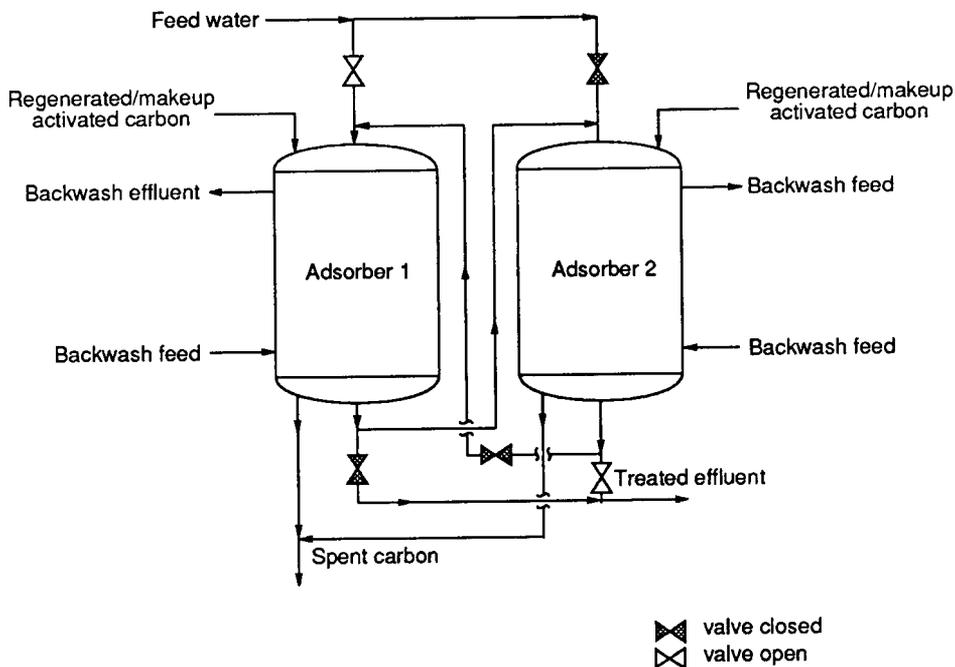


FIGURE 9.8 Two-vessel granular activated-carbon system (38).

(volume) ratios as high as 3000 to 1 have been reported (41), the ratio is generally less than 100 to 1 (40, 42).

A typical packed tower will be 3 to 10 ft (1 to 3 m) in diameter, 15 to 30 ft (5 to 10 m) in height, and packed with ceramic, glass, or plastic media. Towers are generally readily available, and mobile units can be obtained. There are numerous references available on the theory, design, and performance of packed towers (43–46).

Air stripping systems are often used (for load reduction) prior to activated carbon or biological systems to achieve desired treatment levels. Also, since volatile organics are being released to the atmosphere, the designer should consider ambient air quality control requirements.

Biological Processes. In most applications, aerobic biological treatment processes are used for removal of hazardous organic matter. A summary of applications and limitations of activated sludge and other biological processes is presented in Table 9.10.

Proven process technology and microorganism adaptability offer a wide breadth of treatment capability. Even though most organic compounds are considered biodegradable, the practical treatability varies considerably. Several generalizations follow with regard to the ease of treatability by aerobic biological treatment processes (39).

- Unsubstituted nonaromatics or cyclic hydrocarbons are preferred over unsubstituted aromatics.
- Materials with unsaturated bonds, such as alkenes, are preferred over materials with saturated bonds.
- Soluble organics are usually more readily degraded than insoluble materials. Biological treatment is more efficient in removing dissolved or colloidal materials, which are more readily attacked by enzymes. This is not the case, however, for fixed-film treatment systems, which preferentially treat suspended matter.

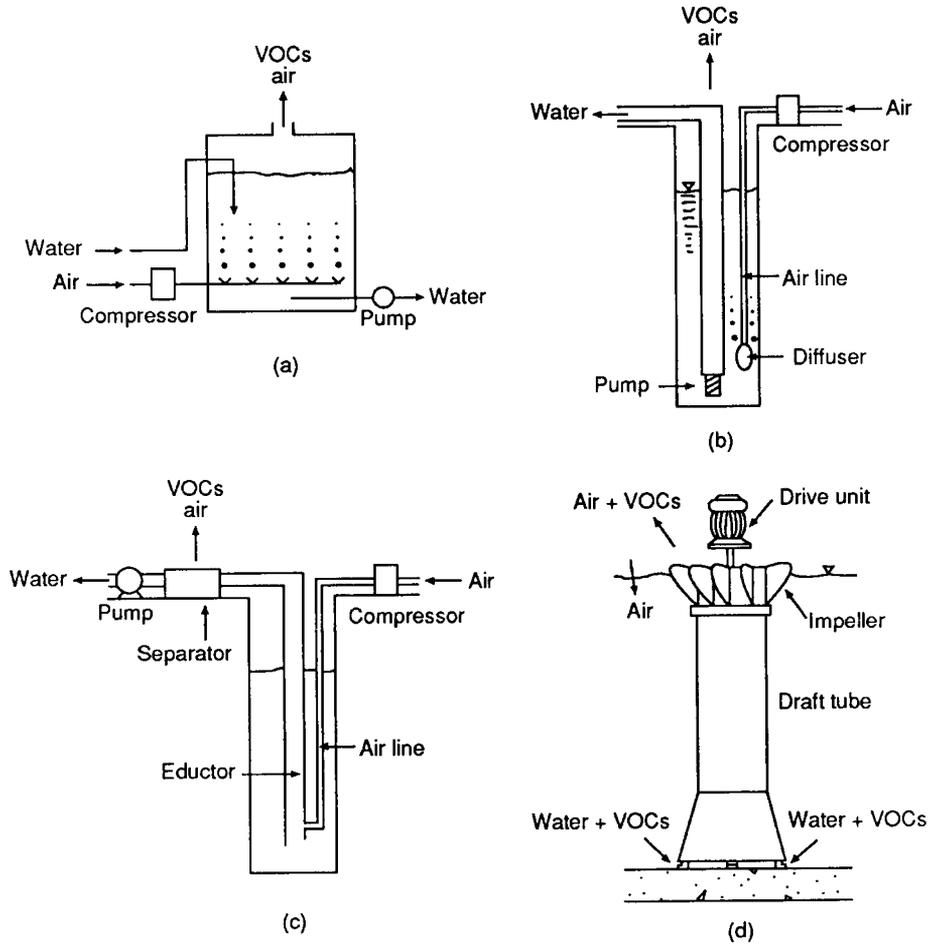


FIGURE 9.9 Air through water stripping devices (a) Diffused-air aerator; (b) in-ground aeration; (c) air lift pump; (d) mechanical surface aerator (40).

TABLE 9.10 Applications and Limitations for Biological Treatment Processes (39)

Process	Applications and limitations
Conventional	Applicable to low-strength wastes; subject to shock loads
Completely mixed conventional	Resistant to shock loads
Extended aeration	Requires low organic load and long detention times; low volume of sludge; available as package plant
Contact stabilization	Not suitable for soluble BOD
Pure oxygen	Suitable for high-strength wastes; low sludge volume; reduced aeration tank volume
Trickling filters	More effective for removal of colloidal and suspended BOD; used primarily as a roughing filter
Rotating biological disk	Can handle large flow variations and high organic shock loads; modular construction provides flexibility to meet increases or decreases in treatment needs

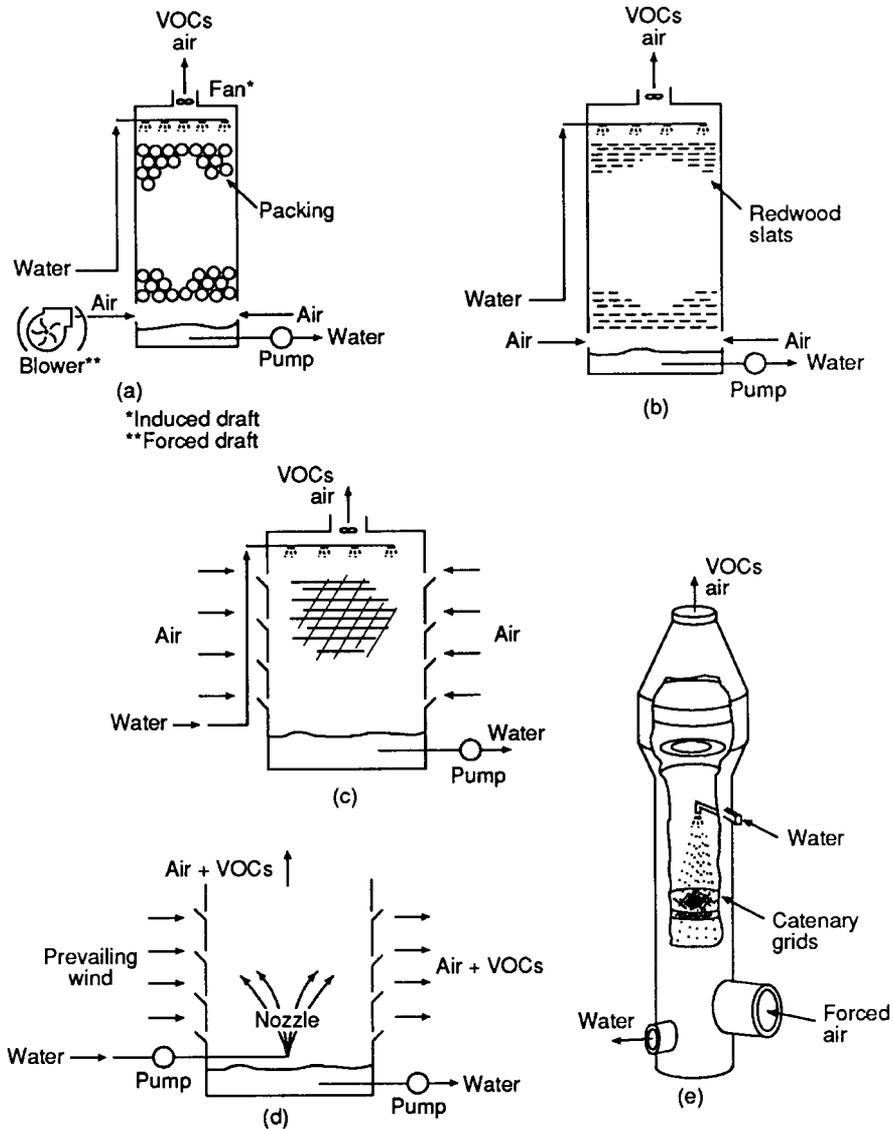


FIGURE 9.10 Water-through-air stripping devices. (a) Packed tower aerator; (b) redwood slat aerator; (c) cooling tower; (d) spray tower; (e) catenary grid scrubber (40).

- The presence of functional groups affects biodegradability. Alcohols, aldehydes, acids, esters, amides, and amino acids are more degradable than corresponding alkanes, olefins, ketones, dicarboxylic acids, nitriles, and chloroalkanes.
- Halogen-substituted compounds are the most refractory to biodegradation; chlorinated aliphatics are generally more refractory than the corresponding aromatics, although the number of halogens and their position is also significant in determining degradation.
- Nitro-substituted compounds are also difficult to degrade, although they are generally less refractory than the halogen-substituted compounds.

The biological treatment process will produce a sludge requiring additional treatment and disposal. Also, the designer must be alert to possible excessive gaseous emissions from the selected aeration process.

Chemical Precipitation. Independently or in combination with oxidation–reduction reactions, chemical precipitation and sedimentation are widely practiced for the removal of metals. Common precipitation reagents are lime, caustic, and sulfide; these advantages and disadvantages are summarized in Table 9.11. Also, calcium and sodium carbonate are used for metals where solubility within a pH range is not sufficient to meet a specific treatment standard (47–49).

Precipitation has many useful applications to hazardous waste treatment, but laboratory jar tests should be used to verify the treatment method. Figure 9.11 illustrates the solubility of several metal hydroxides and sulfides at various pH conditions. Some organic and inorganic substances combine with metals to inhibit settling characteristics. The jar test is also used to select the appropriate chemical; determine dosage rates; assess mixing, flocculation, and settling characteristics; and estimate sludge production and handling requirements.

Filtration. When removal of low levels of suspended solids is required, filtration is a proven, reliable, and effective technology. In particular, granular media filters have demonstrated performance as part of mobile treatment systems as well as on-site systems for hazardous wastewater treatment. A significant design consideration is adequate treatment of filter backwash water which will contain high concentrations of contaminants. Also, the turbulence during backwashing may release volatile emissions, which normally are not a problem.

The EPA has had a successful experience, over a relatively long term, using a physical/chemical system for hazardous waste treatment. Three sand–anthracite medium filters were operated in parallel with a designed, maximum hydraulic loading of 7 gal/min/ft² (285 L/min/m²) (49).

Sedimentation is commonly applied technology for removal of high suspended solid loads. The waste stream may originate from another treatment process, surface runoff, leachate, or dredge slurries. The process may be a batch or continuous operation using impoundments or conventional or high-rated settling basins. Laboratory settling tests are useful in predicting system performance.

For two-phase liquid waste streams, such as mixtures of water with gasoline or oils, gravity separators are used. Chemical additions are necessary if emulsions are present. The greater the difference in water and waste specific gravity, the more effective gravity separation will be.

TABLE 9.11 Applicability of Precipitation Reagents (47)

Lime, Ca(OH) ₂	Caustic, NaOH	Sulfide, NaHS and FeS
Least-expensive reagent	More expensive than lime	Effective treatment at lower concentrations
Generates largest sludge volume	Generates smaller hydroxide sludge volume	Sludges suitable for reclamation
Sludges generally not desirable for reclamation	Sludges suitable for reclamation	

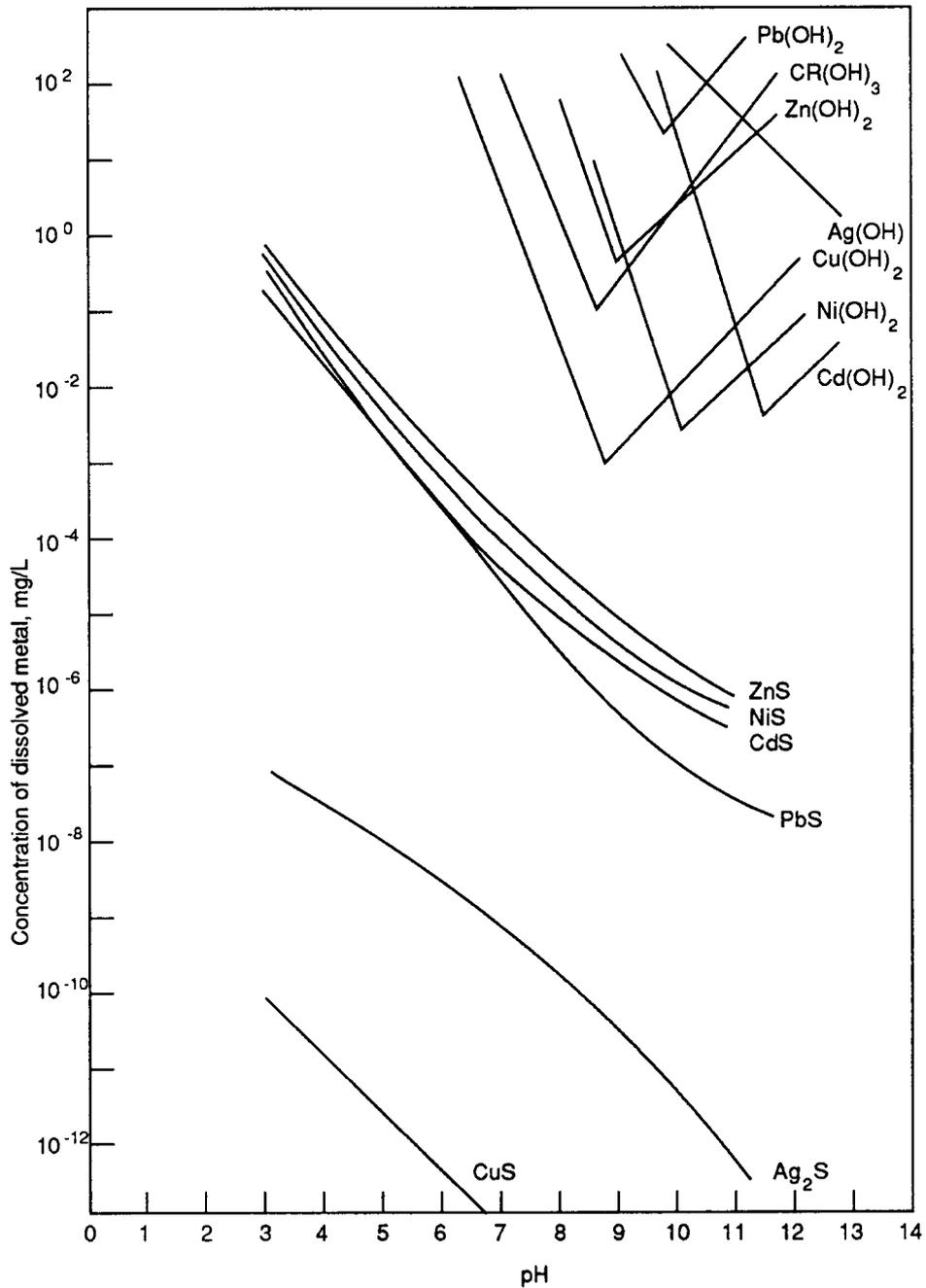


FIGURE 9.11 Solubility of metal hydroxides and sulfides (49).

When water- and waste-specific gravities are only slightly different, dissolved air flotation should be considered. Also, air flotation systems are an alternate for alleviating overloading of sedimentation systems and for quick response for on-site treatment of a spill or other remedial action.

Ion Exchange. The technology for removal of heavy metals (50) and hazardous anions from dilute solutions by ion exchange is well established. Availability and ease of operation make ion exchange units a desirable treatment method for mobile units.

The ion exchange system design must be sensitive to the specific ions to be exchanged, the control of influent suspended solids, and the treatment of regeneration solutions. Also, the mode of operation, manual or automatic, is an important decision.

Ion exchange resins are classified as cation (strong and weak acid) and anion (strong and weak base) exchanges. In order of decreasing preference, Table 9.12 shows the selectivities of strong acid and strong base ion exchange resins for various ionic compounds.

Neutralization. If a waste stream is determined to be hazardous because of corrosivity, neutralization is the only treatment required. More frequently, neutralization is used as a pretreatment system before a variety of biological, chemical, and physical treatment processes.

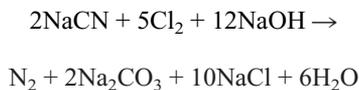
Oxidation–Reduction. Chemical oxidation–reduction reactions are used to reduce toxicity or solubility or to transform a substance to one more easily handled. However, the process designer must take special precautions with the oxidizing and reducing treatment chemicals, which are usually themselves hazardous. Also, the potential of producing undesirable by-products should be investigated during process selection.

Oxidation Process. Chemical oxidation is primarily used for treatment of cyanide and of dilute solutions containing oxidizable organics. Chlorine, calcium, or sodium hypochlorites, chlorine dioxide, ozone, hydrogen peroxide, ozone, and potassium permanganate are common oxidizing chemicals.

A widely used oxidation treatment scheme is the alkaline chlorination of cyanide wastes, using either batch or continuous treatment systems. The process uses caustic to raise the pH to levels of 10.5 to 11 to provide conditions to complete oxidation of the cyanide (and to provide an added safety margin against the development of acidic conditions, which would emit cyanide gas compounds). With the addition of chlorine, the reaction is expressed as:

TABLE 9.12 Selectivity of Ion Exchange Resins (50)

Strong acid cation exchanger	Strong base anion exchanger
Barium	Iodide
Lead	Nitrate
Calcium	Bisulfite
Nickel	Chloride
Cadmium	Cyanide
Copper	Bicarbonate
Zinc	Hydroxide
Magnesium	Fluoride
Potassium	Sulfate
Ammonia	
Sodium	
Hydrogen	



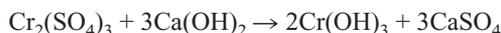
Ozonation has application in oxidation of cyanide as well as other organics, such as alcohols, pesticides, and phenols. Ozone (like chlorine) systems avoid the potential problems of managing hazardous treatment chemicals at the facility; however, air quality issues are the trade-off.

Reduction Process. Chemical reduction is primarily used for treatment of wastes containing hexavalent chromium, mercury, and lead. Common reducing agents include sulfur dioxide, sulfite salts (such as sodium metabisulfite), and ferrous sulfate.

Reduction and precipitation of chromium wastes involves reducing hexavalent chromium to the trivalent state under acidic conditions ($\text{pH} \leq 3$) followed by a precipitation under basic conditions (pH of 8 to 9). Using sulfur dioxide, the reduction reaction of dichromic acid is



Raising the pH with lime, the precipitation reaction is



Reverse Osmosis. The use of reverse osmosis to reduce concentrations of dissolved organic and inorganic solids is limited. In hazardous waste treatment, reverse osmosis is an alternative for removal of highly toxic contaminants (such as metals and pesticides) from low-volume waste streams. Extensive pretreatment is necessary to protect the unit from excessive suspended solids loads, extreme pH values, oils and grease, and other wastewater characteristics that may impede operation or even damage or dissolve the membrane.

The brine stream will contain concentrated pollutants requiring additional treatment. Also, precautions must be taken with solutions used to wash and maintain the membrane and with the disposal of degraded or fouled membranes.

Sorptive Resins. On a case-by-case basis, sorptive resins are used for removal of a wide range of polar and nonpolar organics (49). A good application is in response to a spill or other short-term treatment requirement.

Gaseous Waste Treatment

Absorption, adsorption, and flaring are three primary methods of gaseous waste treatment. Thermal destruction is a fourth and is discussed in a subsequent section with respect to liquid, gas, and solid waste streams.

Absorption. Generally, absorption systems pass gases through water and are most widely used for control of inorganic emissions. The variety of approaches available necessitates case-by-case evaluation (51).

Adsorption. Granular activated carbon adsorption is generally accepted for use by industry for air pollution and odor control. The system is most effective in removal of organic emissions. Many systems are commercially available and suitable for site adaptation by the designer (39, 51).

Flaring. Flares use open flames during normal and/or emergency operations to combust hazardous gases. The system has no special features to control temperature or time of combustion; however, supplemental fuel may be required to sustain the combustion (39, 51, 52). Historically, flares have been used to dis-

pose of waste gases in the oil and gas industry and at wastewater treatment plants having anaerobic digesters.

Regulations for thermal destruction of hazardous wastes limit the practical use of flaring to combustion of relatively simple hydrocarbons, such as methane from digesters or landfill gas collection systems.

Solid Waste Treatment

Solid hazardous wastes include contaminated soils and sediments produced during remedial actions (53–55). Soil slurries and dredged sediments require separation or classification of solids from a waste stream and dewatering of waste streams containing a high solids content. Solid separation in liquid waste treatment and the associated sludge dewatering may also use this treatment technology. Industrial processes and wastewater treatment systems provide a proven data base.

Solids Separation. The methods used to separate and/or classify solids are waste- and site-specific. The separation process is used to concentrate solids and, thus, reduce the volume of the waste stream requiring dewatering and treatment. In most cases, the other more liquid stream requires treatment for suspended and/or dissolved contaminants.

Furthermore, the waste stream particles may be classified by grain size to take advantage of separation efficiencies of equipment and land-area-intensive methods. Also, since many contaminants may be adsorbed on fine-grained materials (i.e., clays and organic matter), the coarse-grained portion of the waste stream may be returned to the environment or treated as a nonhazardous waste. The feasibility of classification controls requires case-by-case investigation and treatability analysis.

The most appropriate solids separation method for a given site depends upon several factors, including (39)

- Volume of contaminated soil or sediment
- Characteristics of the contaminant(s)
- Composition of soils or sediment, including particle sizes (Table 9.13), percent clays, and percent total solids
- Type of excavation or dredging equipment, which determines the percent solids or feed rate, respectively, to solids separation
- Site location and surroundings, including available land area and the present and projected land use

TABLE 9.13 Particle Sizes for Various Soil Categories (39)

USCS classification	U.S. standard sieve size	Particle size, μm
Gravel		
Coarse	$>3/4$ in	
Fine	No. 4– $3/4$ in	
Sand		
Coarse	No. 10–No. 4	
Medium	No. 40–No. 10	
Fine	No. 200–No. 40	
Silt		10–74
Coarse clay		1.0–10
Fine clay		< 1.0

TABLE 9.14 Vibrating Screen Design Data (39)

Parameter	Typical range
Material size range	0.12–6 in (0.32–15 cm)
Incline to horizontal	0°–20°
Width	3–10 ft (1–3 m)
Length	6–30 ft (2–10 m)
Solids loading	300–950 tons/h (275–850 t/h)

Moving Screens. Of the types available (vibrating, revolving, and gyratory), vibrating screens are the most widely used moving screens. Vibrating screens function by using one or more screening decks to allow progressively finer separation through the screen. The oversized particles retained on the screen are substantially dewatered during the process. Units are commercially available for use on site or with a mobile system. Table 9.14 contains typical design information.

Conventional vibrating screens are best suited for separation of relatively dry solids. For high-moisture-content soils or dredge material, sprays are needed to minimize blinding of the screen. Another limitation is the need to avoid wastes containing abrasive material, as this may significantly reduce the useful life of the screen.

Static Screens. One type of static (or stationary) screen applicable for solids separation at hazardous waste sites is the wedge bar screen, which consists of parallel bars mounted on a curved deck. Smaller particles pass through with the liquid, and (dewatered) oversized particles are collected off the top of the screen. Wedge bar screens are normally in widths of 2 to 6 ft (0.6 to 1.8 m) with loading rates of 30 to 200 gal/min/ft² (1750 to 11,750 m³/m²/day).

Since the oversized particles tend to drag out some of the finer particles, a water spray may be used to wash off fines from the coarser solids and to break up accumulations of solids that might blind the screen. This modification is known as hydrosieve or pressure screen.

Wedge bar and hydrosieve units are easier to operate and maintain and are more resistant to abrasive solids than moving screens but are not as efficient a means of solids separation. The lower cost of static screens makes them desirable when high removal efficiencies are not necessary.

Hydraulic Classifiers. Separation and classification of particles ranging in size from fine gravel to fine sands may be effectively accomplished using hydraulic classifiers. These units are best suited for smaller volumes of flow and/or for coarse solids separation before other systems with better capability to remove fines. Hydraulic classifiers do offer the advantage of being truck-mounted for mobile and quick-response applications.

Spiral Classifiers. The spiral classifier consists of a tub-type arrangement with one or two long, shafted screws pulling solids up an incline, while flushing water is added near the top of the incline to wash clay and fine solids from sand and gravel particles. Typical classifiers are limited in application to sand and gravels separation. Some dewatering and classification are achieved. Mobile units are available.

Cyclones. Using centrifugal force, cyclones are used for separation and classification of particles ranging in size from 10 to 2000 μm . Cyclones require that feed solids concentration not be too high and oversized particles be avoided. Thus, pretreatment may be required. The advantages of cyclones include good removal of smaller particle sizes, distinct particle size separation, mobility, and ease of operation.

Impoundments. With or without chemical conditioning, impoundments are used to remove a broad range of particle sizes when adequate land is available. Liners are typically necessary to protect underlying groundwater. Adjustable effluent weirs are used to control flow through the impoundment basin.

Multiple earthen basins are advisable to permit dredging of accumulated solids from one basin while others remain in service. Solids may be classified by using multiple basins in series. If there is a significant

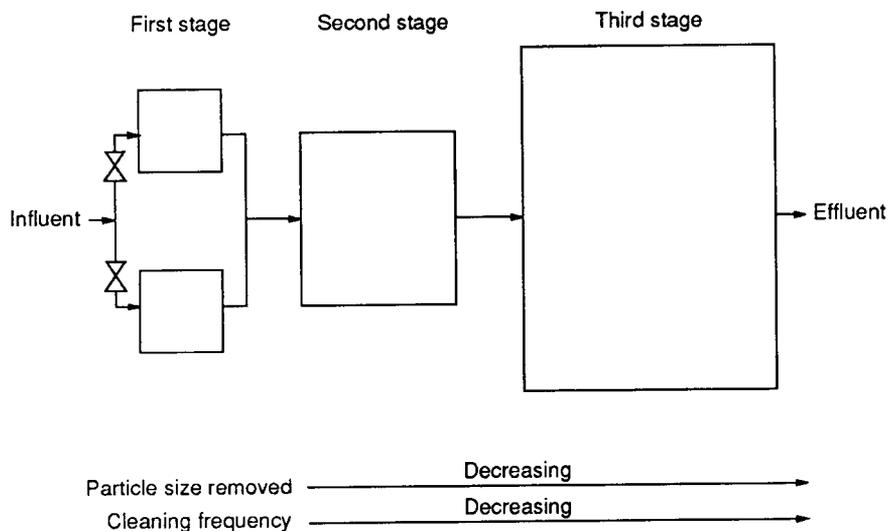


FIGURE 9.12 Multiple impoundment settling basin system.

sand and gravel content in the waste stream, a relatively small basin should be considered as the first treatment unit; such a basin is more practical and cost effective to clean and preserves capacity in the larger impoundments.

A multiple-basin scheme, wherein the second- and third-stage basins remain in service during cleaning, is illustrated in Figure 9.12. In this approach, parallel first-stage basins are used. In other cases, a floating curtain wall or other means may be necessary to avoid excessive solids carryover due to cleaning activities.

Clarifiers. Conventional and high-rate clarifiers with solids collection mechanisms are well suited for small- to medium-scale remedial action projects. Clarifier technology, with and without chemical aids, is well established.

Dewatering. The objectives of dewatering processes include reduction of moisture content for an associated decrease in volume and reduction of costs for subsequent treatment and disposal. The established technology base for wastewater sludge dewatering has application for hazardous waste streams containing contaminated soils or sediments (55).

Numerous mechanical and evaporation/percolation dewatering processes are available. The most prevalent processes are

- Centrifuges: basket and solid bowl
- Filtration: vacuum, belt, and pressure
- Drying beds
- Lagoons

Typical ranges of solids capture for these processes are presented in Table 9.15. Key operational variables are listed in Table 9.16.

Other variables affecting the selection of dewatering process alternatives include

TABLE 9.15 Typical Solids Capture of Dewatering Processes (55)

Process	Typical solids capture, %
Basket centrifuge	80–98
Solid bowl centrifuge	90–98
Belt filter press	85–95
Vacuum filter	88–95
Filter press	> 98
Drying beds	> 99
Sludge lagoons	> 99
Gravity and low-pressure devices	88–95

Note: Solids captures shown are for properly operated dewatering systems with well-conditioned sludge. With improper operation, solids capture as low as 50% has been noted for some processes.

TABLE 9.16 Operational Variables for Dewatering Processes (55)

1. Basket centrifuge	5. Conventional filter press
a. Bowl speed	a. Pressure of feed sludge
b. Time at full speed	b. Filtration time
c. Depth of skimming	c. Use of precoat
d. Sludge feed rate	d. Conditioning chemicals—type and dosage
e. Polymer conditioner	e. Cloth-washing frequency
Dosage utilized	f. Filter cloth used
Point of addition	6. Diaphragm filter press
2. Solid bowl centrifuge	a. Pressure of feed sludge
a. Bowl or conveyor differential speed	b. Filtration time
b. Pool depth	c. Diaphragm pressure
c. Sludge feed rate	d. Diaphragm squeezing time
d. Polymer conditioner	e. Conditioning chemicals
Dosage utilized	Point of addition
Point of addition	Type and dosage
3. Belt filter press	f. Filter cloth used
a. Belt speed	g. Cloth-washing frequency
b. Belt tension	7. Drying beds
c. Washwater flow and pressure	a. Depth of sludge application
d. Belt type	b. Conditioning of sludge
e. Sludge feed rate	c. Duration of drying time
f. Polymer conditioner	d. Method of sludge cake removal
Dosage utilized	8. Sludge lagoons
Point of addition,	a. Frequency of sludge addition
contact time, mixing	b. Method of sludge removal
4. Vacuum filter	c. Method of supernating
a. Quantity of wash H ₂ O used	
b. Drum speed	
c. Vacuum level	
d. Conditioning chemicals—type and dosage	
e. Drum submergence	
f. Vat agitation	
g. Filter medium used	

- Volume of the slurry
- Solids content
- Volatile (organic) solids content
- Particle size
- Particle surface charge and hydration
- Compressibility
- Site conditions, including land availability
- Compatibility with existing and/or planned facilities
- Pretreatment required, especially dosages of conditioning chemicals
- Requirements established by final treatment and disposal alternative(s) (Table 9.17)
- Extent of additional treatment required for the by-product liquid waste stream
- Unit mobility and expected period of performance

Laboratory analyses and treatability studies are used during the selection process to better quantify predicted performance.

Land Disposal

Liquid and/or solid hazardous wastes may be disposed of using the land via landfills, surface impoundments, land application, or underground (deep-well) injection. The EPA has published general environmental performance standards (Table 9.3), and specific standards for owners and operators of land disposal facilities (8, 9, 12).

Landfills. For many years, burial of hazardous wastes was the accepted, if not perhaps the standard, method of disposal. However, EPA regulations (8, 12) have significantly reduced the cost-effectiveness of hazardous waste landfills, especially for remedial actions and other on-site uses (126).

TABLE 9.17 Dewatering Process Compatibility with Subsequent Treatment or Disposal Methods (55)

Dewatering process	Incineration*	Composting	Agricultural land application	Landfill†
Basket centrifuge			X	X
Solid bowl centrifuge	X	X‡	X	X
Belt filter press	X	X‡	X	X
Vacuum filter	X	X‡		X
Filter press	X	X	X§	X
Drying bed		X	X§	X
Sludge lagoon			X	X

*Solids content required for self-sustaining combustion will vary depending upon the percent of solids that are organic and the calorific value of the organics.

†Some states and municipalities have rigid requirements on the solids content of sludges placed in landfills. Local regulations should be checked by the designer.

‡Suitability of this method depends on organic content of sludge. Thermodynamics of composting must be evaluated. Generally, sludges with 20% or greater solids content can be composted, depending on the degree of prior stabilization and weather conditions.

§Soil characteristics are important. For some alkaline soils (i.e., some calcareous soils), land application may not be desirable because of lime in the dewatered sludge cake. For soils with a high sodium content, however, addition of calcium can beneficially increase the calcium/sodium ratio and result in improved tilth. There are very few soils where a problem would be anticipated due to application of sludge cake. Advice of agriculturists is recommended.

For new landfills, new or replacement units at an existing facility, and each lateral expansion of an existing unit, the facility must include two or more liners and a leachate collection system above and between the liners. Normally, synthetic liners are used, but the lowest liner may be clay or similar material with a permeability of 1×10^{-7} cm/s or less. Other landfilling facility changes simply must provide a liner system and a leachate collection and removal system.

Liners must be constructed of materials that prevent the wastes and any by-products from passing into and through the liner. Also, the material must have sufficient strength and thickness (typically 30 mil minimum) to withstand installation and operation stresses as well as the influences of site conditions, including climate and geotechnical and hydrogeologic factors. Numerous reference documents are available to the designer (56–58).

Leachate collection lines should be within 1 ft (30 cm) of the liner. The leachate will require monitoring and treatment, using methods described under the heading of liquid waste treatment (8, 59).

Additional sources of hazardous waste landfill wastewaters requiring collection and treatment include gas collection condensate, truck/equipment washwater, drained free liquids, laboratory wastewaters, and contaminated stormwater. The EPA's new source performance standards (40CFR445) for treatment of leachate and other hazardous waste landfill wastewaters before discharge to surface waters are presented in Table 9.18. The effluent concentration limitations for discharge to publicly owned treatment works (POTW) from new source pretreatment facilities are presented in Table 9.19 (8, 58, 127–130).

Other features of an acceptable hazardous waste landfill include a run-on control system for flows to the active portion of the landfill during a 25-year storm and a runoff system to collect and control the water volume of at least a 24-h, 25-year storm. Also, air quality controls may be required to control wind dispersal of particulates and to manage gaseous releases from the landfill.

Surface Impoundment. Prior to modern design practices, many generators used surface impoundments, such as pits or ponds, to collect liquid and solid wastes. With time, the surface impoundment was filled in or simply abandoned in favor of a new site. The results of these historic practices are the enormous number of remedial action sites.

TABLE 9.18 Hazardous Leachate Treatment Limitations for Discharges to Surface Waters

Pollutant or pollutant property	Maximum for any one day	Monthly average
BOD ₅ , mg/L	160	40
TSS, mg/L	89	27
Ammonia, mg/L	5.9	2.5
Arsenic, mg/L	1.0	0.52
Chromium (total), mg/L	0.86	0.40
Zinc, mg/L	0.37	0.21
Alpha terpineol, mg/L	0.042	0.0 19
Aniline, mg/L	0.024	0.015
Benzene, mg/L	0.14	0.036
Benzoic acid, mg/L	0.12	0.073
Naphthalene, mg/L	0.059	0.022
<i>p</i> -Cresol, mg/L	0.024	0.015
Phenol, mg/L	0.048	0.029
Pyridine, mg/L	0.072	0.025
Toluene, mg/L	0.080	0.026
pH, units (range)	6.0–9.0	6.0–9.0

TABLE 9.19 Hazardous Leachate Pretreatment Limitations for Discharges to POTWs

Pollutant or pollutant property	Maximum for any one day	Monthly average
Ammonia, mg/L	5.9	2.5
Alpha terpineol, mg/L	0.042	0.019
Aniline, mg/L	0.024	0.015
Benzoic acid, mg/L	0.23	0.13
<i>p</i> -Cresol, mg/L	0.024	0.015
Toluene, mg/L	0.080	0.026

Under modern practices, surface impoundments are designed using two or more liners to prevent any waste migration to the subsurface soil or groundwater or to surface waters and a leachate collection system between each liner.

Synthetic liners must be capable of withstanding chemical deterioration from the waste constituents and of physical demands during construction and operation and from variable climatic conditions. Natural material may be used for the lowest liner if it is constructed of at least a 3-ft (1-m) layer of recompacted clay or other suitable material. The layer's permeability should be no more than 1×10^{-7} cm/s (8, 57).

Since a surface impoundment is intended for holding, it must be designed to avoid overtopping from normal operation or abnormal conditions, such as overfilling, wave action, rainfall, run-on, and equipment malfunction. Typically, a minimum freeboard of 2 ft (0.6 m) will be adequate. Earthen dikes must have an impervious core and have exterior protection by grassing, rip-rap, or other means to minimize wind and wastes erosion.

As a result of current requirements and criteria for monitoring and maintenance, retention of all wastes, and proper closure, surface impoundments are not often a cost-effective alternative for handling hazardous wastes.

Land Application. The design of a land application system for hazardous waste should be based on demonstrated success of the treatment zone to completely degrade, transform, or otherwise immobilize each waste constituent (8). Also, unsaturated zone monitoring should be included and other controls, such as runoff and runoff, must be incorporated.

Unless more stringent criteria are required at a specific site, the maximum depth of the treatment zone must be (8)

- No more than 5 ft (1.5 m) from the soil surface
- More than 3 ft (1 m) above the seasonal high-water table

Physical features of the land application system are similar to those for municipal wastewaters (60).

Liquid and solid or semisolid wastes may be applied onto the land surface or into the treatment zone. Chemical reagents may also be applied to aid in the treatment process.

Organic wastes and metals immobilized by the local soil are amenable to land application treatment and disposal. In addition to typical hazardous waste characteristics, land application system design must consider health and environment protection controls of nitrogen, phosphorus, dissolved solids, trace elements, and trace organics. Also, some pretreatment or conditioning is typically necessary.

Deep-Well Injection. Underground or deep-well injection is a disposal method for liquid hazardous waste. The wells are constructed into geological formations beneath and isolated from groundwater drinking supplies. The formation must be determined to have sufficient volume and porosity to accept the wastes for per-

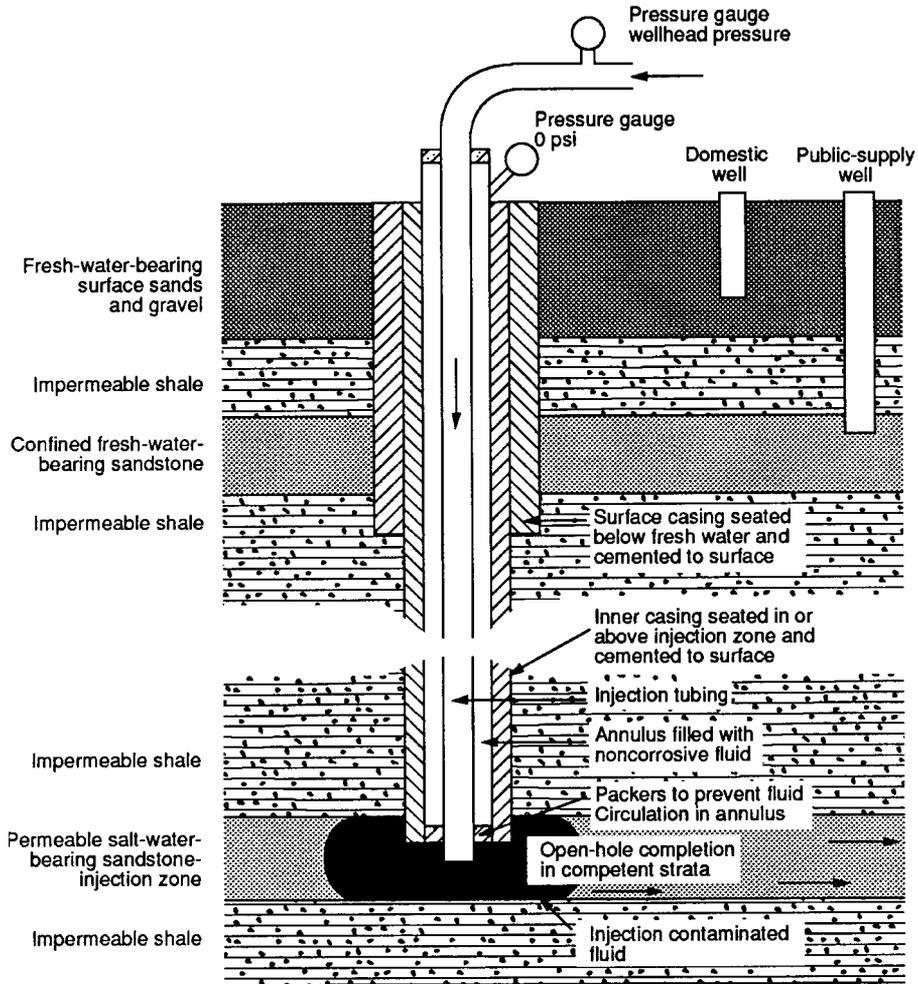


FIGURE 9.13 Features of a deep-well injection system (61).

manent containment. In addition, the design considers the volume and chemical and physical characteristics of the waste and injection pressure. Figure 9.13 illustrates the physical features of an injection well.

Wastes that are viscous or high in suspended solids will foul the injection process and should be avoided. Suitable wastes include concentrated solutions of acids, bases, heavy metals, pesticides, or other wastes that are hard to dispose of because of their high strength. On the other hand, injection wells can be used for disposal of large volumes of dilute wastes, such as contaminated surface waters and groundwater at a remedial action site.

Although technology is available to design, construct, and operate injection wells that satisfy mechanical integrity criteria, technology for determining mechanical integrity must be tested and proven to ensure that sound decisions are made and that groundwater drinking supplies are protected. A combination of tests is required in this process (62).

Solidification and Stabilization

Singularly or in combination, solidification and stabilization methods are viable on-site and off-site treatment alternatives, which accomplish one or more of the following objectives for liquid and solid hazardous wastes (39, 63, 64):

- Improve waste handling or other physical characteristics
- Decrease active surface area for contaminant transfer or loss
- Limit contaminant solubility or toxicity

In the solidification process, waste solids are mechanically held within a solidified matrix. As such, there is little effect on the chemistry of the wastes, and leaching may be a future problem. Stabilization is a chemical process that fixes the waste in a matrix for a decrease in solubility and/or toxicity.

More common solidification and stabilization methods are categorized by the following processes:

- Cement
- Pozzolanic
- Thermoplastic
- Organic polymer

Advantages and disadvantages of these processes are shown in Table 9.20. Other processes include surface encapsulation, self-cementing, and vitrification (39, 63). Table 9.21 illustrates the compatibility of selected waste categories with these solidification and stabilization processes.

Cement Processes. Solidification using Portland cement involves the direct mixing of the waste with the cement to form a hardened matrix. Most processes use Type I Portland cement, which is the typical cement used in the building industry. To a lesser extent, Types II and V are used when the waste contains moderate (150 to 1500 mg/kg) and high (≥ 1500 mg/kg) sulfate concentrations, respectively (63).

Most wastes are not chemically bound by this process and, thus, are subject to leaching. Even heavy metals, which are generally insoluble at the higher pH of a cement mix, may be solubilized and leached by mildly acidic leaching solutions. Additives and coatings are used to reduce leaching potential.

Pozzolanic Processes. Hazardous waste solidification based on lime or lime products usually depends on the reaction of fine-grained siliceous material together with lime and water. This process is referred to not only as the pozzolanic process but also as either a silicate-based or lime-based process.

Use of pozzolanic processes offers many combinations of materials, and a growing number of additives, for evaluation in the laboratory and on site. Soluble silicates, such as sodium and potassium silicate, are more desirable, but industrial by-products, such as fly ash, ground blast furnace slag, and cement kiln dust, may be used. Lime and Portland cement are generally used as the setting agents; however, gypsum, calcium carbonate, and other compounds may be used.

Thermoplastic Processes. Sealing wastes in a matrix, such as asphalt bitumen, paraffin, or polyethylene, is a thermoplastic (physical) process that occurs when the waste is dried, heated, and dispersed in a heated plastic medium. After cooling, the mixture is a rigid but deformable solid. The common range of heating temperatures is 265 to 450°F (130 to 230°C).

Organic Polymer Processes. Urea–formaldehyde and several specialty organic polymers are mixed with the waste. This process physically traps the waste by sealing it in a sponge-like polymer matrix. Normally, the mass is dried before disposal.

TABLE 9.20 Comparison for Solidification and Stabilization Processes (39, 63)

Process	Description	Advantages	Disadvantages
Cement	Slurry of wastes and water is mixed with Portland cement to form a solid	Low costs; readily available mixing equipment; relatively simple process; suitable for use with metals	Solids are suspended, not chemically bound; therefore are subject to leaching; doubles waste volume; requires secondary containment; incompatible with many wastes (organics, some sodium salts, silts, clays, and coal or lignite)
Pozzolanic	Waste is reacted with lime and a fine-grained siliceous material (fly ash, ground blast furnace slag, cement kiln dust) to form a solid	Low cost; readily available mixing equipment; suitable for power-plant wastes (FGD sludges, etc.) as well as a wide range of industrial wastes, including metals, waste oil, and solvents	Increases waste volume; may be subject to leaching; requires secondary containment
Thermoplastic	Waste is dried, heated, and dispensed through a heated plastic matrix of asphalt bitumen, paraffin, or polyethylene	Less increase in volume than with cement- or lime-based processes; reduced leaching relative to cement- or lime-based processes; suitable for radioactive wastes and some industrial wastes	Wastes must be dried before use; high equipment costs; high energy costs; requires trained operators; incompatible with oxidizers, some solvents and greases, some salt, and chelating/complexing agents; requires secondary containment
Organic polymers	Waste is mixed with a prepolymer and a catalyst that causes solidification through formation of a spongelike polymer matrix; urea-formaldehyde or vinyl ester-styrene polymers are used	Suitable for insoluble solids; very successful in limited applications	Pollutants are not chemically bound, subject to leaching; strongly acidic leach water may be produced; requires special equipment and operators some of the catalysts used are corrosive; harmful vapors may be produced; incompatible with oxidizers and some organics; some resins are biodegradable and decompose with time

Thermal Destruction

The thermal destruction or conversion of liquid, gaseous, and solid organic hazardous wastes is a rapidly emerging technology and is reviewed by the process categories of (65–71)

- Combustion
- Gasification
- Liquefaction
- Wet oxidation

Normally, combustion (incineration) technology is applied, and air pollution control devices are necessary for process emissions.

TABLE 9.21 Compatibility of Selected Wastes with Solidification and Stabilization Processes (63)

Waste component	Treatment type						
	Cement	Pozzolanic	Thermoplastic	Organic polymer (UF)*	Surface encapsulation	Self-cementing	Vitrification
Organics							
1. Organic solvents and oils	Many impede setting, may escape as vapor	Many impede setting, may escape as vapor	Organics may vaporize on heating	May retard set of polymers	Must first be absorbed on solid matrix	Fire danger on heating	Wastes decompose at high temperatures
2. Solid organics (e.g., plastics, resins, tars)	Good—often increases durability	Good—often increases durability	Possible use as binding agent	May retard set of polymers	Compatible—many materials are plastic	Fire danger on heating	Wastes decompose at high temperatures
Inorganics							
1. Acid wastes	Cement will neutralize acids	Compatible	Can be neutralized before incorporation	Compatible	Can be neutralized before incorporation	May be neutralized to form sulfate salts	Can be neutralized and incorporated
2. Oxidizers	Compatible	Compatible	May cause matrix breakdown, fire	May cause matrix breakdown	May cause deterioration of encapsulating materials	Compatible if sulfates are present	High temperatures may cause undesirable reactions
3. Sulfates	May retard setting and cause spalling unless special cement is used	Compatible	May dehydrate and rehydrate causing splitting	Compatible	Compatible	Compatible	Compatible in many cases
4. Halides	Easily leached from cement, may retard setting	May retard set, most are easily leached	May dehydrate	Compatible	Compatible	Compatible if sulfates are also present	Compatible in many cases
5. Heavy metals	Compatible	Compatible	Compatible	Acid pH solubilizes metal hydroxides	Compatible	Compatible if sulfates are present	Compatible in many cases
6. Radioactive materials	Compatible	Compatible	Compatible	Compatible	Compatible	Compatible if sulfates are present	Compatible

*Urea-formaldehyde resin.

Combustion. Dried sludge/solids are useful fuels for the combustion processes of incineration and gasification. Of the two, incineration provides better and essentially complete oxidation of the fuel. New and/or improved existing incineration technologies are under development for multiple applications, only one of which is hazardous waste control.

The most common incineration technologies currently applicable to hazardous wastes are (39)

- Rotary kiln
- Single chamber/liquid injection
- Multiple hearth
- Fluidized bed

Table 9.22 summarizes process principles and applications of these processes.

For hazardous waste incineration, the EPA has specified three major performance standards (8):

1. Principal organic hazardous constituents (POHCs) designed in each waste feed must be destroyed and or removed to an efficiency (DRE) of 99.99% or better; dioxins and PCBs must achieve a DRE of 99.9999%. POHCs are hazardous organic substances in the waste feed that are representative of those constituents most difficult to burn and most abundant in the waste.
2. Particulate emissions must not exceed 0.014 lb per dry standard cubic foot (180 mg per dry standard cubic meter (dscm)), corrected to 7% oxygen in the stack gas.
3. Gaseous hydrogen chloride (HCl) emissions must either be controlled to 4 lb/h (1.8 kg/h) or less, or be removed at 99% efficiency.

Additional specific requirements are specified by permit.

Gasification. Gasification produces low- to medium-energy gas, which can be used as a fuel. A market for the gaseous fuel has encouraged research and development of this technology. The applicability of gasification technologies is categorized in Table 9.23 (page 9.52). Data describing the principal gasification technologies are presented in Table 9.24.

Liquefaction. Liquefaction is the thermochemical treatment of hydrocarbonaceous materials to produce a liquid fuel—synthetic oil. Table 9.25 (page 9.53) summarizes process and performance data on the more studied liquefaction technologies.

Wet Oxidation. Wet oxidation involves the oxidation of organic wastes in the absence of air but in the presence of water. The process uses elevated temperatures in the range of 400 to 650°F (200 to 350°C) and high pressures of 1200 to 1800 lb/in² (8300 to 12,400 kN/m²) for oxidation of the organics without vaporization of the water. However, undesirable vapors may be released when the treated wastes are returned to atmospheric conditions. This technology promises benefits for destruction of contaminants that are expensive and/or essentially impractical to dewater by current methods.

REMEDIAL ACTION

The first step in a remedial action is to define the hazardous waste problem using waste characterization, site investigation data, and previous studies. Next, remedial action technologies are evaluated to determine the best methods for use at the specific site.

TABLE 9.22 Summary of Commonly Used Incineration Technologies (66, 73)

Type	Process principle	Application	Combustion temperature, °F (°C)	Residence time	Excess air, % stoichiometric
Rotary kiln	Waste is burned in a rotating refractory cylinder.	Any combustible solid, liquid, or gas	1200–2500 (650–1370)	Seconds for gases; hours for liquids and solids	50–250
Single chamber/liquid injection	Wastes are atomized with high-pressure air or steam and burned in suspension.	Liquids and slurries that can be pumped	1300–3000 (700–1650)	0.1 to 1 s	120–250
Multiple hearth	Wastes descend through several grates to be burned in increasingly hotter combustion zones.	Sludges and granulated solid wastes	1400–1800 (760–980)	Up to several hours	200–400
Fluidized bed	Waste is injected into an agitated bed of heated inert particles. Heat is efficiently transferred to the wastes during combustion.	Organic liquids, gases and granular or well-processed solids	1400–2000 (760–1100)	Seconds for gases and liquids; minutes for solids	100–150

TABLE 9.23 Applicability of Gasification Technologies (70)

1. Moving-bed gasifiers. These gasifiers can handle moisture levels up to 70%. They represent the best chance for high-moisture sludges.
2. Fluidized-bed gasifiers. These gasifiers have operated with moistures up to 50%. With extensive dewatering and/or employing exhaust gas heat for drying, they may prove successful for sludge gasification.
3. Fixed-bed gasifiers. These gasifiers have rather strict feed material requirements. Their applicability will most probably be limited to cogasification of briquets or pellets.

The primary technologies include surface water control, groundwater control, air pollution control, gas control, on-site treatment, on- and off-site disposal, and sediment control. These are discussed in this section (39). Direct waste treatment processes, as presented previously, are also a remedial technology alternative.

Surface Water Control

Surface water control technologies are categorized by one of the following functions (74):

1. Prevention of runoff and/or interception of runoff
2. Prevention of infiltration
3. Control of erosion

TABLE 9.24 Summary of Gasification Technologies (70)

Technology	Type of gas	Type of gasifier	Heat source	Other factors
BSP-LURGI			Starved-air combustion	
Davy Power Gas Corp.	LEG*	Fixed bed	Char combustion	In commercial use
DUVANT	LEG	Fixed bed	Char combustion	In commercial use
American Fyr Feeder	LEG	Moving bed	Char combustion	In commercial use
ANDCO-TORRAX	LEG	Fixed bed	Char combustion	In commercial use
THERMEX	LEG	Fluidized bed	Char combustion	In commercial use
Molten salt	LEG	Molten bed	External	Uses catalyst (Na ₂ CO ₃)
PUROX	MEG†	Fixed bed	Char combustion	Cogasification of sewage
SIMPLEX	MEG	Fixed bed	Char combustion	sludge with municipal solid
UCD-Cogasification	MEG	Fixed bed	Char combustion	waste and/or other materials
Renugas	MEG	Fluidized bed	Char combustion	
Multisolid fluid bed	MEG	Fluidized bed	Hot sand bed	
GERE	MEG	Moving bed	Separate char combustion	
Standard solid fuel	MEG	Moving bed	Separate char combustion	
Wright-Malta	MEG	Moving bed	External	
Solar fired	MEG	Fluidized bed	Solar radiation	
Flash hydrolysis	MEG	Entrained bed	Preheated hydrogen	Uses catalyst (Ni-Fe)

*LEG = Low-energy gas.

†MEG = Medium-energy gas.

TABLE 9.25 Summary of Major Liquefaction Technologies (70)

Process identification	Process conditions										Material requirements*		
	Primary liquefaction reactor(s)					Secondary reactor(s)							
	Category	Status	Temperature, °C	Pressure, kPa	Residence time, min	Recycle† w/w	Catalyst and chemicals	Process	Temperature, °C	Pressure, kPa		Catalyst and chemicals	Hydrogen consumption, kg/100 kg coal
COED	Pyrolysis	Active	320–870	55	NA	—	—	Hydrotreatment	400	17,000	NiMo	1.16	Oxygen, steam
Coalcon	Pyrolysis	Not active	560	3,200–14,000	5–25	—	—	—	—	—	—	2 to 4*	Oxygen, steam
Flash hydrolysis	Pyrolysis	Active	840	3,400	20–200 ms	—	—	—	—	—	—	—	Oxygen, steam
SRC II	Solvent extraction	Active	450–470	14,000	45–75	1.5–2.5	—	—	—	—	—	4.7	Oxygen, steam
EDS	Solvent extraction	Active	400–430	10,000–14,000	NA	1.1–2.0	—	Solvent hydrotreatment	—	—	CoHo	—	—
FRI	Solvent extraction	Lab scale	400	0	6	NA	HF, H ₂ SO ₄ , S ₆ × 5	Solvent hydrotreatment	150	0	NI	NA	—
ADL	Solvent extraction	Lab scale	400	690	60	NA	—	Coking	450	200	—	NA	—
H-Coal	Catalytic hydrogenation	Active	450	21,000	NA	2.4	CoHo	—	—	—	3.4	—	—
Synthoil	Catalytic hydrogenation	Not active	450	21,000	15	1.5–1.9	CoHo	—	—	—	—	—	—
Molten zinc chloride	Catalytic hydrogenation	Active	380–430	14,000	15	1.5	ZnCl ₂	Finishing and ZnCl ₂ combustor	500 930–980	14,000 400	Air + HCl	6.0*	Hydrochloric acid; zinc chloride (0.4 kg/100 kg coal)
CO-steam	Catalytic hydrogenation	Bench scale	430	28,000	120	NA	Na ₂ CO ₃	—	—	—	—	—	Carbon monoxide and steam
Fischer Tropsch	Indirect liquefaction	Commercial	230–330	2,300–2,500	NA	—	Fe/Co	—	—	—	—	—	Oxygen, steam
MTG	Indirect liquefaction	Commercial, 13,000 kg/day	NA	NA	NA	—	Zeolite	Methanol synthesis (ICI)	200–300	5,200–10,400	C4	—	Oxygen, steam

* ALL values reported on dry weight basis of feedstock coal except asterisk, which denotes moisture and ash-free basis (MAF) of coal.

† Recycle ratio (w/w) referenced to feed coal.

Note: NA = not available.

TABLE 9.25 Summary of Major Liquefaction Technologies (70) (continued)

		Process Performance									
		Characteristics of liquid product oils*									
Process identification	Category	Status	Yield, m ³ /Hg	°API†	% N	% S	% O	H/C	Heating value MJ/k8 (HHV)	Other products	Comments
COED	Pyrolysis	Active	0.19	-4/(25)	1.1	2.8	8.5	1.07	34.9	Char 60% w/w dry coal	Final product from hydro-treatment. The liquid yields are highly dependent on coal characteristics.
Coalcon	Pyrolysis	Not active	0.26-0.28	—	—	—	—	—	—	SNG, LPG	
Flash hydrolypyrolysis	Pyrolysis	Active	0.28-0.38	—	0.5	0.2	—	—	—	—	Data based on 30-40% carbon conversion. Performance is highly dependent on mixing characteristics.
SRC II	Solvent extraction	Active	0.46	5/39	0.9/0.4	0.3/0.2	3.3/3.9	1.09/1.64	40.1-45.2	SNG, LPG	
EDS	Solvent extraction	Active	0.44-0.46	-0.5/31.1	0.66/0.21	0.41/0.47	1.83/2.82	1.03/1.53	39.7-42.5	Fuel gas	A separate reactor is required for regenerating the solvent donor in a self-sufficient plant. Product yield and characteristics are a function of recycled solvent characteristics.
FRI	Solvent extraction	Lab scale	0.70	—	—	—	—	—	—	SNO	Requires complex chemicals recovery system and involves handling of strong acids such as HF or HCl.
ADL	Solvent extraction	Lab scale	0.42-0.49	—	—	—	—	—	—	—	Data based on 44-52% coal conversion using a cyclic, batch-type process.

H-coal	Catalytic hydrogenation	540	0.52	13/32.3	0.65/0.42	0.29/9.3	—	—	—	High-pressure reaction
Synthoil	Catalytic hydrogenation	Not active								
Molten zinc chloride	Catalytic hydrogenation	1.1	0.77	—	<0.025	<0.05	—	—	SNG, LPG	Three-stage reactor sequence with intercoolers. Involves handling of strong, corrosive materials and requires controlled conditions to keep ZnCl ₂ in molten state.
CO-steam	Catalytic hydrogenation	Bench scale	0.65	—	—	—	—	—	—	Data based on reported yield of 60% on MAF coal. High-pressure reaction.
Fischer-Tropsch	Indirect liquefaction	Commercial	NA	—	—	—	—	—	LPG, alcohols, waxes, phenols	Relatively complex, requiring several separating and purification stages. 80% of product is high-quality gasoline.
MTG	Indirect liquefaction	Commercial 13,000 kg/day	0.30	—	—	—	—	—	LPG	

*Characteristics reported for bulk oil product except when delineated by two values representing the heavy and light distillate fractions, respectively, as follows: heavy value/light value.

[†]Parentheses denote specific gravity after hydrotreatment; without parentheses denotes raw product.

TABLE 9.26 Summary of Surface Water Control Technologies (39)

Technology	Primary function					
	Prevent or intercept runoff or runoff	Prevent or minimize infiltration	Reduce erosion	Collect and transfer water	Protection from flooding	Discharge water
Capping		X				
Lagoon covers		X				
Grading	X		X			
Revegetation	X	X	X			
Dikes and berms	X		X		X	
Channels and waterways			X	X		
Terraces and benches	X		X			
Chutes and downpipes			X	X		
Seepage basins and ditches						X
Sedimentation basins and ponds	X					X
Levees and floodwalls	X				X	

4. Collection and transfer of water
5. Storage and discharge of water
6. Protection from flooding

As illustrated by the common technologies and their functions in Table 9.26, surface water control normally requires use of combinations of technologies.

Capping. Single- and multilayered caps are designed to minimize contact between infiltrating rainwater and emplaced wastes. Single-layered caps have limited applications and are generally used as temporary measures for a site-specific project.

The EPA recommends a three-layered cap system consisting of an upper vegetative layer underlain by a drainage layer over a low-permeability layer (75). An example of a multilayered cap is shown in Figure 9.14. The low-permeability soil liner layer is intended to provide long-term protection even if the synthetic liner fails after time.

Floating Covers. Prior to a final closure, floating covers are used on a temporary basis to prevent overtopping of a hazardous waste impoundment. A one-piece synthetic liner is placed over the impoundment with perimeter anchors and floating supports. The primary design objective is to keep out rainwater; if required, gas collection is included in the cover design.

Grading. Remedial site grading is used to reshape existing topography and drainage patterns in order to manage infiltration and runoff, including erosion control. Storm sewers and other water-handling measures are often used to effectuate the grading objectives. Grading is also an important task in a final closure plan.

Revegetation. The establishment of a vegetative cover is a cost-effective short- and long-term method to stabilize the land surface from wind and water erosion and to reclaim the land for other uses. Revegetation practices are site-specific and local sources should be consulted on practices in the project area.

Important characteristics of selected grasses and legumes are shown in Table 9.27 (page 9.58). In addition, compatible mixes of shrubs and trees are included in revegetation of sites for long-term stabilization.

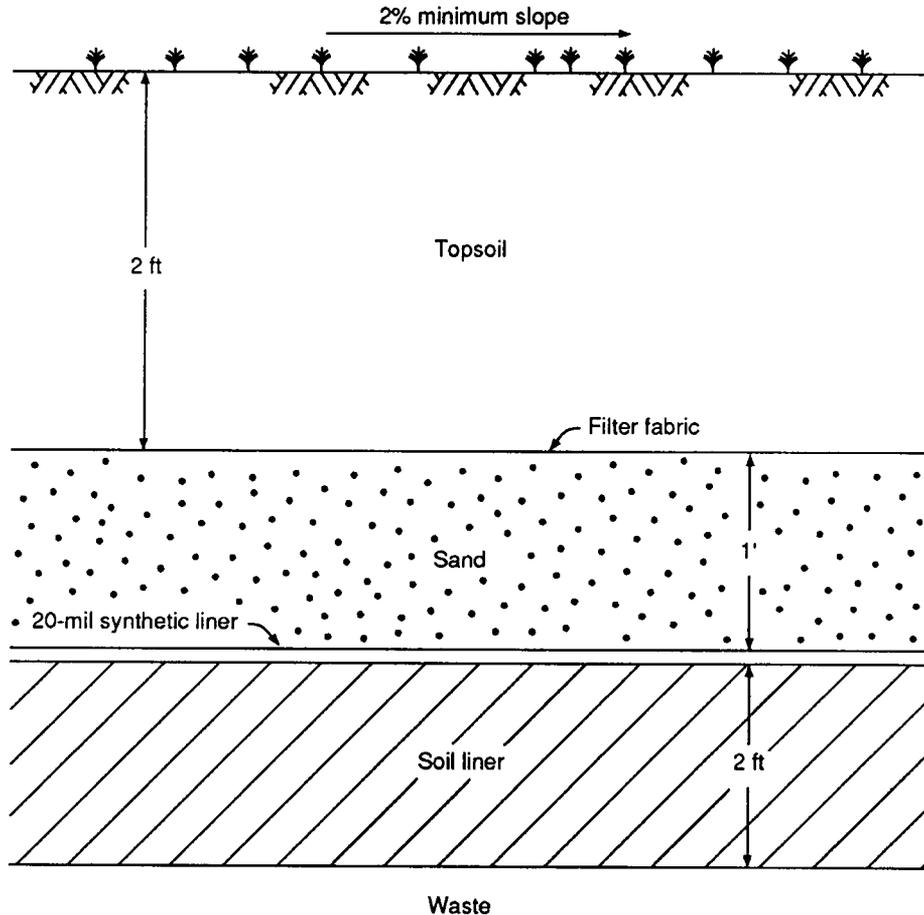


FIGURE 9.14 Typical multilayered cap (39).

Diversion and Collection. Surface water diversion and collection are used to control run-on and runoff and to reduce erosion and, in some cases, to protect sites from flooding and to control surface water discharges. Table 9.25 lists common technologies and their normal period of use in hazardous waste management projects. Since these methods are relatively easy to implement, they are often used as a first or emergency means to contain and control runoff.

Groundwater Control

The principal strategies for control of groundwater contamination are containment of a plume; removal of a plume after stopping the source of contamination; diversion of clean groundwater from an area of contamination or of polluted groundwater from a drinking water supply well; or prevention of leachate formation by artificially lowering the water table beneath a source of contamination. The practiced technologies for implementing these strategies are

TABLE 9.27 Characteristics of Selected Grasses and Legumes (76)

Characteristic	Degree*	Common examples
Texture	Fine	Kentucky bluegrass, bentgrass, red fescue
	Coarse	Smooth bromegrass, reed canarygrass, timothy
Growth height	Short	Kentucky bluegrass, buffalograss, red fescue
	Medium	Redtop, perennial ryegrass
	Tall	Smooth bromegrass, timothy, switchgrass
Growth habit	Bunch	Timothy, big bluestem, sand dropseed, perennial ryegrass
	Sod former	Quackgrass, smooth bromegrass, Kentucky bluegrass, switch grass
Reproduction	Seed	Red and alsike clover, sand dropseed, rye, perennial ryegrass, field bromegrass
	Vegetative	Prairie cordgrass, some bentgrasses
	Seed and vegetative	White clover, crownvetch, quackgrass, Kentucky bluegrass, smooth bromegrass
Annuals	Summer	Rabbit clover, oats, soybeans, corn, sorghum
	Winter	Rye, hairy vetch, field bromegrass
Perennials	Short-lived	Timothy, perennial ryegrass, red and white clover
	Long-lived	Birdsfoot trefoil, crownvetch, Kentucky bluegrass, smooth bromegrass
Maintenance	Difficult	Tall fescue, reed canarygrass, timothy, alfalfa
	Moderate	Kentucky bluegrass, smooth bromegrass
Shallow-rooted	Easy	Crownvetch, white clover, birdsfoot trefoil, big bluestem
	Weak	Sand dropseed, crabgrass, foxtail, white clover
	Strong	Timothy, Kentucky bluegrass
Deep-rooted	Weak	Many weeds
	Strong	Big bluestem, switchgrass, alfalfa, reed canarygrass
Moisture	Dry	Sheep fescue, sand dropseed, smooth bromegrass
	Moderate	Crested wheatgrass, red clover
	Wet	Reed canarygrass, bentgrass
Temperature	Hot	Lehman lovegrass, fourwing saltbush, ryegrass
	Moderate	Orchard grass, Kentucky bluegrass, white clover
	Cold	Alfalfa, hairy vetch, smooth bromegrass, slender wheatgrass

*Variety-specific characteristic, subcharacteristic, or favored condition.

TABLE 9.28 Duration of Surface Water Diversion and Collection Measures (39)

Technology	Duration of normal use
Dikes and berms	Temporary
Channels (earthen and CMP)	Temporary
Waterways	Permanent
Terraces and benches	Temporary and permanent
Chutes	Permanent
Downpipes	Temporary
Seepage ditches and basins	Temporary
Sedimentation basins	Temporary
Levees	Temporary
Floodwalls	Permanent

1. Groundwater pumping
2. Subsurface drains
3. Subsurface barriers
4. In situ treatment

Discussions follow on the first three technologies; in situ treatment is presented later in this chapter.

Groundwater Pumping. The control of groundwater to contain or remove a hazardous waste plume or to adjust the water table to prevent plume formation involves groundwater pumping. This approach involves the extraction of water from or injection of water into wells. The types of wells used include wellpoints, suction wells, ejector wells, and deep wells. The technology base is similar to that used in the construction industry for groundwater control (77).

The selection of the well type depends on the depth of contamination and the local subsurface hydrologic and geologic characteristics. Also, the contaminant characteristics and the disposition of pumpage influence the well system selection.

Subsurface Drains. In this context, subsurface drains are buried conduits used to collect and convey liquids (groundwater) by means of gravity. The system creates a continuous zone of influence in which groundwater flows toward the drains. Like groundwater pumping, subsurface drains are used to contain or remove a plume or to artificially lower the water table. However, the practical use of drains is limited to shallow depths.

The components of a subsurface drainage system are illustrated in Figure 9.15. Manholes (Figure 9.16)

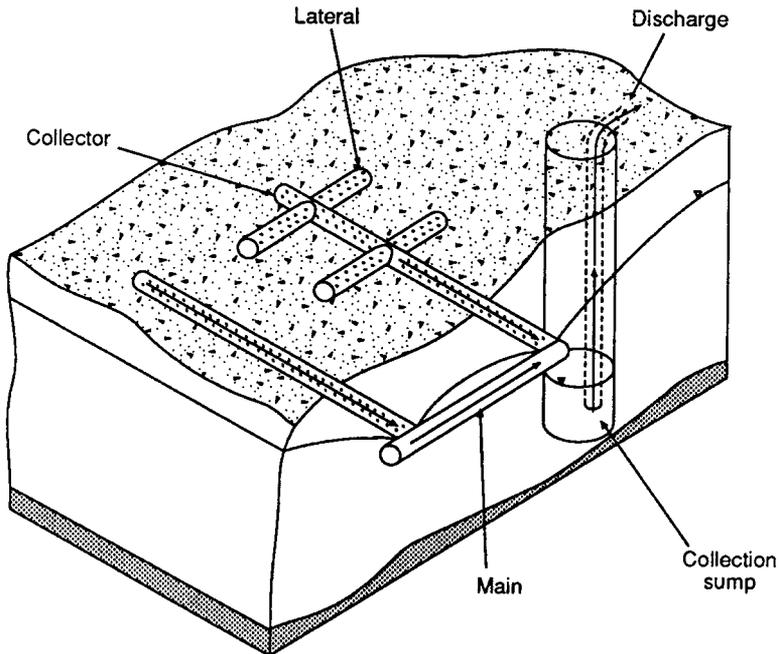


FIGURE 9.15 Subsurface drainage system components (39).

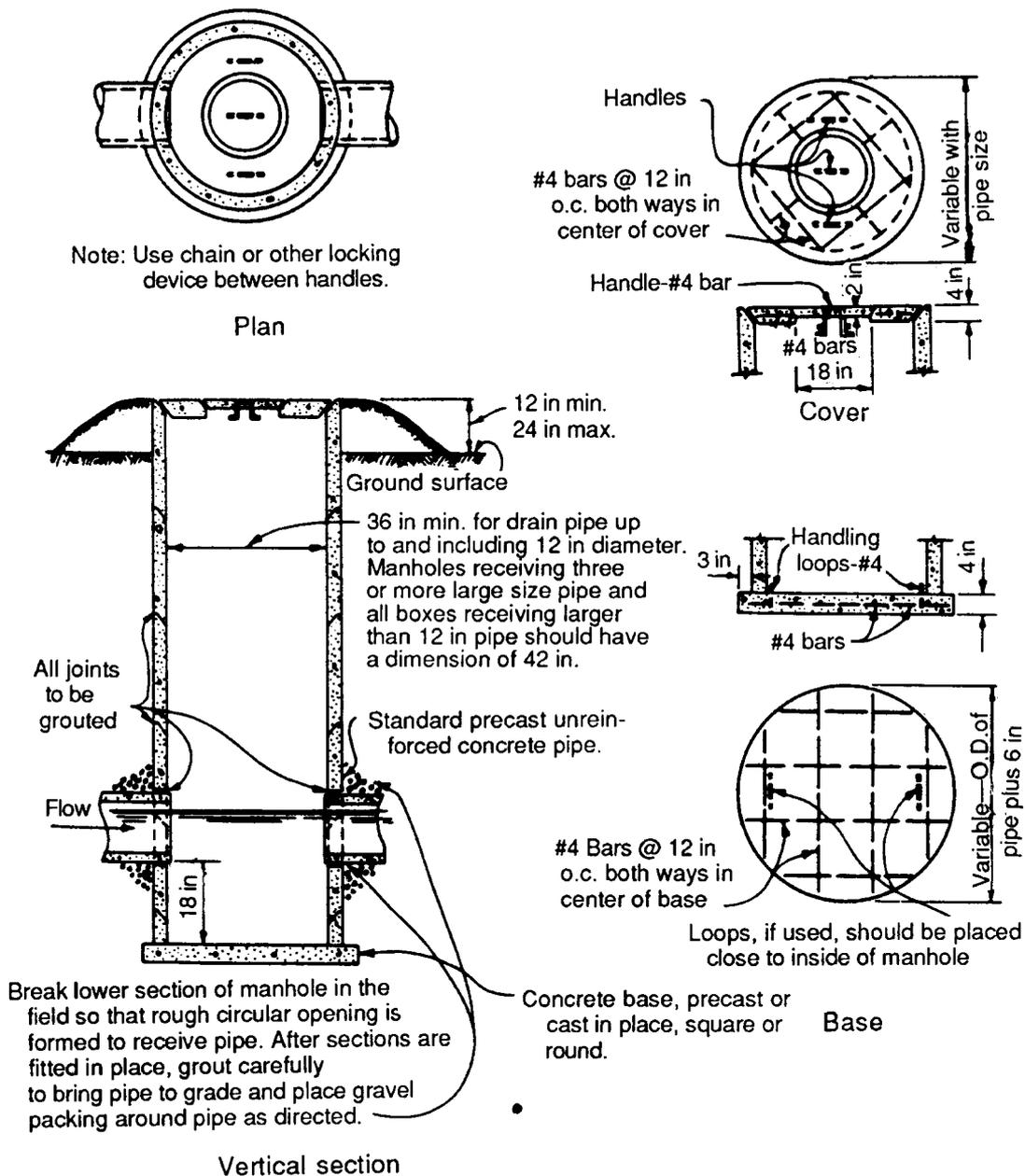


FIGURE 9.16 Typical drainage system manhole (78).

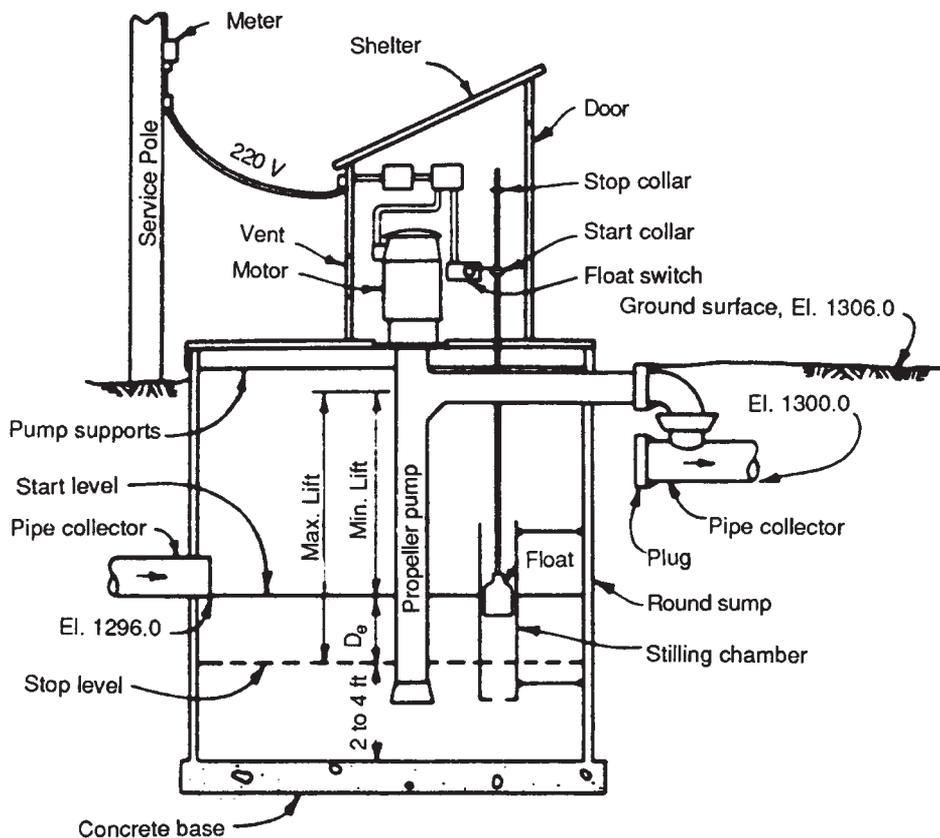


FIGURE 9.17 Typical drainage system pump station (78).

are located at junction points, changes in alignment or grade, and other designated points. Finally, a collection sump and pumping system are designed to pump contaminated groundwater to treatment or to pump in contaminated groundwater to surface drainage. Figure 9.17 shows components a typical pump station.

An example of a one-sided subsurface drain for collecting a contaminated plume is shown in Figure 9.18, which also illustrates how a subsurface barrier is used to reduce the collection of uncontaminated groundwater. Another example is a circumferential drain, Figure 9.19 (page 9.63), used to lower the water table below a hazardous waste site.

Subsurface Barriers. Slurry walls, grouting, sheet piling, and bottom sealing are typical subsurface barrier technologies used to divert groundwater flow or to minimize plume movement and leachate generation.

The most common subsurface barriers are slurry walls, which are constructed in a vertical trench, as illustrated in Figure 9.20 (page 9.64). The placements of slurry walls relative to upgradient, downgradient, and circumferential groundwater are shown in Figure 9.21 (page 9.65).

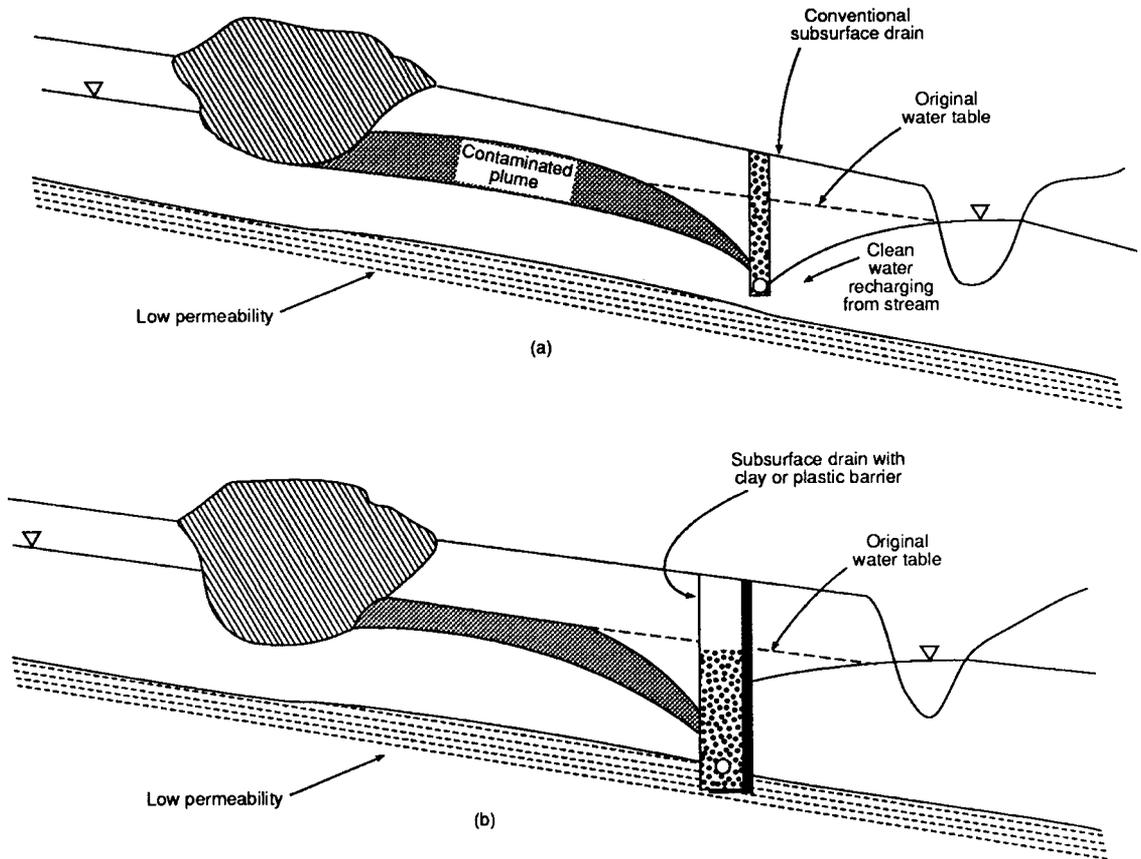


FIGURE 9.18 Subsurface drainage system collecting a contaminated plume. (a) The conventional subsurface drain receives recharges from the stream as well as from the leachate plume, resulting in larger collection and treatment. (b) One-sided drainage reduces collection of uncontaminated groundwater (79).

Air Pollution Control

Gas emissions from surface impoundments and landfills and fugitive dust cause air pollution problems at uncontrolled hazardous waste sites (131).

Gas Emission Control. Vaporization of liquids, venting of entrained gases, and by-products of biological and chemical reactions are the primary source of hazardous gas emissions.

For short-term control of releases from an impoundment, a floating cover may be used or the emissions may be reduced with addition of floating immiscible liquids or floating polypropylene spheres. The effectiveness of the latter options is derived by significantly reducing the air-liquid interface area.

Venting of gases is normally associated with a landfill operation. If an air pollution problem exists, an active interior gas collection and recovery system will most likely be required by a regulatory agency. Gas collection technology is discussed later in this chapter.

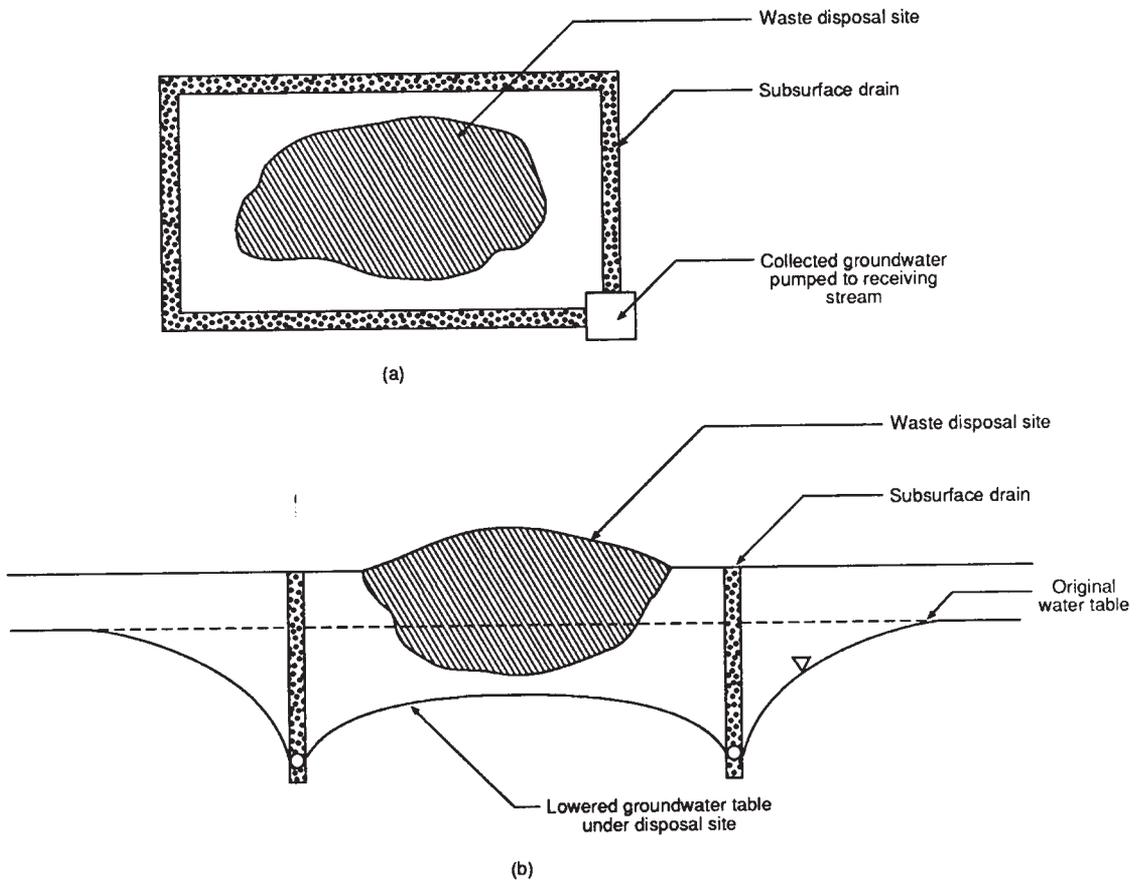


FIGURE 9.19 Subsurface drainage system lowering water table. (a) Plan view; (b) cross section (79).

Fugitive Dust Control. Wind erosion, vehicular traffic, and site excavation permit particles to be raised from the ground to become an air pollution control problem. The combined effects of all three dust sources at a hazardous waste site is most undesirable.

By lowering wind velocity to below that required to move larger particulates, a wind fence is a proven control measure. However, inhalable particles, less than $10\ \mu\text{m}$, are not consistently controlled (81). A wind fence is constructed with polyester, or similar material, screen to heights of 3 to 10 ft (1 to 3 m).

Fugitive dust from vehicular traffic on haul roads and exposed surfaces and from excavation and stockpiling of hazardous wastes may be controlled with dust suppressants (82). In some cases, a simple water spray may be used, as is commonly done on unpaved roads and at construction sites. In most cases, suppressants are required to enhance a bond between soil particles for a longer period of time. Table 9.29 lists some commercially available suppressants, their formulation, and recommended application data.

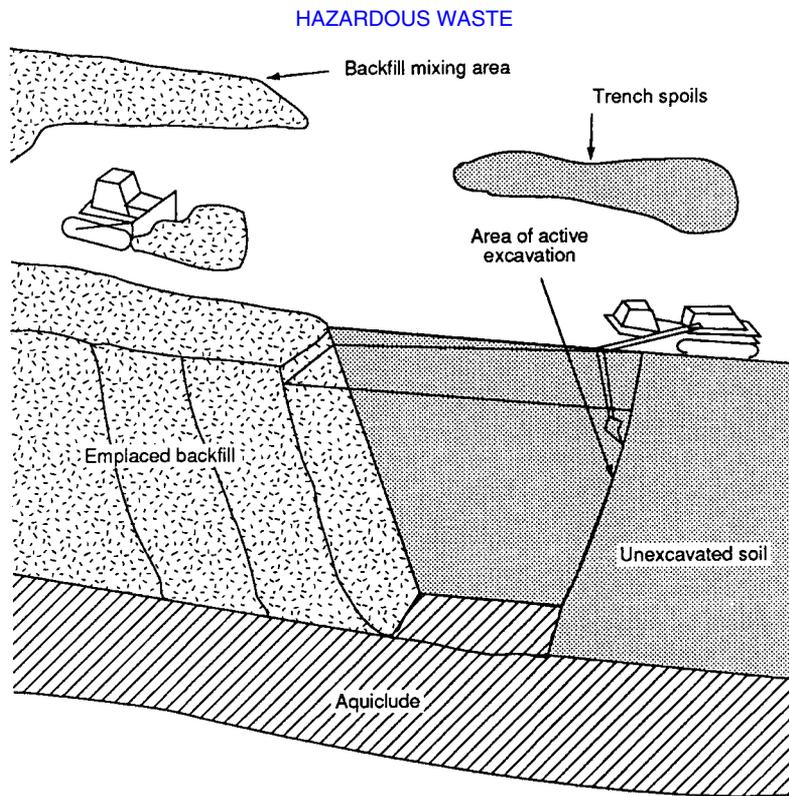


FIGURE 9.20 Slurry trench construction (80).

TABLE 9.29 Commercial Dust Suppressant Data (82)

Name	Formulation	Application concentration, %*	Application rate, gal/yd ²
Soil Seal	Latex acrylic copolymer	3	1.0
AMSCO-RES 4281	Carboxylated styrene-butadiene copolymer	20	0.6
Supac 8 NP	Nonwoven geotextile	8 oz/yd	12-ft rolls
Flambinder	Lignosulfonate	17	0.5
Genaqua	Vinyl acetate resin	10	0.2
Curasol	Synthetic resin	3	0.3
M166 and M167	Latex	7 (M166) 0.2 (M167)	0.5
Coherex CRF	Petroleum resin	25	0.5
Sherman process (no grass seed)	Straw mulch with emulsified asphalt	NA	NA
Sherman process (with grass seed)	Straw mulch with emulsified asphalt	NA	NA
Terra Track I	Vegetable gum	0.3	1.4

*Percent compound in aqueous solution, unless otherwise indicated.

NA = not applicable.

Note: 1 oz/yd = 32 mg/m; 1 gal/yd² = 0.22 L/m².

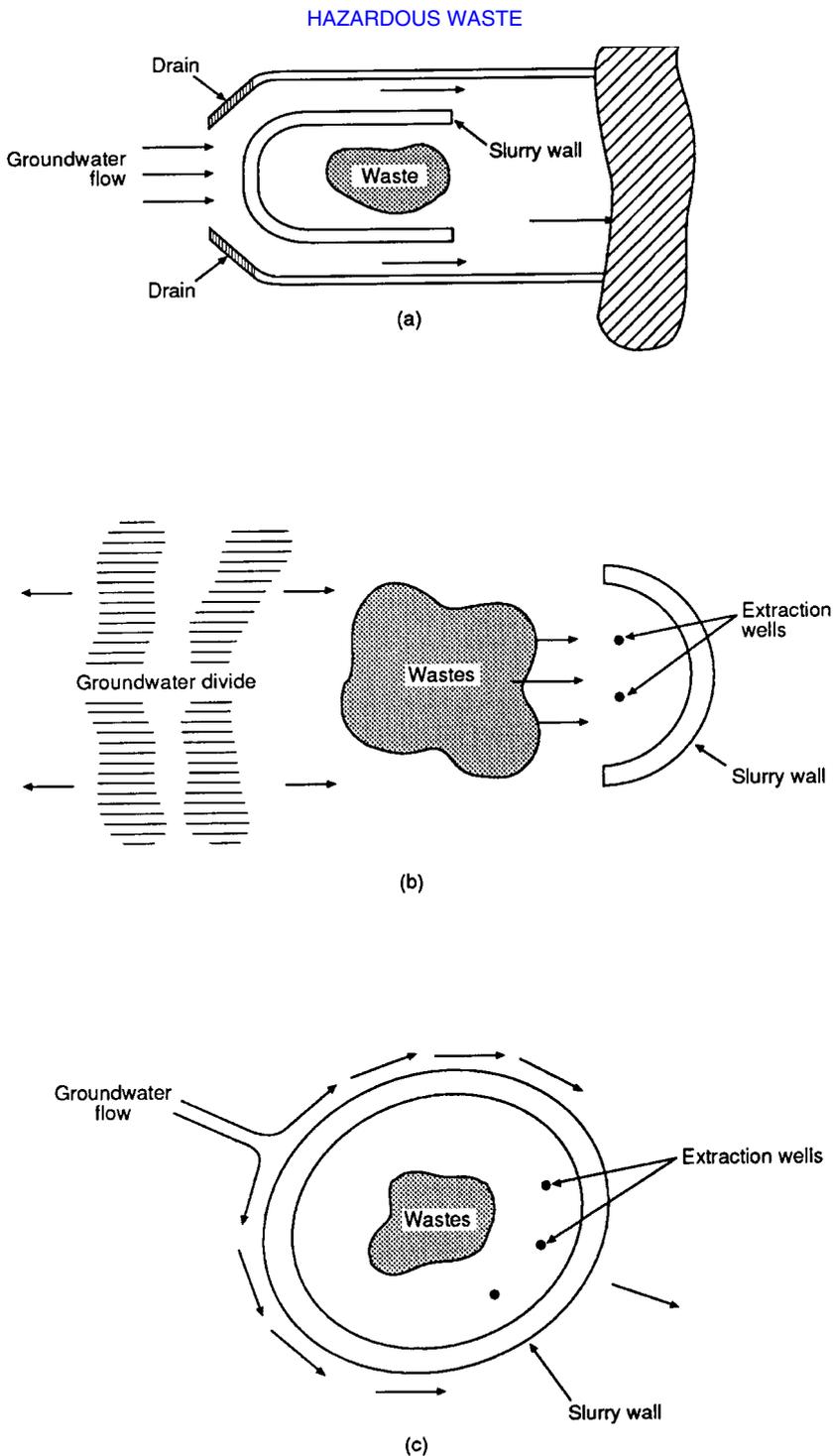


FIGURE 9.21 Slurry wall placement. (a) Upgradient; (b) downgradient; (c) circumferential (80).

Gas Control

Subsurface gas production is typically the result of biological degradation of organics but may also result from volatilization of organic liquids. In either case, a potential hazard exists. The technologies normally considered for gas control are either passive or active collection systems.

Passive Gas Collection. Subsurface gas migration may be controlled without mechanical components by passive gas collection systems. Passive systems are categorized as either high- or low-permeability systems, which may be used singularly or in combinations.

The principal aspects of a high-permeability system are shown in Figure 9.22. A low-permeability system is similar except that a synthetic membrane or other barrier is used, as illustrated in Figure 9.23 (page 9.68).

Active Gas Collection. Perimeter or interior active gas collection systems use mechanical means, such as a centrifugal blower, to alter pressure gradients and, thus, extract gas for treatment, utilization, or controlled venting. Figure 9.24 (page 9.69) illustrates the components of both active systems at a closed landfill site. The principal features of a gas extraction well are shown in Figure 9.25 (page 9.70).

In Situ Treatment

Biological, chemical, and physical processes are available for in situ treatment of hazardous wastes (39, 84, 85). In situ treatment lacks the proven performance record of most remedial technologies.

Biological Treatment. The biological treatment processes, also called bioreclamation, require the stimulation of microorganisms to decompose a specific organic contaminant(s). Research and development of this technology is proving successful, especially with use of aerobic processes (86). A hypothetical configuration of an aerobic system using extraction and injection wells appears in Figure 9.26 (page 9.71).

Chemical Treatment. In situ chemical treatment uses injection of a chemical(s) to degrade, immobilize, or flush out a contaminant. Tables 9.30 and 9.31 (page 9.72 and 9.73) summarize treatment methods for organic and inorganic wastes, respectively.

Physical Treatment. The least-developed in situ technologies are physical processes to immobilize or detoxify contaminants. Methods under investigation include heating for thermal decomposition, vaporization or distillation of organics, and freezing of surrounding ground as a temporary means to contain the contaminant until other remedial action is taken.

Waste and Soil Disposal

Following excavation and removal of hazardous wastes and contaminated soils, on-site treatment and disposal facilities, such as a landfill, may be used. Otherwise, hauling to an off-site landfill, incinerator, or other treatment works is required. The required excavation and removal equipment is the same as conventional heavy construction equipment (39, 87). Applications, capabilities, and limitations of various equipment types are shown in Table 9.32.

Sediment Disposal

Contamination of bottom sediments may result from direct discharges, surface runoff, groundwater drainage, or other means that deposit or otherwise contribute to accumulation of hazardous wastes. When

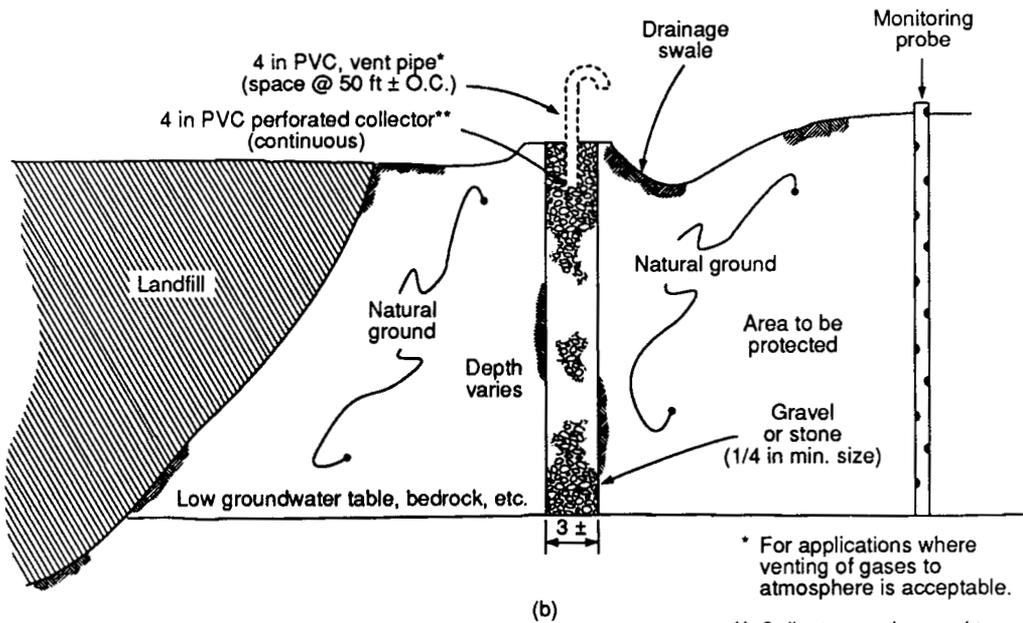
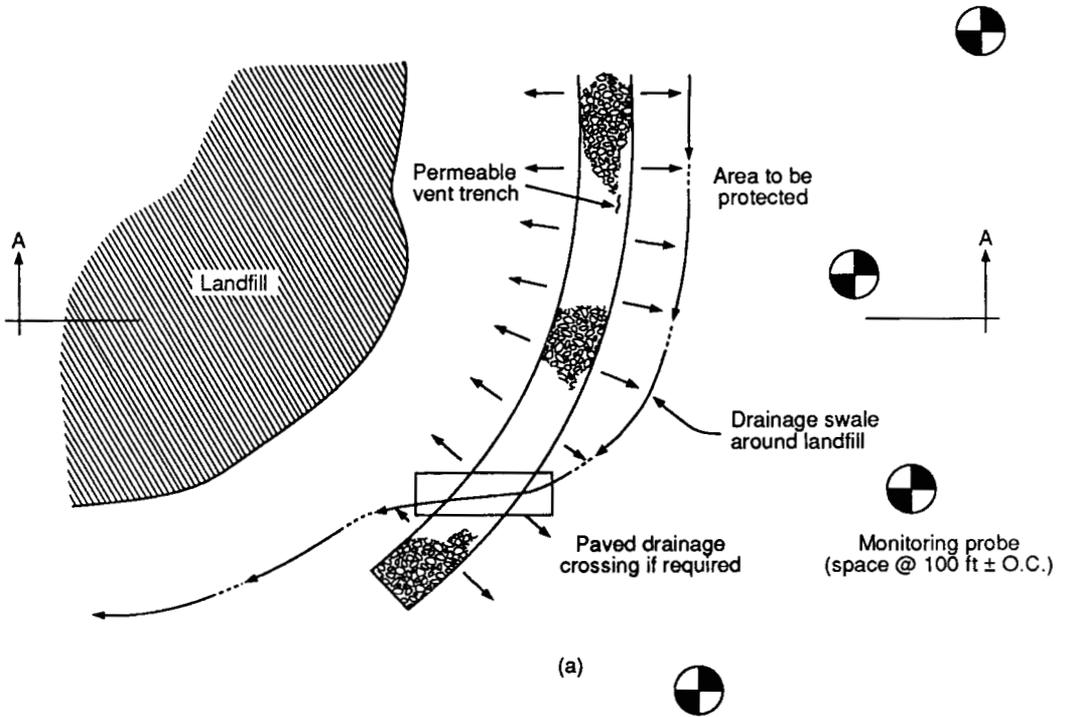


FIGURE 9.22 Passive gas collection using a permeable trench. (a) Plan view; (b) section A-A (83).

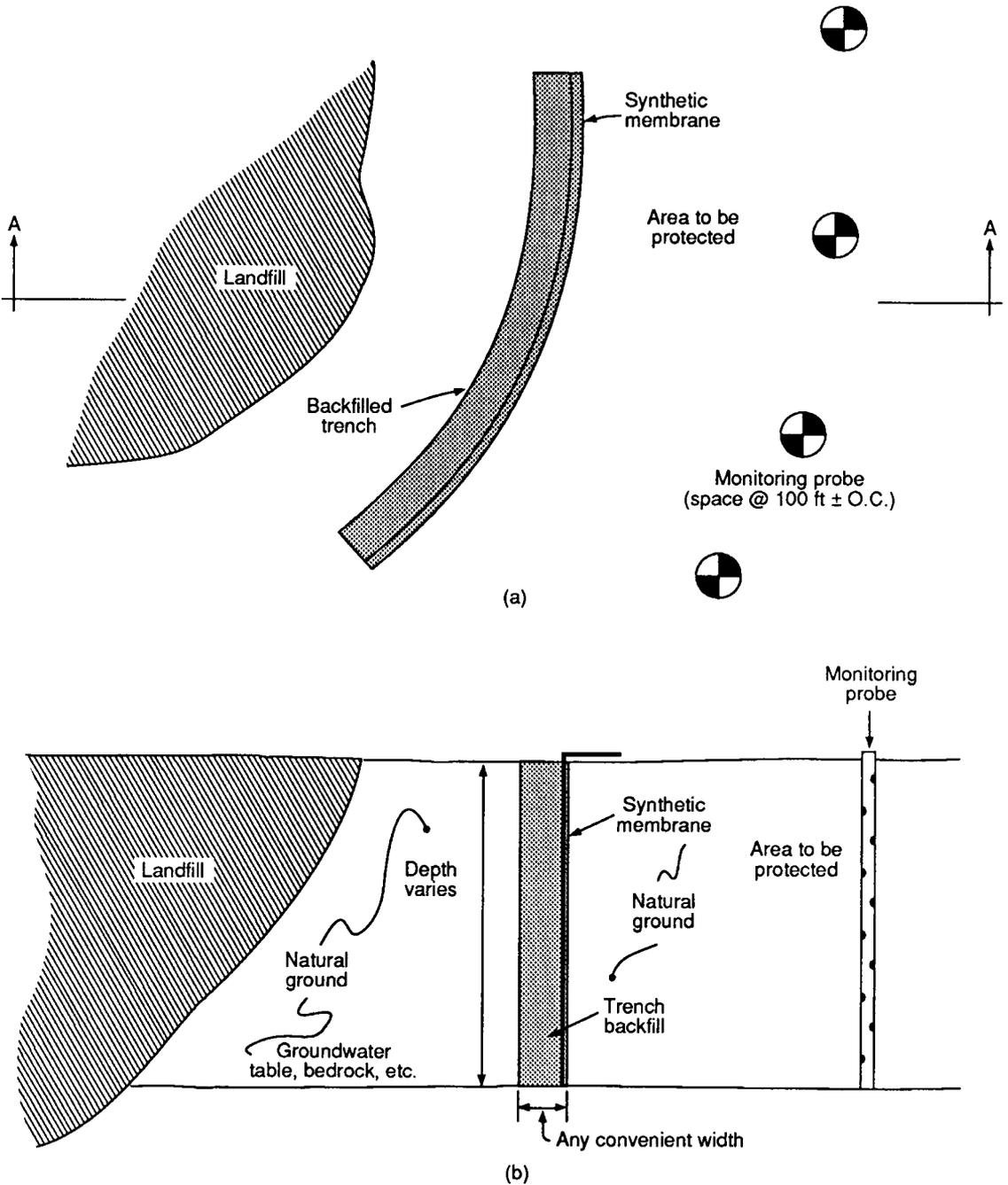


FIGURE 9.23 Passive gas collection using a synthetic membrane. (a) Plan view; (b) section A-A (83).

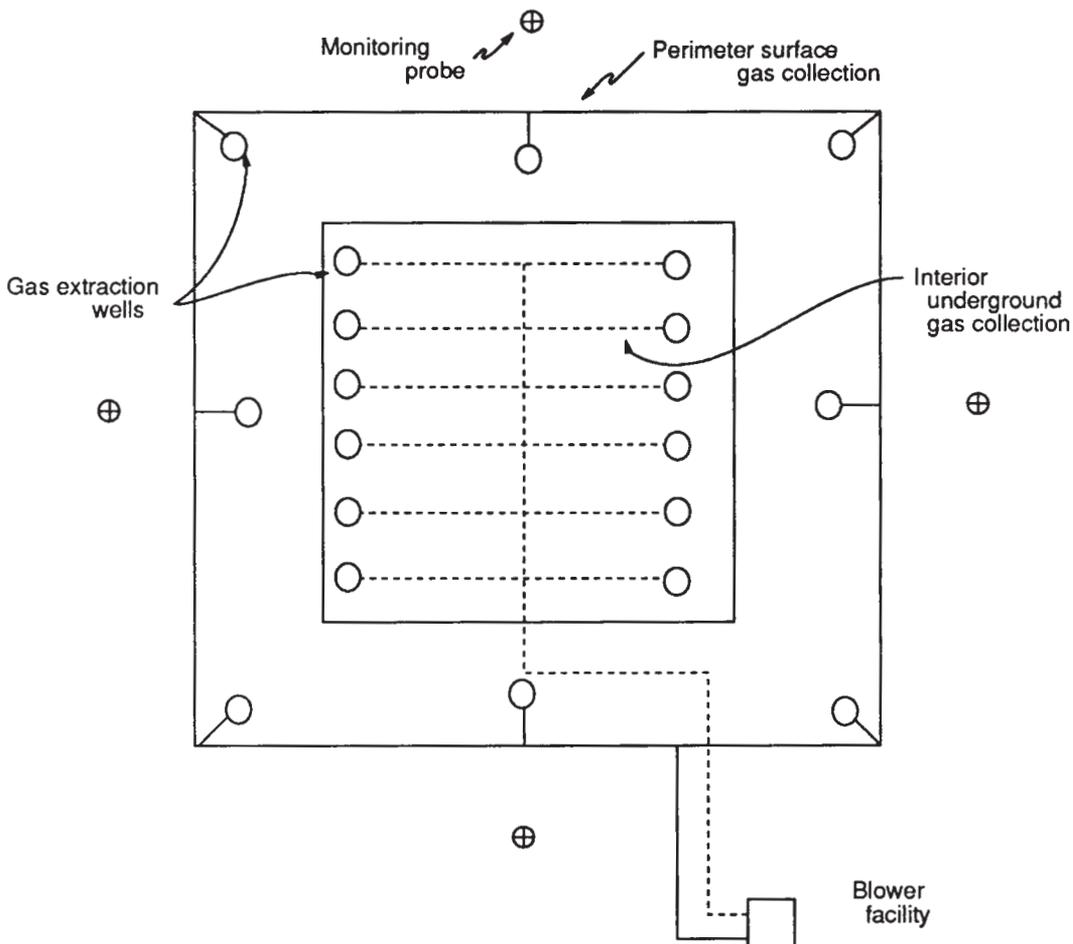


FIGURE 9.24 Active gas collection systems at a closed landfill site.

needed, remedial action involves sediment removal by dredging for subsequent treatment and disposal. Other measures, such as containment and in situ treatment, are not yet well demonstrated.

Typical applications and other factors affecting selection of dredging equipment are presented in Tables 9.33 and 9.34 (page 9.76 and 9.77). Regardless of the selected equipment, care must be taken during the dredging operation to minimize the resuspension of contaminants.

UNDERGROUND STORAGE TANKS

The detection of leakage from underground storage tanks and the implementation of corrective actions are a widespread issue in hazardous waste control and in protection of groundwater quality.

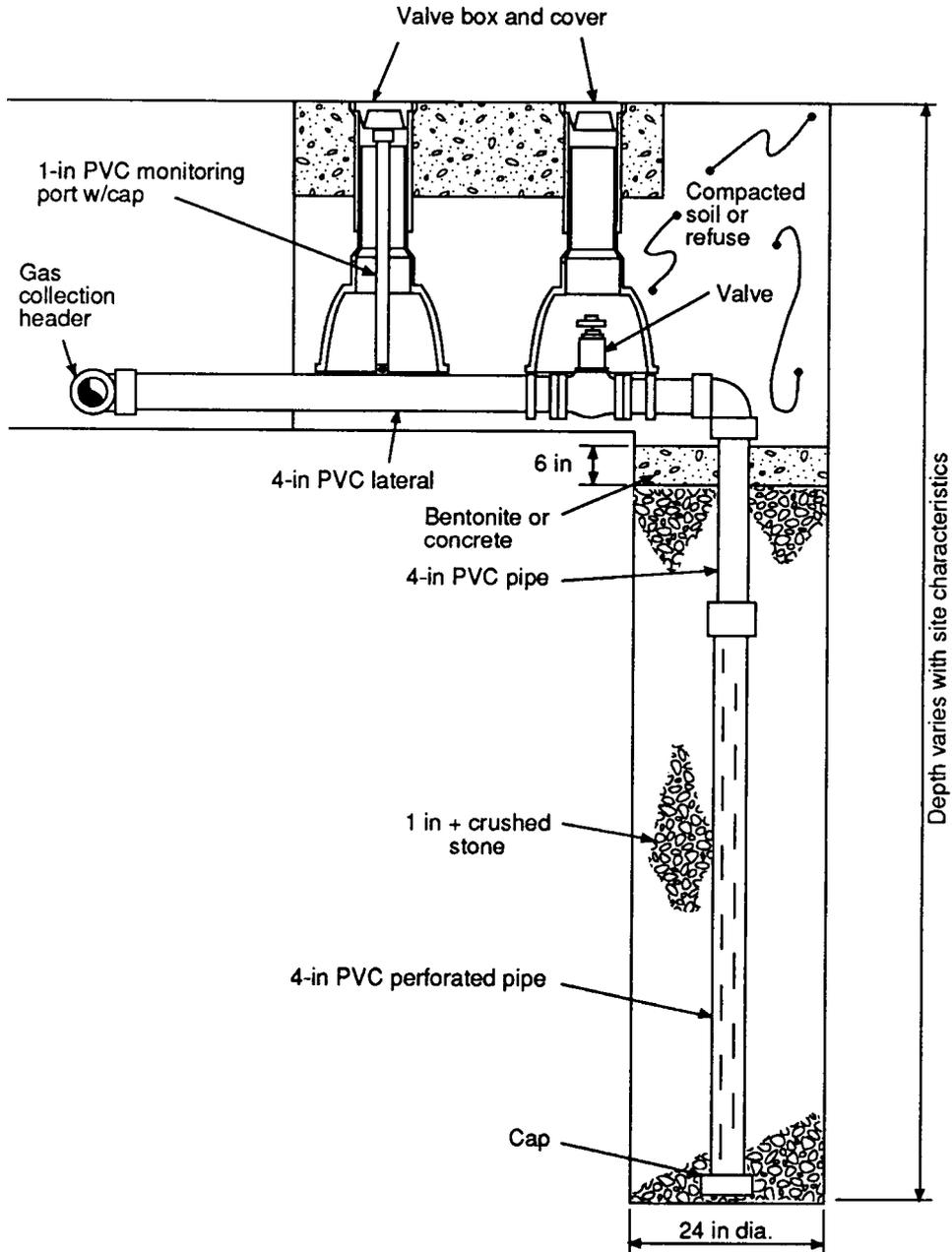


FIGURE 9.25 Gas extraction well (83).

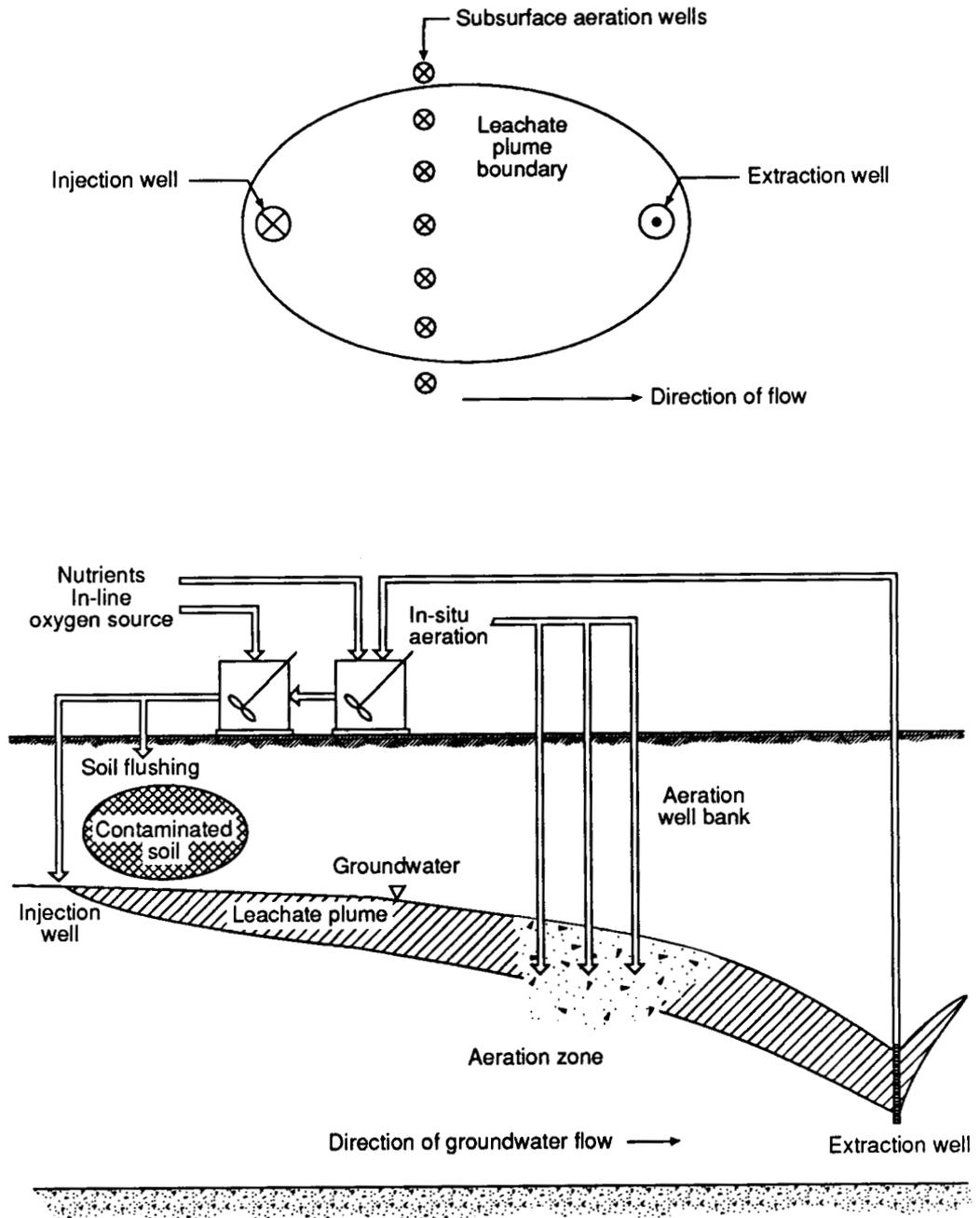


FIGURE 9.26 Bioreclamation of soil and groundwater (39).

TABLE 9.30 In Situ Chemical Treatment Methods for Organics (39)

Method	Amenable to treatment	Treatment reagents	Process
Soil flushing Water flushing Water with surfactants	Hydrophilic compounds (high solubility, low K_{ow}) Hydrophobic compounds (low solubility, high K_{ow})	Water Aqueous solutions of surfactants	Contaminated soils are flooded with water or a water-chemical mixture and the elutriated solution is collected. Contaminants are mobilized into solution by reason of solubility, formation of emulsion, or reaction.
Oxidation	Benzene and substituted benzenes Phenols Halogenated phenols Nitro aromatics PAHs Heterocyclic nitrogen and oxygen compounds Aldehydes and ketones Sulfides, disulfides Esters Amides Carbamates Organophosphorus compounds Certain pesticides (i.e., parathion, malathion, 2-4D esters, DDT)	Ozone, hypochlorite, or hydrogen peroxide	Oxidation state of compounds is increased by loss of electrons. Contaminants are detoxified, mobility is increased, or compounds are made more amenable to biological degradation.
Hydrolysis (base-catalyzed)	Aliphatic, aromatic, and oxygenated monomers vinyl chloride isoprene acrylonitrile	Water with lime or NaOH	Attack of nucleophile (e.g., water or hydroxyl ion) on an electrophile (e.g., carbon or phosphorus) resulting in bond cleavage and displacement of the leaving group.
Polymerization		Catalyst activation	Conversion of a compound to a larger chemical multiple of itself. Reduces mobility of compound in soil.

TABLE 9.31 In Situ Chemical Treatment Methods for Inorganics (39)

Method	Inorganics amenable to treatment	Treatment reagents	Process
Precipitation		Sodium or calcium sulfide	Formation of insoluble metal precipitate, thereby reducing the mobility of the metal
Sulfide	Heavy metals	Lime, calcium carbonate	
Carbonate and hydroxide	Heavy metals	Superphosphate fertilizer	
Phosphate	Heavy metals		
Soil flushing			
Acids or bases	Heavy metals	Dilute solutions of acids or bases	Involves solubilizing the metals followed by extraction of the metal ions
Chelates	Heavy metals	Chelating agents such as citric acid or EDTA	Formation of stable metal chelates; depending on chelating agent, metal chelate is either strongly sorbed to soil or is highly mobile and can be flushed using water or dilute acid solutions
Oxidation	Trivalent arsenic	Potassium permanganate	Oxidizes trivalent arsenic to pentavalent arsenic, and results in precipitation of arsenic-iron-manganese compounds
Reduction	Hexavalent chromium	Ferrous sulfate	Reduces Cr(VI) to Cr(III)
	Hexavalent selenium	Ferrous sulfate	Reduces Se(VI) to Se(IV)

Underground Tank Design

The design of underground storage tanks and associated piping begins with consideration of two primary issues:

1. Assessment of need for underground facilities
2. Protection from external corrosion

These issues apply to new installations as well as replacement facilities deemed necessary by a risk assessment or by a system failure.

The methods available to protect against corrosion include cathodic protection, coatings and linings, and/or electric isolation or the use of corrosion-resistant materials for tank construction. Common underground storage tank types include

- Asphalt-coated steel
- Double-walled fiberglass
- Double-walled steel
- Epoxy-coated steel
- Fiberglass-coated steel
- Fiberglass-coated double-walled steel
- Fiberglass

TABLE 9.32 Excavation and Removal Equipment Analysis (39)

Equipment type	Excavation of cover materials	Excavation depths	Precise excavation around buried	Removal of liquids	Hauling capabilities	Limitations/disadvantages
Backhoes (and attachments)	X	X	X			Maximum digging depth limited to 30 ft (10 m) with conventional backhoes. Maximum reach limited to 100 ft (30 m).
Dozers and loaders (and attachments)	X	X	X			Limited in speed and mobility in marshy and swampy areas. Have to be used in combination with other excavation equipment such as backhoes.
Cranes and attachments (clamshells, draglines)	X	X	X			Difficult to spot bucket for scraping and dumping. Cannot backfill or compact. Very limited mobility; slow.
Scrapers	X				X	Excellent loading action; needs to be used simultaneously with other excavation equipment such as dozer and loaders.
Haulers					X	Used only for transport purposes and not for excavation.
Pumping system				X		Not well suited for removal of liquids with high solids content.
Industrial vacuums	X		X	X	X	Cost-effective only for large-scale operations.

Other tank design considerations include site conditions, secondary containment, internal corrosion protection, leak detection and monitoring, leak control countermeasure planning, routine inspection and maintenance of all system components, and controls on product inventory. Also, it is important to ensure adherence to proper construction and installation procedures.

Piping system design has similar considerations. Newer facilities are using fiberglass or plastic materials, and some suppliers offer double-walled pipes for added safety. Lined trenches offer means of secondary containment for single-walled pipes.

Leak Detection

In order to minimize the adverse impacts of leaks from underground storage tanks, an ongoing tank-monitoring program and accurate leak-detection systems are necessary. If and when a leak is discovered, a prompt and accurate characterization of the extent of release and its migration are needed to plan corrective response actions.

Tank Monitoring. Volumetric and nonvolumetric leak-testing techniques are available. Other detection techniques include tracking tank inventories and environmental monitoring.

Regardless of the tank-monitoring technique and its believed accuracy, a number of variables may impact the results. Table 9.35 (page 9.79) lists major variables and their impact. Care during the tank testing can compensate for many of these variables.

Volumetric Leak Testing. Volumetric procedures are quantitative in that they identify the presence of a leak and yield a leak-rate measurement. Common volumetric tank testing systems are summarized in Table 9.36 (page 9.80).

Nonvolumetric Leak Testing. A number of nonvolumetric (qualitative) leak-testing methods are presented in Table 9.37 (page 9.81). If this test identifies the presence of a leak, a volumetric test is used to determine the rate of leakage.

Inventory Monitoring. Inventory control through record keeping of product purchase and consumption along with tank inventory monitoring is a simple and economical leak-detection method. However, monitoring methods for level measurements and product weight are not sensitive to lower volume leakage. Table 9.38 (page 9.82) summarizes three common inventory monitoring techniques; these methods do not require interruption of tank service or a set degree of tank fullness.

Environmental Monitoring. The quantity, rate of loss, and the source (in the case of multiple tanks) of leakage is not known when leak effects are detected by environmental monitoring. The monitoring method is effective in that it may be the first indicator of leakage, and environmental monitoring helps to define product migration and to access leak-abatement efforts. The principles of a variety of monitoring methods are presented in Table 9.39 (page 9.82).

Monitoring wells are the most prevalent form of environmental monitoring for underground storage tanks. More and more sites are incorporating wells either by owner preference and/or by requirements of regulatory agencies. Table 9.40 (page 9.83) lists the principles, advantages, and disadvantages of selected drilling methods for construction of monitoring wells.

Several means are available for sample collection; Figure 9.27 (page 9.86) is a matrix of sensitive chemical constituents compared with various sampling mechanisms. Table 9.41 (page 9.87) presents a groundwater sampling protocol along with goals and recommendations for each step.

Contaminant Migration. The migration of leaking contaminants is affected by the quantity and characteristics of the leakage and by the local subsurface conditions. Small quantities may be retained by the soils around the underground tank, whereas larger quantities will percolate away from the tank.

Characteristics of the contaminants are important in predicting the reactions of liquid and/or gas phases

TABLE 9.33 Comparison of Dredge Equipment to Selection Criteria (39)

Typical application	Type of dredge											
	Mechanical				Hydraulic				Pneumatic			
	Clamshell	Dragline	Backhoe	Plain suction	Cutterhead	Dustpan	Hopper	Portable	Air lift	Pneuma	Oozer	
Volume	A,B	A,B		B,C	B,C	B,C	B,C	A,B	A,B,E	B,C,E	B,C,E	
Setting	D,E,F,G,H	D,E,F	D,E	G,H	G,H	G,H	H	E,F,G	F,G,H	F,G,H	F,G,H	
Auxiliary facilities	I,J,M	I,J,M	I	J,K,L	J,K,L	J,K,L	None	K,L,M	J,K,L,M	J,K,L,M	J,K,L,M	
Depth limitations	None ⁴	None ³	None	35 ²	35 ²	52	12-30 ¹	11/26 ⁵	None	— ⁴	— ⁴	
Minimum, ft	None ⁶	50 ^{3~}	60	12-50 ²	602	602	36-65 ¹	15-58 ⁵	None	None ⁶	None ⁶	
Maximum, ft												
Precision obtainable												
Horizontal, ft	1 ³	1/2	1	2-3 ³	2-3 ³	2-3 ³	10 ³	1 ³	1	2-3	2-3	
Vertical, ft	1	1	1	1	1	1	1	1	1	1 ³	1	
Rate of production	30-600 ³	30-60 ¹		25-2500 ²	3500 ³	3500 ³	500-2000 ³	50-1850 ⁵	— ⁹	60-390 ²	— ⁹	
Turbidity and resuspension	High ³	High ³	High	Avg. ³	Avg. ³	Avg. ³	Avg. ³	Low ³	Low	Low ³	Low	
Availability	T	T	T	X ⁸	R ⁸	Q ⁸ X ⁸	Q ⁸ S ^x	U	V	V	W	
Transportability	4	4	I	5	5	5	6	I3 ⁸	4	4	4	
Slurry solids content	in situ	in situ	in situ	10-15%	10-20%	10-20%	10-20%	10-40%	10-20%	10-20%	up to 80% of in situ	30-70%

¹See Table 4-3, p. 54, Hand et al., 1978 (Ref. 88).²See Table 4-2, p. 51, Hand et al., 1978 (Ref. 88).³See Table 4-4, p. 59, Hand et al., 1978 (Ref. 88).⁴Determined by draft of vessel; if not vessel-mounted, there is no limiting minimum depth.⁵See Table 1, p. 12, Clark, 1983 (Ref. 87).⁶Limited only by availability of support equipment (e.g., cables, winches, etc.).⁷See Tables 4-8 through 4-11, pp. 76-79, Hand et al., 1978 (Ref. 88).⁸See Table 4-6, p. 69, Hand et al., 1978 (Ref. 88).

Legend:

Volume

- A = Small-scale, less than 1000 yd³ (see footnote 7)
 - B = Medium-scale, 1000 to 200,000 yd³ (see footnote 7)
 - C = Large-scale, greater than 200,000 yd³ (see footnote 7)
- Setting
- D = Narrow and/or very shallow (less than 5 ft) streams
 - E = Shallow (less than 20 ft) streams and rivers, navigable by small vessels
 - F = Inland lakes and ponds
 - G = Inland navigable channels and lake and coastal harbors
 - H = Great Lakes and coastal harbors

Auxiliary facilities

- I = Dump trucks
- J = Barges
- K = Transport piping
- L = Settling impoundments
- M = Crane

Availability

- Q = All or most owned by U.S. Army Corps of Engineers
- R = Based in most major harbors and commercial waterways
- S = Based in some coastal and Great Lakes harbors
- T = Widely available in general earthwork applications
- U = Widely available from contractors and vendors
- V = Limited availability through U.S. distributors
- W = Not generally available in the United States
- X = Generally available on inland commercial waterways
- Y = Can be fabricated
- Z = Very few operating in United States

Transportability

- 1 = Dredge can be moved over existing roads "as is" or with slight modification (see footnote 5)
- 2 = Dredge can be moved over existing roads after disassembling to three or fewer pieces (see footnote 5)
- 3 = Dredge can be moved over existing roads after disassembling to more than three pieces (see footnote 5)
- 4 = Dredge head can be removed over existing roads as is" or with slight modification and mounted on conventional vessel or crane.
- 5 = Transport restricted to navigation channels (greater than 5 ft depth) due to draft
- 6 = Transport restricted to deep (greater than 12 ft) navigation channels due to draft

TABLE 9.34 Comparison of Dredge Equipment for Applicability to Waste Site Conditions (39)

Factor	Type of dredge													
	Mechanical						Hydraulic						Pneumatic	
	Dragline	Backhoe	Clamshell	Plain suction	Cutterhead	Dustpan	Hopper	Portable	Air lift	Pneuma	Oozer			
Vessel draft, ft	*	*	*	5-6	3-14	5-14	12-31	11/2	3-6	N/A	7			
Hindrance to traffic	Small	Small	Small	Significant	Some	Significant	Small	Some	Some	Some	Significant			
Maximum wave height	<3	<3	<3	<3	<3	<3	<7	<1	<3	<3	<3			
Operation near structures	Yes	Yes	Yes	No	No	No	No	No	No	No	No			
Operation in open water	Limited	Limited	Limited	Limited	Limited	No	Yes	No	No	Limited	Yes			
Capable of consolidating sediments	Yes	Yes	Yes	No	Yes	Yes	No	No	No	No	Yes			
Susceptible to debris damage	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes			
Suitable for liquid or solid removal	Solid	Solid	Solid	Both	Both	Both	Both	Both	Both	Both	Both			

*Depends on draft of supporting structure. Most barges have a draft of 5 to 6 ft.

Note: 1 ft 0.3 m.

TABLE 9.35 Major Variables Affecting Leak Detection (61, 90)

Variable	Impact
Temperature change	Expansion or contraction of a tank and its contents can mask leak and/or leak rate.
Water table	Hydrostatic head and surface tension forces caused by groundwater may mask tank leaks partially or completely.
Tank deformation	Changes or distortions of the tank due to changes in pressure or temperature can cause an apparent volume change when none exists.
Vapor pockets	Vapor pockets formed when the tank must be overfilled for testing can be released during a test or expand or contract from temperature and pressure changes and cause an apparent change in volume.
Product evaporation	Product evaporation can cause a decrease in volume that must be accounted for during a test.
Piping leaks	Leaks in piping can cause misleading results during a tank test because many test methods cannot differentiate between piping leaks and tank leaks.
Tank geometry	Differences between the actual tank specifications and nominal manufacturer's specification can affect the accuracy of change in liquid volume calculations.
Wind	When fill pipes or vents are left open, wind can cause an irregular fluctuation of pressure on the surface of the liquid and/or a wave on the liquid-free surface that may affect test results.
Vibration	Vibration can cause waves on the free surface of the liquid that can cause inaccurate test results.
Noise	Some nonvolumetric test methods are sound-sensitive, and sound vibrations can cause waves to affect volumetric test results.
Equipment accuracy	Equipment accuracy can change with the environment (e.g., temperature and pressure).
Operator error	The more complicated a test method, the greater the chance for operator error, such as not adequately sealing the tanks if required by the test method in use.
Type of liquid stored	The physical properties of the liquid (including effects of possible contaminants) can affect the applicability or repeatability of a detection method (e.g., viscosity can affect the sound characteristics of leaks in acoustical leak-detection methods).
Power vibration	Power vibration can affect instrument readings.
Instrumentation limitation	Instruments must be operated within their design range or accuracy will decrease.
Atmospheric pressure	A change in this parameter has the greatest effect when vapor pockets are in the tank, particularly for leak-rate determination.
Tank inclination	The volume change per unit of level change is different in an inclined tank than in a level one.

TABLE 9.36 Volumetric Leak Testing Methods (61, 90)

Method	Principle	Claimed accuracy, gal/h	Total downtime for testing	Requires empty or full tank for test
Ainlay tank integrity testing	Pressure measurement by a coil-type manometer to determine product level change in a propane bubbling system	0.02	10–12 h (filled a night before 1.5-h testing)	Full
ARCO HTC underground tank detector	Level change measurement by float and light-sensing system	0.05	4–6 h	No
Certi-Tec testing	Monitoring of pressure changes resulting from product level changes	0.05	4–6 h	Full
“Ethyl” tank sentry	Level change magnification by a J tube manometer	Sensitive to 0.02-in level change	Typically 10 h	No
EZY-CHEK leak detector	Pressure measurement to determine product level change in an air bubbling system	Less than 0.01	4–6 h (2 h waiting after fill-up, 1-h test)	Full
Fluid-static (standpipe) testing	Pressurizing of system by a standpipe; keeping the level constant by product addition or removal; measuring rate of volume change	Gross	Several days	Full
Heath Petro Tite tank and line testing (Kent-Moore)	Pressurizing of system by a standpipe; keeping the level constant by product addition or removal; measuring volume change; product circulation by pump	Less than 0.05	6–8 h	Full
Helium differential pressure testing	Leak detection by differential pressure change in an empty tank; leak rate estimation by Bernoulli’s equation	Less than 0.05	Minimum 48 h	Empty
Mooney tank test detector	Measuring level change with a dipstick	0.02	14–16 h* (12 to 14 h waiting after fill-up)	Full
PACE tank tester	Magnification of pressure change in a sealed tank by using a tube (based on manometer principle)	Less than 0.05	14 h	Full
PALD–2 leak detector	Pressurizing system with nitrogen at three different pressures; level measurement by an electrooptical device; estimate of leak rate based on the size of leak and pressure difference across the leak	Less than 0.05	14 h (preferably 1 day before, testing, 1-h fill includes sealing time)	Full
Pneumatic testing	Pressurizing system with air or other gas; leak rate measurement by change in pressure	Gross	Several hours	No
Tank auditor	Principle of buoyancy	0.00001 in the fill pipe; 0.03 at the center of a 10.5-ft-diameter tank	1.5–3 h	Typically full
Two-tube laser interferometer system	Measuring level change by laser beam and its reflection	Less than 0.05	4–5 h [†]	No (at existing level)

*Including the time for tank end stabilization when testing with standpipe.

[†]Including 1 to 2 h for reference tube temperature equilibrium.

TABLE 9.37 Nonvolumetric Leak Testing Methods (61, 90)

Method	Principle	Claimed accuracy, gal/h	Total downtime for testing	Requires empty or full tank for test
Acoustical Monitoring System (AMS)	Sound detection of vibration and elastic waves generated by a leak in a nitrogen-pressurized system; triangulation techniques to detect leak location	Does not provide leak rate; detects leaks as low as 0.01 gal/h	1–2 h	No
Leybold–Heraeus helium detector, Ultratest M2	Rapid diffusivity of helium mixing of a tracer gas with products at the bottom of the tank; helium detected by a sniffer mass spectrometer	Does not provide leak rate; helium could leak through 0.005-in leak size	None	No
Smith and Denison helium test	Rapid diffusivity of helium; differential pressure measurement, helium detection outside a tank	Provides the maximum possible leak detection based on the size of the leak (does not provide leak rates); helium could leak through 0.05 in leak size	Few—24 h (excludes sealing time)	Empty
TRC rapid leak detector for underground tanks and pipes	Rapid diffusion of tracer gas; mixing of a tracer gas with product; tracer gas detected by a sniffer mass spectrometer with a vacuum pump	Does not provide leak rate; tracer gas could leak through 0.005-in leak size	None	No
Ultrasonic leak detector (Ultrasound)	Vacuuming the system (5 lb/in ²); scanning entire tank wall by ultrasound device; noting the sound of the leak by headphones and registering it on a meter	Does not provide leak rate; a leak as small as 0.001 gal/h of air could be detected; a leak through 0.005 in could be detected	Few hours (includes tank preparation and 20-min test)	Empty
VacuTect (Tanknology)	Applying vacuum at higher than product static head; detecting bubbling noise by hydrophone; estimating approximate leak rate by experience	Provides approximate leak rate	1 h	No
Varian leak detector (SPY2000 or 938–41)	Similar to Smith and Denison	Similar to Smith and Denison	Few—24 h (excludes sealing time)	Empty

TABLE 9.38 Inventory Monitoring Methods (61, 90)

Method	Principle	Claimed accuracy
Gage stick	Measuring product level with dipstick when station is closed	Gross
MFP-414 TLG leak detector	Monitoring product weight by measuring pressure and density at the top, middle, and bottom of tank	Sensitive to 0.1% of product height change
TLS-150	Using electronic level measurement device or programmed microprocessor inventory system	Sensitive to 0.1-in, level change

within the soil in unsaturated and saturated zones. In the saturated zones, some substances will dissolve and others will be immiscible. Immiscible substances with a specific gravity less than 1.0 will be found near the water table in the upper reaches of the saturation zone. The denser substances with a specific gravity greater than 1.0 will tend to move downward through the saturation zone. These denser substances are most likely to enter groundwater water supplies and to migrate to and through fractures in the impervious strata to confined aquifers. Table 9.42 (page 9.87) illustrates potential contaminant dispersion within different subsurface zones.

Inventory-type records are the best source of data to quantify the leakage. Initially, the extent of the release may be measured using existing underground structures and accesses to ground and surface waters. Typical sampling points are presented in Table 9.43 (page 9.88).

When new sampling points are needed, direct and indirect methods are available. Soil borings are a direct method of sampling for gas and liquid migration. Indirect methods include electrical resistivity and electro-

TABLE 9.39 Environmental Monitoring Methods (61, 90)

Method	Principle
Collection sumps	Using collection mechanism of product in collection sump through sloped floor under the storage tank
Dye method	Hydrocarbon detection by use of soluble dye through perforated pipe
Groundwater and soil sampling	Water and soil sampling
Interstitial monitoring in double-walled tanks	Monitoring in interstitial space between the walls of double-walled tanks with vacuum or fluid sensors
LASP	Diffusion of gas and vapor to a plastic material
Observation wells	Product sensing in liquid through monitoring wells at areas with high groundwater
Pollulert and Leak-X	Difference in thermal conductivity of water and hydrocarbons through monitoring wells
Remote infrared sensing	Determining soil temperature characteristic change due to the presence of hydrocarbons
Surface geophysical methods	Hydrocarbon detection by ground-penetrating radar, electromagnetic induction, or resistivity techniques
U-tubes	Product sensing in liquid; collection sump for product directed through a horizontal pipe installed under a tank
Vapor wells	Monitoring of vapor through monitoring well

TABLE 9.40 Selected Drilling Methods for Monitoring Well Construction (91)

Method	Drilling principle	Advantages	Disadvantages
Drive point	1.25- to 2-in ID casing with pointed screen mechanically driven	Inexpensive Easy to install, by hand if necessary Water samples can be collected as driving proceeds Depending on overburden, a good seal between casing and formation can be achieved	Difficult to sample from smaller-diameter drive points if water level is below suction lift. Bailing possible No formation samples can be collected Limited to fairly soft materials. Hard to penetrate compact, gravelly materials Hard to develop. Screen may become clogged if thick clays are penetrated PVC and Teflon casing and screen are not strong enough to be driven. Must use metal construction materials which may influence some water quality determinations
Auger, hollow- and solid-stem	Successive 5-ft flights of spiral-shaped drill stem are rotated into the ground to create a hole. Cuttings are brought to the surface by the turning action of the auger	Inexpensive Fairly simple operation Small rigs can get too difficult-to-reach areas. Quick setup time Can quickly construct shallow wells in firm, noncavey materials, No drilling fluid required Use of hollow-stem augers greatly facilitates collection of split-spoon samples Small-diameter wells can be built inside hollow-stem flights when geologic materials are cavey	Depth of penetration limited, especially in cavey materials. Maximum depth 150 ft Cannot be used in rock or well-cemented formations Difficult to drill in cobbles or boulders Log of well is difficult to interpret without collection of split spoons due to the lag time for cuttings to reach ground surface. Vertical leakage of water through borehole during drilling is likely to occur Solid-stem limited to fine-grained, unconsolidated materials that will not collapse when unsupported. With hollow-stem flights, heaving materials can present a problem. May need to add water down auger to control heaving or wash materials from auger before completing well (continues)

TABLE 9.40 Selected Drilling Methods for Monitoring Well Construction (91) (continued)

Method	Drilling principle	Advantages	Disadvantages
Jetting	Washing action of water forced out of the bottom of the drill rod clears hole to allow penetration. Cuttings brought to surface by water flowing up the outside of the drill rod	Inexpensive. Driller often not needed for shallow holes, In firm, noncavey deposits where hole will stand open, well construction fairly simple	Somewhat slow, especially with increasing depth Extremely difficult to use in very coarse materials, i.e., cobbles and boulders A water supply is needed that is under enough pressure to penetrate the geologic materials present Difficult-to-interpret sequence of geologic materials from cuttings Maximum depth 150 ft, depending on geology and water pressure capabilities
Cable-tool (percussion)	Hole created by dropping a heavy "string" of drill tools into well bore, crushing materials at bottom Cuttings are removed occasionally by bailer. Generally, casing is driven just ahead of the bottom of the hole; a hole greater than 6 inch in diameter is usually made	Can be used in rock formations as well as unconsolidated formations. Fairly accurate logs can be prepared from cuttings if collected often enough Driving a casing ahead of hole minimizes cross-contamination by vertical leakage of formation waters. Core samples can be obtained easily	Requires an experienced driller Heavy steel drive pipe used to keep hole open and drilling "tools" can limit accessibility Cannot run some geophysical logs due to presence of drive pipe. Relatively slow drilling method
Hydraulic rotary	Rotating bit breaks formation, cuttings are brought to the surface by a circulating fluid (mud). Mud is forced down the interior of the drill stem, out the bit, and up the annulus between the drill stem and hole wall Cuttings are removed by settling in a "mud pit" at the ground surface and the mud is circulated back down the drill stem	Drilling is fairly quick in all types of geologic materials Borehole will stay open from formation of a mud wall on sides of borehole by the circulating drilling mud. Eases geophysical logging and well construction Geologic cores can be collected. Virtually unlimited depths possible	Expensive, requires experienced driller and fair amount of peripheral equipment Completed well may be difficult to develop, especially small-diameter wells, because of mud wall on borehole Geologic logging by visual inspection of cuttings is fair due to presence of drilling mud. Thin beds of sand, gravel, or clay may be missed Presence of drilling mud can contaminate water samples, especially the organic, biodegradable muds Circulation of drilling fluid through a contaminated zone can create a hazard at the ground surface with the mud pit and cross-contaminate clean zones during circulation

TABLE 9.40 Selected Drilling Methods for Monitoring Well Construction (91) (continued)

Method	Drilling principle	Advantages	Disadvantages
Reverse rotary	Similar to hydraulic rotary method except the drilling fluid is circulated down the borehole outside the drill stem and is pumped up the inside, just the reverse of the normal rotary method. Water is used as the drilling fluid, rather than a mud, and the hole is kept open by the hydrostatic pressure of the water standing in the borehole	Creates a very "clean" hole, not dirtied with drilling mud. Can be used in all geologic formations. Very deep penetrations possible Split-spoon sampling possible	A large water supply is needed to maintain hydrostatic pressure in deep holes and when highly conductive formations are encountered Expensive—experienced driller and much peripheral equipment required Hole diameters are usually large, commonly 18 in or greater. Cross-contamination from circulating water likely Geologic samples brought to surface are generally poor, circulating water will "wash" finer materials from sample
Air rotary	Very similar to hydraulic rotary, the main difference being that air is used as the primary drilling fluid as opposed so mud or water	Can be used in all geologic formations; most successful in highly fractured environments. Useful at any depth Fairly quick Drilling mud or water not required	Relatively expensive Cross-contamination from vertical communication possible Air will be mixed with the water in the hole and blown from the hole, potentially creating unwanted reactions with contaminants; may affect "representative" samples Cuttings and water blown from the hole can pose a hazard to crew and surrounding environment if toxic compounds encountered Organic foam additives to aid cuttings removal may contaminate samples
Air-percussion rotary or downhole-hammer	Air rotary with a reciprocating hammer connected to the bit to fracture rock	Very fast penetrations Useful in all geologic formations Only small amounts of water needed for dust and bit temperature control Cross-contamination potential can be reduced by driving casing	Relatively expensive As with most hydraulic rotary methods, the rig is fairly heavy, limiting accessibility Vertical mixing of water and air creates cross-contamination potential Hazard posed to surface environment if toxic compounds encountered Organic foam additives for cuttings removal may contaminate samples

Type of constituent	Example of constituent	Positive-displacement bladder pumps	Thief, in situ, or dual-check-valve bailers	Mechanical positive-displacement pumps	Gas-drive devices	Suction mechanisms	
		← Increasing reliability of sampling mechanisms →					
		← Increasing sample sensitivity →					
Volatile organic compounds	Chloroform TOX CH ₃ Hg	Superior performance for most applications	May be adequate if well purging is assured	May be adequate if design and operation are controlled	Not recommended	Not recommended	
Organometallics							
Dissolved gases	O ₂ , CO ₂	Superior performance for most applications	May be adequate if well purging is assured	May be adequate if design and operation are controlled	Not recommended	Not recommended	
Well-purging parameters	pH, Ω ⁻¹ Eh						
Trace inorganics metal species	Fe, Cu	Superior performance for most applications	May be adequate if well purging is assured	Adequate	May be adequate	May be adequate if materials are appropriate	
Reduced species	NO ₂ ⁻ , S ⁻						
Major cations & Anions	Na ⁺ , K ⁺ , Ca ^{**} Mg ^{**} Cl ⁻ , SO ₄ ⁻	Superior performance for most applications	May be adequate if well purging is assured	Adequate	Adequate	Adequate	

FIGURE 9.27 Groundwater sampling sensitivity and reliability matrix (91).

TABLE 9.41 Generalized Groundwater Sampling Protocol (91)

Step	Goal	Recommendations
Hydrologic measurements	Establish nonpumping water level	Measure the water level to ± 0.01 ft (± 0.3 cm)
Well purging	Remove or isolate stagnant H ₂ O, which would otherwise bias representative sample	Pump water until well purging parameters (e.g., pH, T, Ω^{-1} Eh) stabilize to $\pm 10\%$ over at least two successive well volumes pumped
Sample collection	Collect samples at land surface or in well-bore with minimal disturbance of sample chemistry	Pumping rates should be limited to ~ 100 mL/min for volatile organics and gas-sensitive parameters
Filtration and preservation	Filtration permits determination of soluble constituents and is a form of preservation. It should be done in the field as soon as possible after collection	Filter: Trace metals, inorganic anions and cations, alkalinity. Do not filter: TOC, TOX, volatile organic compound samples; other organic compound samples only when required
Field determinations	Field analyses of samples will effectively avoid bias in determining parameters and constituents which do not store well; e.g., gases, alkalinity, pH	Samples for determining gases, alkalinity, and pH should be analyzed in the field if at all possible
Field blanks and standards	These blanks and standards will permit the correction of analytical results for changes which may occur after sample collection: preservation, storage, and transport	At least one blank and one standard for each sensitive parameter should be made up in the field on each day of sampling. Spiked samples are also recommended for good QA/QC
Sample storage and transport	Refrigerate and protect samples to minimize their chemical alteration prior to analysis	Observe maximum sample holding or storage periods recommended by the Agency. Documentation of actual holding periods should be carefully performed

TABLE 9.42 Subsurface Conditions and Contaminant Dispersion

Subsurface profile	Subsurface conditions		Contaminant dispersion		
	Zone characteristics	Immiscible contaminant	Contaminated soil	Dissolved contaminant	Contaminant gases
Ground surface	—	—	—	—	X
Zone of aeration					
Intermediate zone	(Gravitational water)	X	X	—	X
Capillary zone	Capillary water	X	X	X	—
Water table	—	—	—	—	—
Zone of saturation	Groundwater	X	X	X	—
Impervious strata	—	—	—	—	—
Confined aquifer	Groundwater	X	X	X	—

TABLE 9.43 Existing Sampling Points for Contaminant Migration Testing

Sample type	Sampling points
Gas	Underground structures Building substructures Storm sewers Sanitary sewers
Liquid	Groundwater Monitoring wells Residential water supply wells Public water supply wells Surface water Streams and lakes Storm sewers Sanitary sewers

magnetic conductivity geophysical survey techniques for detection of inorganic and some organic liquids. Instruments for the measurement of soil gases include ground probes, accumulator devices, surface flux chambers, and downhole flux chambers.

Corrective Actions

Once a sudden or existing leak is detected; the quantity and character of the leakage are known; and the contaminant migration is assessed, correction technologies are evaluated for use in initial correction action and, if necessary, in subsequent, permanent measures. Tables follow to summarize the options and alternatives for corrective action responses.

Corrective Action Technologies. Whether a release involves gasoline or other petroleum products from gas stations or tank farms or hazardous substances from other underground storage tanks, a number of corrective measures are available. Table 9.44 shows potentially applicable corrective action technologies relative to release volume and chemical characteristics.

In addition to the considerations relative to the actual contaminant released, corrective action decisions also require site-specific information. Applicable site data needs for potential technologies are presented in Table 9.45 (page 9.92).

Initial Response Actions. Generally, the first response action to a discovered release is to remove any remaining product from the leaking tank. Table 9.46 (page 9.96) lists potential situations and their associated initial response corrective actions. All initial response actions are intended to minimize immediate risk to human health and the environment and to stabilize the situation if permanent corrective actions must be planned and implemented.

Permanent Response Actions. When initial corrective actions do not adequately abate the leakage problem, permanent corrective action goals, objectives, and standards are established, and alternative technologies are studied and evaluated. Examples of permanent corrective actions for a variety of site-specific problems are shown in Table 9.47 (page 9.97). Permitting, design, implementation, and monitoring are the final steps in permanent response actions.

TABLE 9.44 Applications of Typical Corrective Action Technologies (61)

Technology	Small- to moderate-volume recent gasoline or petroleum release (gas station or tank farms)	Large-volume or long-term chronic gasoline or petroleum release (gas station or tank farms)	Release from tanks containing hazardous substances (organic)	Release from tanks containing hazardous substances (inorganic)
Removal and excavation of tank, soil, and sediment				
Tank removal	•	•	•	•
Soil excavation	•	•	•	•
Sediment removal		•	•	•
On-site and off-site treatment and disposal of contaminants				
Solidification or stabilization		•	•	•
Landfilling				
Landfarming	•	•	•	
Soil washing				
Thermal destruction		•	•	
Aqueous waste treatment	•	•	•	•
Deep-well injection				
Free product recovery				
Dual pump systems	•	•	•	•
Floating filter pumps	•	•	•	•
Surface oil and water separators	•	•	•	
Groundwater recovery systems				
Groundwater pumping	•	•	•	•
Subsurface drabs	•	•	•	•
Subsurface barriers				
Slurry walls		•	•	•
Grouting		•	•	•
Sheet piles				
Hydraulic barriers	•	•	•	•
In situ treatment				
Chemical treatment			•	•
Physical treatment				
Soil flushing	•	•	•	•
Biostimulation	•	•	•	•
Groundwater treatment				
Air stripping	•	•	•	
Carbon adsorption	•	•	•	
Biological treatment	•	•	•	

(continues)

TABLE 9.44 Applications of Typical Corrective Action Technologies (61) (continued)

Technology	Small- to moderate-volume recent gasoline or petroleum release (gas station or tank farms)	Large-volume or long-term chronic gasoline or petroleum release (gas station or tank farms)	Release from tanks containing hazardous substances (organic)	Release from tanks containing hazardous substances (inorganic)
Groundwater treatment				
Precipitation, flocculation, sedimentation				•
Dissolved air flotation				•
Granular media filtration				•
Ion-exchange resin adsorption				•
Oxidation–reduction				•
Neutralization				•
Steam stripping			•	
Reverse osmosis				•
Sludge dewatering				•
Vapor migration control, collection, and treatment				
Passive collection systems		•	•	
Active collection systems		•	•	
Ventilation of structures	•	•	•	
Adsorption			•	
Flaring				
Surface water and drainage controls				
Diversion and collection systems	•	•	•	•
Grading	•	•	•	•
Capping		•	•	•
Revegetation	•	•	•	•
Restoration of contaminated water supplies and sewer lines				
Alternative central water supplies		•	•	•
Alternative point-of-use water supplies	•	•	•	•
Treatment of central water supplies		•	•	•
Treatment of point-of-use water supplies		•	•	•
Replacement of water and sewer lines		•	•	•
Cleaning and restoration of water and sewer lines	•	•	•	•

LOW-LEVEL RADIOACTIVE WASTE

Low-level wastes (LLWs) are a special category of hazardous wastes. They are generated from a variety of commercial sources, including government and industrial research, nuclear power, diagnostic and therapeutic medical services, and manufacturing processes (Figure 9.28, page 9.98). Prior to the 1960s, the federal government was responsible for the disposal of LLW from commercial sources at federally owned facilities. In 1963, Congress allowed the development of privately operated disposal sites to accommodate the increasing volumes of wastes from commercial operations. However, in the mid-1970s, many of these commercial sites were either closed or operations curtailed. In some cases, poor management and lack of regulatory guidance were cited. In other instances, the sites were found to be geologically and environmentally unsuitable for containing the wastes.

LLW are generated throughout the United States (Table 9.48, page 9.99), and the management and proper disposal of these wastes became a critical issue. In 1981, the Nuclear Regulatory Commission developed new requirements for LLW management (94). Earlier, in 1980, Congress enacted the Low-Level Radioactive Waste Policy Act (Public Law 96-573). This act designates the individual states as the responsible level of government for low-level waste management and encouraged the development of regional compacts to implement this policy and to establish regional LLW disposal facilities. The act also provided that after January 1, 1986, compacts approved by Congress could restrict the use of these facilities to the disposal of LLW generated within the compact region.

The compacting process envisioned by Congress in the early 1980's has not been achieved. Indeed the politics of waste disposal have virtually Balkanized the United States. Currently, nine regional compacts have been formed but nine states are "unaffiliated," meaning that they have either chosen to develop their own LLW disposal sites, or have been ejected from a compact for failure to comply with requirements (95). Although considerable money and effort has been spent, it has not resulted in any new disposal sites and only the Richland, Washington, and Barnwell, South Carolina, sites remain to serve commercial LLW generators. In part this is due to litigation by antinuclear activists, failure of State governments to take actions in the selection and approval of new facilities, and failure of the federal government to make timely land withdrawals needed to create new sites; however, the need for new disposal capacity has diminished over the past 20 years due to advances in treatment, volume reduction technologies, and waste minimization. Trends in low-level waste generation (Figure 9.29, page 9.100) indicate that fewer new sites are needed and therefore urgency felt in the late 1970's and early 1980's no longer prevails upon governments to take timely actions in addressing this public health issue.

The basic management principles and concepts for LLW are similar to those for hazardous chemicals. Many of the same concerns for protection of workers, the public, and the environment are used in planning for radioactive waste treatment and disposal. The major differences between managing chemical and radioactive waste lies in the recognition of hazards associated with radiation and knowledge of the applicable regulations. The hazards from exposure (both internal and external) to radiation are fairly well understood, and protection requirements have been developed. These requirements have been incorporated into regulations governing LLW management, making the management of LLW sometimes far simpler than those for hazardous chemicals.

This section provides basic information on radiation protection and regulations governing the management of LLW. Many of the treatment and disposal concepts for other hazardous wastes are applicable to LLW (93–116).

Principles of Radiation Protection

In order to understand the basis for the formulation of LLW regulatory requirements and guidelines, one must be familiar with the principles of radiation protection, including major terms, concepts, and calcula-

TABLE 9.45 Site Information for Use in Evaluating Corrective Action Alternatives (61, 92)

Technology	Geographic and topographic characteristics							Land and water use patterns				
	Precipitation	Temperature	Evapotranspiration	Topography	Accessibility	Site size	Proximity to surface water	Proximity to human interfaces	Current water-use patterns	Future water-use patterns	Current land-use patterns	Growth projections
Removal/excavation of tank, soil and sediment												
<i>Tank removal</i>		•		•	•							
<i>Soil excavation</i>		•		•	•							
Sediment removal		•		•	•							
On-site and off-site treatment and disposal of contaminants												
Solidification and stabilization	•	•	•	•	•	•						
Landfilling	•		•	•	•	•	•		•		•	
<i>Landfarming</i>	•		•	•	•	•	•	•	•	•	•	
Soil washing	•		•	•	•	•	•	•	•	•	•	•
Thermal destruction	•	•	•	•	•	•	•	•	•	•	•	•
<i>Aqueous waste treatment</i>	•	•	•	•	•	•	•	•	•	•	•	•
Deep-well injection				•			•	•		•		
Free product recovery												
<i>Dual pump systems</i>	•				•							
<i>Floating filter pumps</i>	•				•							
<i>Surface oil and water separators</i>	•		•	•	•	•						
Groundwater recovery systems												
<i>Groundwater pumping</i>	•				•			•		•		•
<i>Subsurface drains</i>	•				•			•		•		•
Subsurface barriers												
Slurry walls	•	•		•	•	•		•	•	•	•	•
Grouting	•	•		•	•	•		•	•	•	•	•
Sheet piles	•			•	•	•		•	•	•	•	•
<i>Hydraulic barriers</i>	•	•		•	•	•		•	•	•	•	•
In situ treatment												
<i>Soil flushing</i>	•	•			•	•		•				
<i>Biostimulation</i>	•	•			•	•		•				
Chemical treatment	•	•			•	•		•				
Physical treatment	•	•			•	•		•				
Groundwater treatment												
Air stripping	•	•	•	•	•	•						
Carbon adsorption	•		•	•	•	•						
Biological treatment	•		•	•	•	•						
Precipitation, flocculation, sedimentation	•		•	•	•	•						
Dissolved air flotation	•		•	•	•	•						
Granular media filtration	•		•	•	•	•						

Hydrogeologic characteristics	
Soil profiles	• • • • •
Soil physical properties	• • • • •
Soil chemical properties	• • • • •
Depth to bedrock	• • • • •
Depth to groundwater	• • • • •
Aquifer physical properties	• • • • •
Groundwater flow rate (volume)	• • • • •
Groundwater flow direction	• • • • •
Recharge areas	• • • • •
Recharge rates	• • • • •
Aquifer characteristics	• • • • •
Natural groundwater quality	• • • • •

(continues)

TABLE 9.45 Site Information for Use in Evaluating Corrective Action Alternatives (61, 92) (continued)

Technology	Geographic and topographic characteristics							Land and water use patterns			
	Precipitation	Temperature	Evapotranspiration	Topography	Accessibility	Site size	Proximity to surface water	Proximity to human interfaces	Current water-use patterns	Future water-use patterns	Current land-use patterns
Groundwater treatment											
Ion-exchange resin adsorption	•		•	•	•	•	•	•	•	•	•
Oxidation–reduction	•		•	•	•	•	•	•	•	•	•
Neutralization	•		•	•	•	•	•	•	•	•	•
Steam stripping	•		•	•	•	•	•	•	•	•	•
Reverse osmosis	•		•	•	•	•	•	•	•	•	•
Sludge dewatering	•		•	•	•	•	•	•	•	•	•
Vapor migration control, collection, and treatment											
<i>Passive collection systems</i>	•			•	•	•		•		•	•
<i>Active collection systems</i>	•			•	•	•		•		•	•
<i>Ventilation of structures</i>	•			•	•	•		•		•	•
Adsorption	•			•	•	•		•		•	•
Flaring											
Surface water and drainage controls											
<i>Diversion and collection systems</i>	•		•	•	•	•	•	•	•	•	•
<i>Grading</i>	•		•	•	•	•	•	•	•	•	•
<i>Capping</i>	•		•	•	•	•	•	•	•	•	•
<i>Revegetation</i>	•		•	•	•	•	•	•	•	•	•
Restoration of contaminated water supplies and sewer lines											
Alternative central water supplies	•	•	•	•	•	•	•	•	•	•	•
Alternative point-of-use water supplies	•	•	•	•	•	•	•	•	•	•	•
Treatment of central water supplies	•	•	•	•	•	•	•	•	•	•	•
Treatment of point-of-use water supplies	•	•	•	•	•	•	•	•	•	•	•
Replacement of water and sewer lines			•	•	•	•	•	•	•	•	•
Cleaning and restoration of water and sewer lines			•	•	•	•	•	•	•	•	•

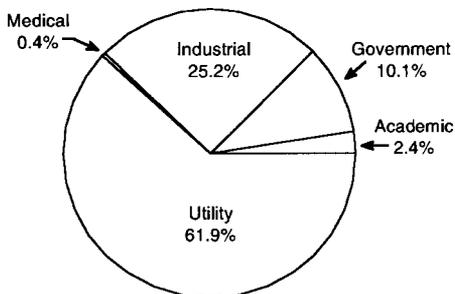
Note: Technologies in italics are likely to be used in response to UST releases at gasoline stations.

TABLE 9.47 Potential Site-Specific Problems and Associated Permanent Corrective Actions (39, 61)

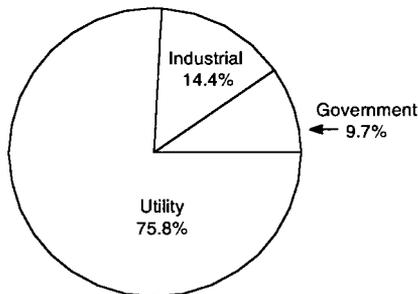
Situation	Corrective action categories										
	Removal or excavation of soil and sediments	On-site and off-site treatment and disposal of contaminants	Free product recovery	Groundwater recovery systems	Subsurface barriers	In situ treatment	Groundwater treatment	Vapor migration control, collection, and treatment	Surface water drainage controls	Restoration of contaminated water supplies and sewer lines	
Volatilization of chemicals into air Hazardous particulates released to atmosphere Dust generation by heavy construction or other site activities Contaminated site runoff Erosion of surface by water Surface seepage of released substance Flood hazard or contact of surface water body with released substance Released substance migrating vertically or horizontally High water table which may result in groundwater contamination or interfere with other corrective action Precipitation infiltrating site and accelerating released substance migration Explosive or toxic vapors migrating laterally underground Contaminated surface water, groundwater, or other aqueous or liquid waste Contaminated soils Toxic and/or explosive vapors that have been collected Contaminated stream banks and sediments Contaminated drinking water distribution system Contaminated utilities Free product in groundwater and soils	•							•	•	•	

Generator Category	Volume (ft ³)	Activity (curies)
Academic	9,994.41	48.54
Government	42,623.44	44,368.74
Industrial	106,562.83	65,833.52
Medical	1,628.52	8.83
Utility	261,204.04	345,950.19
Total	422,013.24	456,209.81

VOLUME PERCENTAGE BY SOURCE
(Percentages <.1% are not displayed)

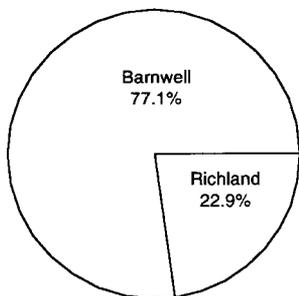


ACTIVITY PERCENTAGE BY SOURCE
(Percentages <.1% are not displayed)



Disposal Site	Volume (ft ³)	Activity (curies)
Barnwell	325,560.62	455,213.81
Richland	96,452.62	997.01
Total	422,103.24	456,210.82

VOLUME PERCENTAGE BY DISPOSAL SITE
(Percentages <.1% are not displayed)



ACTIVITY PERCENTAGE BY DISPOSAL SITE
(Percentages <.1% are not displayed)

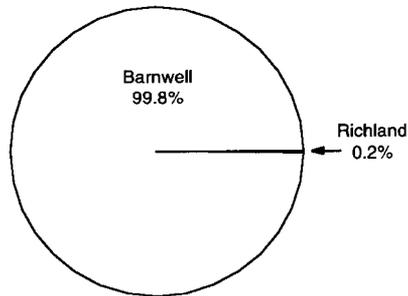


FIGURE 9.28 Low-level radioactive wastes received at commercial disposal sites in 1996 (93).

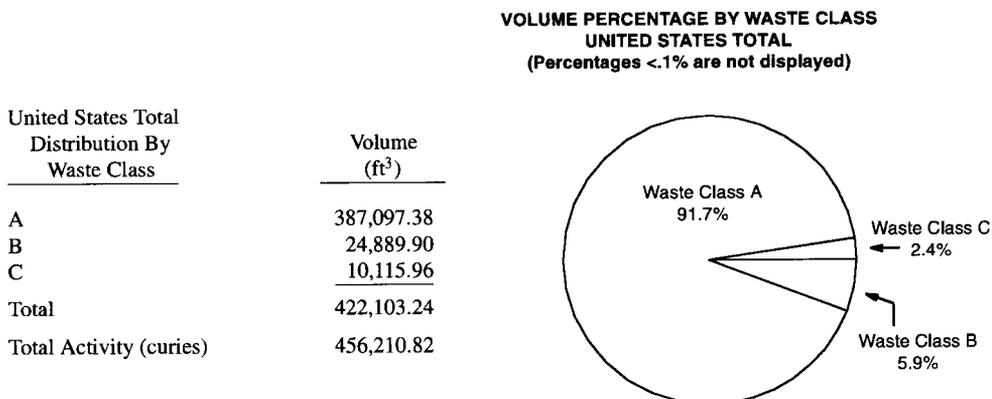


FIGURE 9.28 Low-level radioactive wastes received at commercial disposal sites in 1996 (93) (continued).

Table 9.48 Typical Low-level Waste Forms by Generator Categories (93)

Academic
Compacted trash or solids
Institutional laboratory or biological waste
Absorbed liquids
Animal carcasses
Government
Compacted trash or solids
Contaminated plant hardware
Absorbed liquids
Industrial
Depleted uranium
Compacted trash or solids
Contaminated plant hardware
Absorbed liquids
Sealed sources
Medical
Compacted trash or solids
Institutional laboratory or biological waste
Absorbed liquids
Sealed sources
Utilities
Spent resins
Evaporator bottoms and concentrated waste
Filter sludge
Dry compressible waste
Irradiated components
Contaminated plant hardware

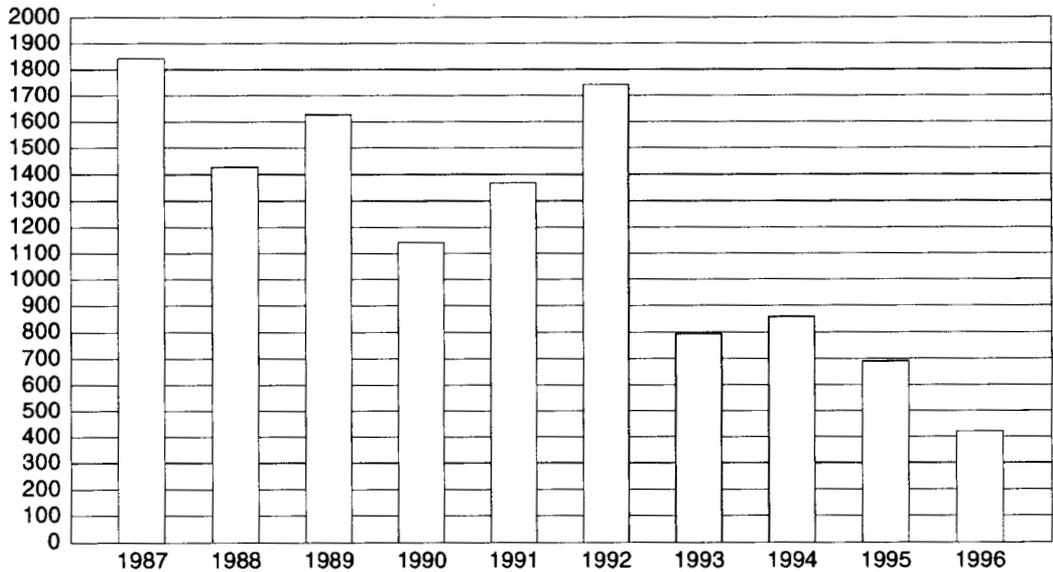
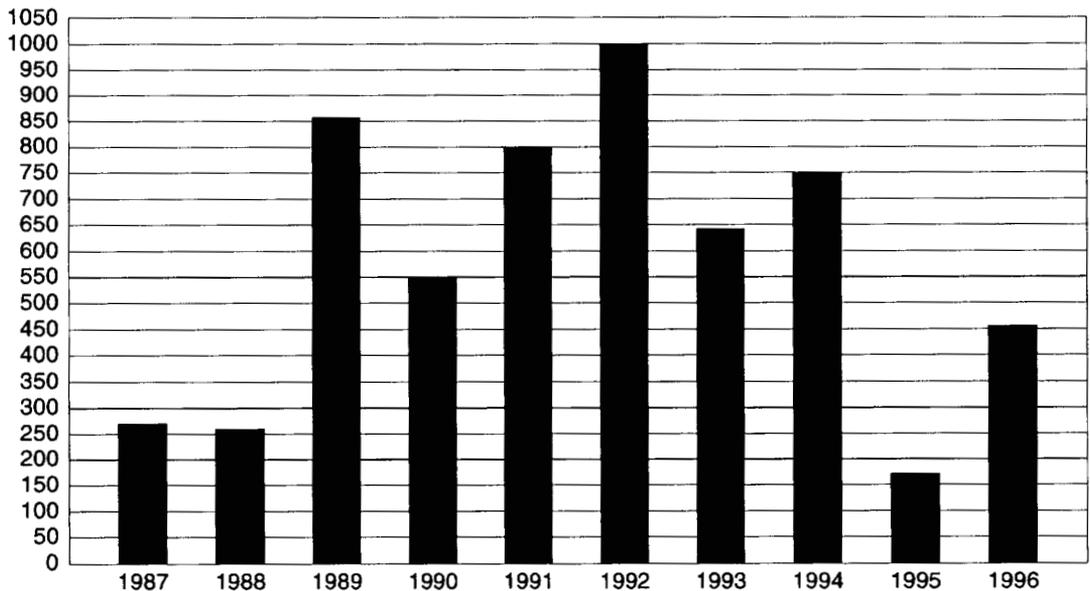
Volumes in Cubic Feet x 1000**Activities in Curies x 1000****FIGURE 9.29** Trends in U.S. low-level radioactive waste disposal (93).

TABLE 9.49 Radiation Protection Terms

Absorbed dose—the energy imparted by ionizing radiation per unit mass of irradiated material. The units of absorbed dose are the rad and the gray (Gy).

Activity—the rate of disintegration (transformation) or decay of radioactive material. The units of activity are the curie (Ci) and the becquerel (Bq).

ALARA (acronym for “as low as is reasonably achievable”)—means making every reasonable effort to maintain exposures to radiation as far below the dose limits as is practical, taking into account state of technology, cost, and other societal and socioeconomic factors.

Annual limit on intake (ALI)—the derived limit for the amount of radioactive material taken into the body of an adult worker by inhalation or ingestion in a year.

Background radiation—radiation from cosmic sources; naturally occurring radioactive material, including radon (except as decay product of source or special nuclear material); and global fallout as it exists in the environment from the testing of nuclear explosive devices or from past nuclear accidents.

Becquerel (Bq)—one becquerel = 1 disintegration per second

Collective dose—the sum of the individual doses received in a given period of time by a specified population from exposure to a specified source of radiation.

Committed dose equivalent (HT,50)—the dose equivalent to organs or tissues of reference (T) that will be received from an intake of radioactive material by an individual during the 50-year period following the intake.

Committed effective dose equivalent (HE,50)—the sum of the products of the weighting factors applicable to each of the body organs or tissues that are irradiated and the committed dose equivalent to these organs or tissues.

Curie—one curie = 3.7×10^{10} disintegrations per second = 3.7×10^{10} becquerels = 2.22×10^{12} disintegrations per minute.

Deep-dose equivalent (Hd), which applies to external whole-body exposure, is the dose equivalent at a tissue depth of 1 cm (1000 mg/cm^2).

Derived air concentration (DAC)—the concentration of a given radionuclide in air which, if breathed by the reference man for a working year of 2,000 hours under conditions of light work (inhalation rate 1.2 cubic meters of air per hour), results in an intake of one ALI.

Dose or radiation dose—a generic term that means absorbed dose, dose equivalent, effective dose equivalent, committed dose equivalent, committed effective dose equivalent, or total effective dose equivalent.

Dose equivalent (HT)—the product of the absorbed dose in tissue, quality factor, and all other necessary modifying factors at the location of interest. The units of dose equivalent are the rem and sievert (Sv).

Gray (Gy)—one gray is equal to an absorbed dose of 1 J/kg (100 rads).

High-radiation area—an area, accessible to individuals, in which radiation levels from radiation sources external to the body could result in an individual receiving a dose equivalent in excess of 0.1 rem (1 mSv) in 1 hour at 30 cm from the radiation source or 30 cm from any surface that the radiation penetrates.

Occupational dose—the dose received by an individual in the course of employment in which the individual’s assigned duties involve exposure to radiation or to radioactive material. Occupational dose does not include dose received from background radiation or from any medical administration the individual has received.

Public dose—the dose received by a member of the public from exposure to radiation or the release of radioactive materials. Public dose does not include occupational dose or doses received from background radiation, or from any medical administration the individual has received.

Quality Factor (Q)—a measure of potential damage caused to a tissue from various type and energies of radiation.

Rad—is the special unit of absorbed dose. One rad is equal to an absorbed dose of 100 ergs/gram or 0.01 J/kg (0.01 Gy).

Radiation area means—an area, accessible to individuals, in which radiation levels could result in an individual receiving a dose equivalent in excess of 0.005 rem (0.05 mSv) in 1 hour at 30 cm from the radiation source or from any surface that the radiation penetrates.

Rem (acronym for “Roentgen equivalent mammal”)—the dose equivalent in rems is equal to the absorbed dose in rads multiplied by the quality factor (1 rem = 0.01 sievert).

Sievert—the dose equivalent in sieverts is equal to the absorbed dose in grays multiplied by the quality factor (1 Sv = 100 rems).

Very high radiation area—an area, accessible to individuals, in which radiation levels from radiation sources external to the body could result in an individual receiving an absorbed dose in excess of 500 rads (5 Gy) in 1 hour at 1 m from a radiation source or 1 m from any surface that the radiation penetrates.

tion methods. The primary regulations governing radiation protection are found in 10 CFR Part 20, “Standard for Protection Against Radiation,” revised in March of 1991 to conform to nomenclature and standards of the International Commission on Radiation Protection (ICRP) in their ICRP Publication 26 (96) and Publication 30 (105). There are several Regulatory Guides issued by the Nuclear Regulatory Commission that provide the extensive and often complicated methods for assuring compliance with the regulation (115, 116, 118). This manual only provides an overview of radiation protection and reader is directed to the identified regulations and references for more in-depth information.

The degree of hazard associated with a particular radioactive material is based on the nuclide and the type and energy of radiation emitted. All radioactive particles are ionizing to a certain extent. As the radiation particle or ray passes through human tissue, it liberates electrons from the atoms it strikes, leaving the atom ionized. This may result in damage or death to the cells in the tissue.

There are four basic types of radiation:

Alpha Particles: These are the least penetrating but most energetic of radiation types. Composed of two protons and two neutrons (a helium nucleus), the alpha particle is positively charged and relatively massive. Because of its size, the alpha particle is easily stopped in a few centimeters of air and alpha-emitting wastes require little if any shielding. The danger from alpha-emitting nuclides comes from ingestion or inhalation, when the particle energy is transferred directly to adjacent cells.

Beta Particles: These are charged electrons and are more penetrating than alpha particles. Some beta particles can penetrate skin and travel up to several meters in air. Like alpha-emitting nuclides, the principal hazards result from ingestion or inhalation. Beta wastes usually require some level of shielding, depending upon concentration.

Gamma Radiation: This is the most penetrating of the radiation types. Composed of photons, gamma rays are similar to x-rays but more energetic. Gamma-emitting nuclides pose hazards from both internal and external sources and usually require some level of shielding.

Neutron Radiation: This is composed of high-energy neutral particles. Since neutrons have no charge, they can travel long distances in air and other materials. Along with gamma rays, neutrons present the greatest hazards for external exposure and require special shielding.

Radiation: Source and Effect. There are many sources of radiation in the environment to which humans are exposed on a daily basis. The principal ones are cosmic radiation and radiation from the naturally occurring radioactive nuclides of radium and radon. The average human living at sea level can expect to receive approximately 140 mrem per year in background radiation, of which 80 mrem is from cosmic radiation and 40 in rems is from natural radioactive material in the soil, water, air, and food chain (97).

The damage to humans from radiation depends on the energy transmitted, number of cells affected, and the types of cells involved. The common term for measuring damage is called *dose* and has units of rad or rem (Table 9.49).

For over 50 years, scientists have been studying the effects of radiation and radioactive materials on humans. Although there is still a considerable debate over the effects of low doses, there has been a substantial amount of data gathered on the effects of high doses of radiation. Table 9.50 presents the biological effect of radiation from gamma sources. It should be noted that while there are few noticeable effects from doses below 20 rem, it is assumed that a linear dose–response relationship exists, and, therefore, there will always be some measure of hazard associated with every level of exposure.

Radiation Protection Guidelines. Various commissions and agencies for radiation protection have defined guidelines for radiation workers. Exposure limits for radiation workers are higher than those allowed to the general population. In all cases, when dealing with radiation exposures, the guiding principles of “as low as reasonably achievable” (ALARA) should be used (99, 118, 132).

In most documents and radiation guides associated with LLW, dose values are reported as “whole-body”

TABLE 9.50 Biological Effects of Radiation (98)*

Whole-body radiation	
15–25 rem	Minimum exposure detectable from statistics of group blood counts
50 rem	Minimum exposure detectable from individual's body count
75 rem	Exposure likely to produce nausea in 10% of people
100 rem	Exposure likely to produce depilation in 10% of people
200 rem	Exposure causing disabling sickness in 90% of people
400–500 rem	Median lethal exposure
>600 rem	Lethal in 3–5 days from intestinal damage
>10,000 rem	Lethal in 24 h from central nervous system damage
Localized Effects	
Fetus	Significant probability of malformation if irradiated early in pregnancy: 10–20 rads
Gonads	Brief functional sterility: 50 rads
	Sterility for 1 to 2 years: 250 rads
	Permanent sterility: 600 rads
Eye	Change in optic lens: 200 rads
	Clinically significant cataract: 600 rads
Skin	Threshold erythema (100 keV): 300 rads
	Threshold erythema (1000 keV): 1000 rads
	Raw, moist area: 1500 rads Ulceration, slow healing: 5000–7000 rads

*The table gives the effects to humans that would be encountered at various exposure levels. The information applies only to x-rays or gamma radiation.

doses. This term refers to external gamma or x-ray radiation exposure to the entire body. Another commonly used term is effective dose equivalent (EDE), which incorporates exposures from internal and external sources to each body part and prorates the doses (depending on energy and tissue sensitivity) into a single value. Many of the reference documents and regulations currently in use have not yet been modified to address EDEs, and there remains some confusion in addressing individual and population dose requirements. However, for purposes of reporting compliance with 10 CFR Part 20, the EDE must be determined.

10 CFR Part 20 guidelines for occupational exposures for workers are:

- (1) An annual limit, which is the more limiting of (i) The total effective dose equivalent being equal to 5 rems (0.05 Sv); or (ii) The sum of the deep-dose equivalent and the committed dose equivalent to any individual organ or tissue other than the lens of the eye being equal to 50 rems (0.5 Sv).
- (2) The annual limits to the lens of the eye, to the skin, and to the extremities, which are: (i) A lens dose equivalent of 15 rems (0.15 Sv), and (ii) A shallow-dose equivalent of 50 rems (0.50 Sv) to the skin or to any extremity.

Radiation Protection Methods. A guiding principle behind all radioactive and hazardous materials management is “as low a reasonably achievable” (ALARA). Inherent in this philosophy is the assumption the risks cannot be completely eliminated, but every effort should be made to reduce exposures to the lowest possible extent.

There are three factors which determine the exposure to individuals handling radioactive materials:

- Time of exposure
- Distance from the source
- Amount of shielding present

In assessing exposures, both external and internal dose must be considered. External exposures require reducing the exposure rate through distance or shielding, or limiting the exposure time. Internal exposures result from the inhalation, ingestion, or injection of radioactive materials or absorption of contamination on the body.

External Exposures. While controlling external exposure times is an effective means of reducing doses, it is also the least preferred because it requires active monitoring and measurement. The simplest method is distance. Radiation levels decrease proportionally to the square of the distance ($1/r^2$). This is called the “inverse-square law” and holds true for point sources or in situations where the distance from the source is substantially further than the length or area of the source. For sources of different sizes and configurations, radiation patterns will follow different formulas.

Shielding is used to absorb or scatter the radiation before it reaches the receptor. Alpha particles have a very short range, even in air, and shielding against them is usually not required. For other types of radiation, shielding requirements will vary considerably (103, 104).

Internal Exposures. Estimating internal exposures requires knowledge of the nuclide, type of radiations emitted, organs affected, and biological half-life. Each radionuclide emits radiation at differing energy levels and rates. Both the type and energy of the radiation will determine the potential for damage. Certain nuclides tend to concentrate in various parts of the body and therefore will cause greater damage to those organs. Also, the rate at which the body eliminates the nuclides will determine the length of exposure. Internal dosimetry and dose calculations are complex and require specialized training (95, 105).

Nuclide Characteristics. Information about many of the nuclides commonly found in LLW and the nuclear industry is compiled in Table 9.51. This table is based on the earlier version of 10 CFR Part 20 calculation methods and therefore should not be used for purposes of insuring compliance with the regulation. However, for purposes of estimation and comparison, this table is a useful tool. An explanation of terms follows:

Isotope: The chemical symbol and atomic mass of the isotope are presented in the alphabetical order by element name.

Radiation: The typical radiations emitted for each isotope in the order of importance. Other radiations by be emitted by short-lived decay daughters. Isotopes with daughters of less than 25 years half-life are noted with the symbol D.

MPC Air: The maximum permissible concentration (MPC) in air for workers is based on an assumed 40-h work week. MPCs for air for public exposure range approximately 10 to 20 times less than for workers.

MPC Water: The MPC for public drinking water are based on a normal consumption of 2 liters (0.5 gallons) per day.

MPBB: The maximum permissible body burden (MPBB) is the amount of the isotope maintained in the body which will result in the maximum allowable exposure to the organ(s) of concern. The values listed are for isotope absorbed in the bloodstream and deposited in the body.

C.O.: The critical organ (C.O.) is the organ that would receive the highest allowable dose from an MPBB of the isotope. The abbreviations listed are:

T.B.—total body	Pan.—pancreas	Kid.—kidney	Spl.—spleen
B.T.—body tissue	Pros.—prostate	Thy.—thyroid	Tes.—testes

Radioactive Half-life: The time required for one-half of an isotope’s disintegrations to occur.

Biological Half-life: The time required for one-half of the isotope to be removed from the critical organ by biological processes. When combined with the radioactive half-life, the “effective” half-life can be calculated by:

TABLE 9.51 Nuclide Information (97)

Isotope	Radiations	MPC air workers ($\mu\text{Ci/cc}$)	MPC water public, $\mu\text{Ci/cc}$	MPBB, μCi	C.O.	Half-life		Ci/g	Rhm per Ci^*
						Radioactive	Biological days		
Ac-227	α, β^-, γ D	2×10^{-12}	2×10^{-6}	0.03	Bone	21.6 y	73,000	72.9	—
Ac-228	β^-, γ D	2×10^{-8}	9×10^{-5}	0.04	Bone	6.13 h	73,000	2.24×10^{-6}	0.26
Am-241	α, γ γ	6×10^{-12}	4×10^{-6}	0.05	Bone	458 y	73,000	3.24	0.01
Am-242m	x, α D	6×10^{-12}	4×10^{-6}	0.07	Bone	152 y	73,000	9.72	—
Am-242	β^-, x D	4×10^{-8}	1×10^{-4}	0.06	Liver	16.01 h	3,480	8.09×10^5	0
Am-243	α, γ D	6×10^{-12}	4×10^{-6}	0.05	Bone	7.95×10^3 y	73,000	0.185	0.02
Am-244	β^-, γ D	4×10^{-6}	5×10^{-3}	0.2	Bone	10.1 h	73,000	1.27×10^6	0.42
Sb-122	β^-, γ	1×10^{-7}	3×10^{-5}	20	T.B.	2.80 d	38	3.82×10^5	0.23
Sb-124	β^-, γ	2×10^{-8}	2×10^{-5}	10	T.B.	60.4 d	38	1.74×10^4	0.98
Sb-125	β^-, γ D	3×10^{-8}	1×10^{-4}	40	Lung	2.71 y	100	1060	0.24
Ar-37	x	6×10^{-3}				35.1 d		1.01×10^5	0
Ar-41	β^-, γ	2×10^{-6}				1.83 h		4.18×10^7	0.65
As-73	γ D	4×10^{-7}	5×10^{-4}	300	T.B.	80.3 d	280	2.23×10^4	0.003
As-74	β^-, β^+, γ	1×10^{-7}	5×10^{-5}	40	T.B.	17.9 d	280	9.86×10^4	0.42
As-76	β^-, γ	1×10^{-7}	2×10^{-5}	20	T.B.	26.4 h	280	1.56×10^6	0.20
As-77	β^-, γ D	4×10^{-7}	8×10^{-5}	80	T.B.	38.7 h	280	1.05×10^6	0.006
At-211	α, x, D	7×10^{-9}	2×10^{-6}	0.02	Thy.	7.21 h	36	2.06×10^6	—
Ba-131	γ D	4×10^{-7}	2×10^{-4}	50	T.B.	12.0 d	65	8.31×10^4	0.22
Ba-140	β^-, γ D	4×10^{-8}	2×10^{-5}	4	Bone	12.80 d	65	7.29×10^4	0.14
Bk-249	$\beta^-, \gamma, \text{n}$ D	9×10^{-10}	6×10^{-4}	0.7	Bone	314 d	73,000	1670	<0.001
Bk-250	β^-, γ D	1×10^{-7}	2×10^{-4}	0.05	Bone	193.3 m	73,000	3.89×10^6	0.47
Be-7	γ	1×10^{-6}	2×10^{-3}	600	T.B.	53.6 d	280	3.48×10^5	0.03
Bi-206	e^-, γ D	1×10^{-7}	4×10^{-5}	1	Kid.	6.243 d	6	1.02×10^5	1.7
Bi-207	e^-, γ D	1×10^{-8}	6×10^{-5}	2	Kid.	30.2 y	6	54.2	0.82
Bi-210	β^-, α	6×10^{-9}	4×10^{-5}	0.04	Kid.	5.013 d	6	1.24×10^5	<0.001
Bi-212	α, β^-, γ D	1×10^{-7}	4×10^{-4}	0.02	Kid.	60.60 m	6	1.46×10^7	0.051
Br-82	β^-, γ	2×10^{-7}	4×10^{-5}	10	T.B.	35.34 h	8	1.08×10^6	1.3

(continued)

TABLE 9.51 Nuclide Information (97) (continued)

Isotope	Radiations	MPC air workers ($\mu\text{Ci/cc}$)	MPC water public, $\mu\text{Ci/cc}$	MPBB, μCi	C.O.	Half-life			Rhm per Ci^*
						Radioactive	Biological days	Ci/g	
Cd-109	e^-, γ, D	5×10^{-8}	2×10^{-4}	20	Liver	453d	200	2640	0
Cd-115m	β^-, γ	4×10^{-8}	3×10^{-5}	3	Liver	43 d	200	2.60×10^4	0.017
Cd-us	β^-, γ, D	2×10^{-7}	3×10^{-5}	3	Liver	53.5h	200	5.09×10^5	0.11
Ca-45	β^-	3×10^{-8}	9×10^{-6}	30	Bone	165 d	18,000	1.76×10^4	0
Ca-47	β^-, γ, D	2×10^{-7}	3×10^{-5}	5	Bone	4.535 d	18,000	6.13×10^5	0.54
Cf-249	α, γ, n	2×10^{-12}	4×10^{-6}	0.04	Bone	360 y	73,000	4.0	0.19
Cf-250	α, n, x	5×10^{-12}	1×10^{-5}	0.04	Bone	13.2 y	73,000	108	—
Cf-251	α, γ	2×10^{-12}	4×10^{-6}	0.04	Bone	~800 y	73,000	2	—
Cf-252	α, n, x	2×10^{-11}	2×10^{-5}	0.01	Bone	2.646 y	73,000	536	—
Cf-253	β^-, α, D	8×10^{-10}	1×10^{-4}	0.04	Bone	17.6 d	73,000	2.93×10^4	—
Cf-254	α, n	5×10^{-12}	1×10^{-7}	7×10	Bone	60.5 d	73,000	8510	—
C-14	β^-	4×10^{-6}	8×10^{-4}	300	Fat	5730 y	12	4.46	0
Ce-139	e^-, γ, D	$2 \times 10^{-7\ddagger}$	$3 \times 10^{-4\ddagger}$	100 [†]	Liver [†]	140 d	293	6700	0.065
Ce-141	β^-, γ, D	2×10^{-7}	9×10^{-5}	30	Liver	32.5 d	293	2.85×10^4	0.033
Ce-143	β^-, γ, D	2×10^{-7}	4×10^{-5}	7	Liver	33 h	293	6.6×10^5	0.15
Ce-144	β^-, γ, D	6×10^{-9}	1×10^{-5}	5	Bone	284 d	1500	3190	0.008
Cs-131	x	3×10^{-6}	9×10^{-4}	700	T.B.	9.70 d	70	1.03×10^5	0
Cs-134m	e^-, γ, β^-, D	6×10^{-6}	1×10^{-3}	100	T.B.	2.895 h	70	8.08×10^6	0.008
Cs-134	β^-, γ	1×10^{-8}	9×10^{-6}	20	T.B.	2.046 y	70	1300	0.88
Cs-135	β^-	9×10^{-8}	1×10^{-4}	200	Liver	3.0×10^6 y	90	8.8×10^{-4}	0
Cs-136	β^-, γ, D	2×10^{-7}	6×10^{-5}	30	T.B.	13.7 d	70	7.01×10^4	1.2 [†]
Cs-137	β^-, γ, D	1×10^{-8}	2×10^{-5}	30	T.B.	30.0 y	70	87.0	0.32 [§]
Cs-138	β^-, γ	$4 \times 10^{-5\ddagger}$	2×10^{-5}	30	T.B.	32.2 m	70	4.2×10^7	1.0
Cl-36	β^-, β^+	2×10^{-8}	6×10^{-5}	80	T.B.	3.08×10^5 y	29	0.0323	<0.001
Cl-38	β^-, γ	2×10^{-6}	4×10^{-4}	9	T.B.	37.29 m	29	1.33×10^8	0.75

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Cr-51	e ⁻ , γ	2 × 10 ⁻⁶	2 × 10 ⁻³	800	T.B.	27.8 d	616	9.21 × 10 ⁴	0.018 [§]
Co-57	e ⁻ , γ	2 × 10 ⁻⁷	4 × 10 ⁻⁴	200	T.B.	270 d	9.5	8480	0.093
Co-58m	e ⁻ , x D	9 × 10 ⁻⁶	2 × 10 ⁻³	200	T.B.	9.2 h	9.5	5.9 × 10 ⁶	0
Co-58	β ⁺ , γ	5 × 10 ⁻⁸	9 × 10 ⁻⁵	30	T.B.	71.3 d	9.5	3.16 × 10 ⁴	0.54
Co-60	β ⁻ , γ	9 × 10 ⁻⁹	3 × 10 ⁻⁵	10	T.B.	5.263 y	9.5	1133	1.3 [§]
Cu-64	β ⁻ , β ⁺ , γ	1 10 ⁻⁶	2 × 10 ⁻⁴	10	Spl.	12.80h	2	3.82 × 10 ⁶	0.11
Cm-242	α, n, γ	1 × 10 ⁻¹⁰	2 × 10 ⁻⁵	0.05	Liver	162.5 d	3,000	3320—	—
Cm-243	α, γ	6 × 10 ⁻¹²	5 × 10 ⁻⁶	0.09	Bone	32 y	73,000	46	0.041
Cm-244	α, n, γ	9 × 10 ⁻¹²	7 × 10 ⁻⁶	0.1	Bone	17.6 y	73,000	83.3	—
Cm-245	α, γ D	5 × 10 ⁻¹²	4 × 10 ⁻⁶	0.04	Bone	9.3 × 10 ³	73,000	0.16	0.015
Cm-246	α, n, x	5 × 10 ⁻¹²	4 × 10 ⁻⁶	0.05	Bone	5.5 × 10 ³ y	73,000	0.26	0
Cm-247	α D	5 × 10 ⁻¹²	4 × 10 ⁻⁶	0.04	Bone	1.6 × 10 ⁵	73,000	9.1 × 10 ⁻⁵	—
Cm-248	α, n, x	6 × 10 ⁻¹³	4 × 10 ⁻⁷	0.005	Bone	4.7 × 10 ³ y	73,000	3.1 × 10 ⁻³	—
Cm-249	β ⁻ D	1 × 10 ⁻⁵	2 × 10 ⁻³	1	Bone	64 m	73,000	1.2 × 10 ⁷	—
Dy-165	β ⁻ , γ	2 × 10 ⁻⁶	4 × 10 ⁻⁴	10	Bone	139.2 m	1000	8.18 × 10 ⁶	—
Dy-166	β ⁻ , γ D	2 × 10 ⁻⁷	4 × 10 ⁻⁵	5	Bone	81.5 h	1000	2.32 × 10 ⁵	0.007
Es-253	α, γ, n D	6 × 10 ⁻¹⁰	2 × 10 ⁻⁵	0.04	Bone	20.47 d	73,000	2.52 × 10 ⁴	—
Es-254m	β ⁻ , γ, n D	5 × 10 ⁻⁹	2 × 10 ⁻⁵	0.02	Bone	39.3 h	73,000	3.14 × 10 ⁵	0.26
Es-254	α, γ, n D	2 × 10 ⁻¹¹	1 × 10 ⁻⁵	0.02	Bone	276 d	73,000	1860	0.001
Es-255	β ⁻ , α, n D	4 × 10 ⁻¹⁰	3 × 10 ⁻⁵	0.04	Bone	38.3 d	73,000	1.34 × 10 ⁴	—
Er-169	β ⁻ , γ D	4 × 10 ⁻⁷	9 × 10 ⁻⁵	30	Bone	9.6 d	1000	8.0 × 10 ⁴	0.004
Er-171	β ⁻ , γ D	6 × 10 ⁻⁷	1 × 10 ⁻⁴	9	Bone	7.52 h	1000	2.44 × 10 ⁶	—
Eu-152m	β ⁻ , β ⁺ , γ D	3 × 10 ⁻⁷	6 × 10 ⁻⁵	8	Liver	9.3 h	127	2.2 × 10 ⁶	0.15
Eu-152	β ⁻ , β ⁺ , γ D	1 × 10 ⁻⁸	8 × 10 ⁻⁵	20	Kid.	12.7 y	1480	185 0.53	—
Eu-154	β ⁻ , γ	4 × 10 ⁻⁹	2 × 10 ⁻⁵	5	Kid.	16 y	1480	145	0.63
Eu-155	β ⁻ , γ	7 × 10 ⁻⁸	2 × 10 ⁻⁵	70	Kid.	1.811 y	1480	1270	0.021

(continues)

TABLE 9.51 Nuclide Information (97) (continued)

Isotope	Radiations	MPC air workers ($\mu\text{Ci/cc}$)	MPC water public, $\mu\text{Ci/cc}$	MPBB, μCi	C.O.	Half-life			Rhm per Ci^*
						Radioactive	Biological days	Ci/g	
Fm-254	n, α , x D	6×10^{-8}	1×10^{-4}	0.02	Bone	3.24 h	73,000	3.81×10^6	—
Fm-255	α , n, γ	1×10^{-8}	3×10^{-5}	0.04	Bone	20.1 h	73,000	6.11×10^5	<0.001
Fm-256	n, α	2×10^{-9}	9×10^{-7}	8×10^{-4}	Bone	2.7 h	73,000	4.5×10^6	—
F.P.	β^- , β^+ , γ	$3 \times 10^{-9\text{H}}$							—
F-18	β^+ , γ	3×10^{-6}	5×10^{-4}	20	T.B.	109.7 m	808	9.52×10^7	0.56
Gd-153	e^- , γ	9×10^{-8}	2×10^{-4}	90	Bone	242 d	1000	3530	0.024
Gd-159	β^- , γ D	4×10^{-7}	8×10^{-5}	20	Bone	18.0 h	1000	1.09×10^6	0.019
Ga-72	β^- , γ	2×10^{-7}	4×10^{-5}	5	Liver	14.12 h	4.8	3.08×10^6	1.2
Ge-71	x	6×10^{-6}	2×10^{-3}	100	Kid.	11.4 d	12	1.61×10^6	0
Au-196	β^- , γ	6×10^{-7}	1×10^{-4}	40	T.B.	6.18 d	120	1.08×10^5	0.24
Au-198	β^- , γ	2×10^{-7}	5×10^{-5}	20	Kid.	2.697 d	280	2.44×10^5	0.23 [§]
Au-199	β^- , γ	8×10^{-7}	2×10^{-4}	70	Kid.	3.15 d	280	2.08×10^5	0.037
Hf-181	p, γ	4×10^{-8}	7×10^{-5}	4	Spl.	42.5 d	350	1.70×10^4	0.27
Ho-166	β^- , γ	2×10^{-7}	3×10^{-5}	5	Bone	26.9 h	1000	7.02×10^5	0.011
H-3	β^-	5×10^{-6}	3×10^{-3}	1000	B.T.	12.262 y	12	9720	0
In-113m	e^- , γ	7×10^{-6}	1×10^{-3}	30	Kid.	99.8 m	60	1.66×10^7	0.14
In-114m	e^- , γ D	2×10^{-8}	2×10^{-5}	6	Kid.	50.0 d	60	2.29×10^4	0.042
In-115m	β^- , γ	2×10^{-6}	4×10^{-4}	30	Kid.	4.50 h	60	6.05×10^6	0.091
In-115	β^-	3×10^{-8}	9×10^{-5}	30	Kid.	6×10^{14} y	60	5×10^{-12}	0
I-125	e^- , γ	5×10^{-9}	2×10^{-7}			60.2 d	138	1.73×10^4	0.004
I-126	β^- , β^+ , γ	8×10^{-9}	3×10^{-7}	1	Thy.	12.8 d	138	8.09×10^4	0.20
I-129	β^- , γ	2×10^{-9}	6×10^{-8}	3	Thy.	1.7×10^{-7} y	138	1.60×10^{-4}	0.004

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I-131	β^- , γ D	9×10^{-9}	3×10^{-7}	0.7	Thy.	8.05 d	138	1.24×10^5	0.21
I-132	β^- , γ	2×10^{-7}	8×10^{-6}	0.3	Thy.	2.26h	138	1.05×10^7	1.3
I-133	β^- , γ D	3×10^{-8}	1×10^{-6}	0.3	Thy.	20.3 h	138	1.16×10^6	0.27
I-134	β^- , γ	5×10^{-7}	2×10^{-5}	0.2	Thy.	52.0 m	138	2.70×10^7	1.1
I-135	β^- , γ D	1×10^{-7}	4×10^{-6}	0.3	Thy.	6.68 h	138	3.47×10^6	0.91
Ir-190	e^- , γ	4×10^{-7}	2×10^{-4}	40	Liver	11 d	27	6.2×10^4	—
Ir-192	β^- , γ	3×10^{-8}	4×10^{-5}	6	Kid.	74.2 d	50	9160	0.5 [§]
Ir-194	β^- , γ	2×10^{-7}	3×10^{-5}	7	Kid.	17.4 h	50	9.28×10^5	—
Fe-55	γ	9×10^{-7}	8×10^{-4}	1000	Spl.	2.60 y	600	2500	—
Fe-59	β^- , γ	5×10^{-9}	5×10^{-5}	20	Spl.	45.6 d	600	4.85×10^4	0.63
Kr-85m	β^- , γ D	6×10^{-6}				4.4 h		8.4×10^6	0.075
Kr-85	β^- , γ	1×10^{-5}				10.76 y		391	0.001
Kr-87	β^- , γ	1×10^{-6}				76 m		2.8×10^7	0.64
Kr-88	β^- , γ D	1×10^{-6}				2.80 h		1.27×10^7	—
La-140	β^- , γ	1×10^{-7}	2×10^{-5}	9	Liver	40.22 h	400	5.56×10^5	1.1
Pb-203	e^- , γ	2×10^{-6}	4×10^{-4}	30	Kid.	52.1 h	531	2.96×10^5	0.14
Pb-210	β^- , γ D	1×10^{-10}	1×10^{-7}	0.4	Kid.	20.4 y	531	83.5	0.002
Pb-212	β^- , γ D	2×10^{-8}	2×10^{-5}	0.02	Kid.	10.64 h	531	1.39×10^6	0.064
Lu-177	β^- , γ	5×10^{-7}	1×10^{-4}	20	Bone	6.74 d	1000	1.09×10^5	0.008
Mn-52	β^+ , γ	1×10^{-7}	3×10^{-5}	5	Pan.	5.60 d	5.7	4.48×10^5	1.7
Mn-54	e^- , γ	4×10^{-8}	1×10^{-4}	20	Liver	303 d	25	7980	0.46
Mn-56	β^- , γ	5×10^{-7}	1×10^{-4}	2	Pan.	2.576 h	5.7	2.17×10^7	0.85
Hg-197m	e^- , γ Δ	7×10^{-7}	2×10^{-4}	4	Kid.	24 h	14.5	6.6×10^5	0.037 [†]
Hg-197	e^- , γ	1×10^{-6}	3×10^{-4}	20	Kid.	65 h	14.5	2.4×10^5	0.008
Hg-203	β^- , γ	7×10^{-8}	2×10^{-5}	4	Kid.	46.9 d	14.5	1.37×10^4	0.12
Mo-99	β^- , γ D	2×10^{-7}	4×10^{-5}	8	Kid.	66.7 h	3	4.75×10^5	0.076

(continues)

TABLE 9.51 Nuclide Information (97) (continued)

Isotope	Radiations	MPC air workers (μCi/cc)	MPC water public, μCi/cc	MPBB, μCi	C.O.	Half-life		Ci/g	Rhm per Ci*
						Radioactive	Biological days		
Nd-144	α	8 × 10 ⁻¹¹	7 × 10 ⁻⁵	0.1	Bone	2.4 × 10 ¹⁵ y	1500	1.0 × 10 ⁻¹²	0
Nd-147	β ⁻ , γ D	2 × 10 ⁻⁷	6 × 10 ⁻⁵	10	Liver	11.06 d	131	8.03 × 10 ⁴	0.064
Nd-149	β ⁻ , γ D	1 × 10 ⁻⁶	3 × 10 ⁻⁴	3	Liver	1.8 h	131	1.2 × 10 ⁷	0.17
Np-237	α, γ D	4 × 10 ⁻¹²	3 × 10 ⁻⁶	0.06	Bone	2.14 × 10 ⁶ y	73,000	7.05 × 10 ⁻⁴	0.017
Np-239	β ⁻ , γ	7 × 10 ⁻⁵	1 × 10 ⁻⁴	30	Bone	2.346 d	73,000	2.33 × 10 ⁵	0.050
Ni-59	β ⁻ , γ	5 × 10 ⁻⁷	2 × 10 ⁻⁴	1000	Bone	8 × 10 ⁴ y	800	0.08	—
Ni-63	γ	6 × 10 ⁻⁸	3 × 10 ⁻⁵	200	Bone	92 y	800	62	0
Ni-65	β ⁻	5 × 10 ⁻⁷	1 × 10 ⁻⁴	4	Bone	2.564 h	800	1.88 × 10 ⁷	0.29
Nb-93m	e ⁻ , x	1 × 10 ⁻⁷	4 × 10 ⁻⁴	200	Bone	13.6 y	1000	283	0
Nb-95	β ⁻ , γ	1 × 10 ⁻⁷	1 × 10 ⁻⁴	40	T.B.	35.0 d	760	3.93 × 10 ⁴	0.43
Nb-97	β ⁻ , γ	5 × 10 ⁻⁶	9 × 10 ⁻⁴	10	Bone	72 m	1000	2.7 × 10 ⁷	0.36
Os-185	e ⁻ , γ	5 × 10 ⁻⁸	7 × 10 ⁻⁵	8	Kid.	93.6 d	5	7540	0.36
Os-191m	e ⁻ , γ x D	9 × 10 ⁻⁶	2 × 10 ⁻³	100	Kid.	13.0 h	5	1.26 × 10 ⁶	0
Os-191	β ⁻ , γ D	4 × 10 ⁻⁷	2 × 10 ⁻⁴	20	Kid.	15.0 d	5	4.56 × 10 ⁴	0.015
Os-193	β ⁻ , γ	3 × 10 ⁻⁷	5 × 10 ⁻⁵	10	Kid.	31.5 h	5	5.15 × 10 ⁵	0.028
Pd-103	γ D	7 × 10 ⁻⁷	3 × 10 ⁻⁴	20	Kid.	17.0 d	30	7.46 × 10 ⁴	<0.001
Pd-109	β ⁻ , γ D	4 × 10 ⁻⁷	7 × 10 ⁻⁵	7	Kid.	13.47 h	30	2.13 × 10 ⁶	0.002‡
P-32	β ⁻	7 × 10 ⁻⁸	2 × 10 ⁻⁵	6	Bone	14.28 d	1155	2.86 × 10 ⁵	0
Pt-191	e ⁻ , γ	6 × 10 ⁻⁷	1 × 10 ⁻⁴	10	Kid.	3.00 d	60	2.28 × 10 ⁵	0.054
Pt-193m	e ⁻ , x	5 × 10 ⁻⁶	1 × 10 ⁻³	100	Kid.	4.3 d	60	1.6 × 10 ⁵	0
Pt-197m	β ⁻ , γ D	5 × 10 ⁻⁶	9 × 10 ⁻⁴	5	Kid.	78 m	60	1.2 × 10 ⁷	0.028
Pt-197	β ⁻ , γ	6 × 10 ⁻⁷	1 × 10 ⁻⁴	10	Kid.	18 h	60	8.8 × 10 ⁵	0.013
Pu-238	α, γ	2 × 10 ⁻¹²	5 × 10 ⁻⁶	0.04	Bone	86.4 y	73,000	17.4	<0.001
Pu-239	α, γ	2 × 10 ⁻¹²	5 × 10 ⁻⁶	0.04	Bone	24,390 y	73,000	0.0614	<0.001
Pu-240	α, γ	2 × 10 ⁻¹²	5 × 10 ⁻⁶	0.04	Bone	6580 y	73,000	0.226	<0.001
Pu-241	β ⁻ , γ	9 × 10 ⁻¹¹	2 × 10 ⁻⁴	0.9	Bone	13.2 y	73,000	112	<0.001
Pu-242	α, x	2 × 10 ⁻¹²	5 × 10 ⁻⁶	0.05	Bone	3.79 × 10 ⁵ y	73,000	3.90 × 10 ⁻³	0
Pu-243	β ⁻ , γ	2 × 10 ⁻⁶	3 × 10 ⁻⁴	7	Bone	4.98 h	73,000	2.59 × 10 ⁶	0.009
Pu-244	α D	2 × 10 ⁻¹²	4 × 10 ⁻⁶	0.04	Bone	7.6 × 10 ⁷ y	73,000	1.9 × 10 ⁻⁵	—

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Po-210	α, γ	2×10^{-10}	7×10^{-7}	0.03	Spl.	138.40 d	60	4490	<0.001
K-42	β^-, γ	1×10^{-7}	2×10^{-5}	10	T. B.	12.36 h	58	6.04×10^6	0.14
Pr-142	β^-, γ	2×10^{-7}	3×10^{-5}	7	Bone	19.2 h	1500	1.15×10^6	0.028
Pr-143	β^-	2×10^{-7}	5×10^{-5}	20	Bone	13.59 d	1500	6.72×10^4	0
Pm-147	β^-	6×10^{-8}	2×10^{-4}	60	Bone	2.62 y	1500	929	0
Pm-149	β^-, γ	2×10^{-7}	4×10^{-5}	20	Bone	53.1 h	1500	396×10^5	0.004
Pa-230	$\beta^-, \alpha, \gamma, D$	8×10^{-10}	2×10^{-4}	0.07	Bone	17.7 d	73,000	3.21×10^4	0.45
Pa-231	α, γ, D	1×10^{-12}	9×10^{-7}	0.02	Bone	3.25×10^4 y	73,000	0.0476	0.016
Pa-233	β^-, γ	2×10^{-7}	1×10^{-4}	40	Kid.	27.0 d	51,000	2.08×10^4	0.075
Ra-223	α, γ, D	2×10^{-10}	7×10^{-7}	0.05	Bone	11.435 d	16,400	5.12×10^4	—
Ra-224	α, γ, D	7×10^{-10}	2×10^{-6}	0.06	Bone	3.64 d	16,400	1.60×10^5	—
Ra-226	α, γ, D	3×10^{-11}	3×10^{-8}	0.1	Bone	1602 y	16,400	0.988	0.825 ^{††}
Ra-228	β^-, D	4×10^{-11}	3×10^{-8}	0.06	Bone	6.7 y	16,400	230	—
Rn-220	α, γ, D	3×10^{-7}				55.3 s		9.27×10^8	—
Rn-222	α, γ, D	1×10^{-7}				3.8229 d		1.54×10^5	—
Re-183	e^-, γ	2×10^{-7}	3×10^{-4}	80	T. B.	71 d	7	1.0×10^4	—
Re-186	β^-, γ	2×10^{-7}	5×10^{-5}	20	Thy.	88.9 h	3	1.89×10^5	0.006
Re-187	β^-	5 ×	2×10^{-3}	300	Skin	4.3×10^{10} y	25	4.4×10^{-8}	0
Re-188	β^-, γ	2×10^{-7}	3×10^{-5}	7	Thy.	16.7 h	3	9.98×10^5	—
Rh-103m	e^-, γ	6×10^{-5}	1×10^{-2}	200	Kid.	57.5 m	28	3.17×10^7	<0.001
Rh-105	β^-, γ	5×10^{-7}	1×10^{-4}	40	Kid.	35.88 h	28	8.32×10^5	0.042
Rb-86	$e^-, \gamma\gamma$	7×10^{-8}	2×10^{-5}	30	T. B.	18.66 d	45	8.14×10^4	0.051
Rb-87	β^-	7×10^{-8}	1×10^{-4}	200	T. B.	4.8×10^{10} y	45	8.6×10^{-8}	0
Ru-97	e^-, γ	2×10^{-6}	3×10^{-4}	30	Kid.	2.88 d	2.5	4.67×10^5	0.12
Ru-103	β^-, γ, D	8×10^{-6}	8×10^{-5}	20	Kid.	39.5 d	2.5	3.21×10^4	0.27
Ru-105	β^-, γ, D	5×10^{-7}	1×10^{-4}	2	Kid.	4.44 h	2.5	6.72×10^6	0.35
Ru-106	β^-, D	6×10^{-9}	1×10^{-5}	3	Kid.	368 d	2.5	3350	0.11 [‡]

(continues)

TABLE 9.51 Nuclide Information (97) (continued)

Isotope	Radiations	MPC air workers ($\mu\text{Ci/cc}$)	MPC water public, $\mu\text{Ci/cc}$	MPBB, μCi	C.O.	Half-life		Rhm per Ci^*	
						Radioactive	Biological days		
Sm-147	α	7×10^{-11}	6×10^{-5}	0.1	Bone	1.05×10^{11} y	1500	2.32×10^{-8}	0
Sm-151	β^-, γ	6×10^{-8}	4×10^{-4}	100	Bone	87 y	1500	27	0.006
Sm-153	β^-, γ	4×10^{-7}	8×10^{-5}	20	Liver	46.8 h	187	4.38×10^5	—
Sc-46	β^-, γ	2×10^{-8}	4×10^{-5}	10	Liver	83.9 d	36	3.38×10^4	1.1
Sc-47	β^-, γ	5×10^{-7}	9×10^{-5}	50	Liver	3.43 d	36	8.10×10^5	0.055
Sc-48	β^-, γ	1×10^{-7}	3×10^{-5}	9	T.B.	1.83 d	30	1.49×10^6	1.8
Se-75	e^-, γ	1×10^{-7}	3×10^{-4}	90	Kid.	120.4 d	11	1.45×10^4	0.20
Si-31	β^-	1×10^{-6}	2×10^{-4}	10	Lung	2.62 h	60	3.86×10^7	<0.001
Ag-105	e^-, γ	8×10^{-8}	1×10^{-4}	30	T.B.	40 d	5	3.1×10^4	0.21
Ag-110m	β^-, γ, D	1×10^{-8}	3×10^{-5}	10	T.B.	255 d	5	4650	1.4
Ag-111	β^-, γ	2×10^{-7}	4×10^{-5}	20	Kid.	7.5 d	10	1.6×10^5	0.013
Na-22	β^+, γ	9×10^{-9}	3×10^{-5}	10	T.B.	2.62 y	II	6210	1.2
Na-24	β^-, γ	1×10^{-7}	3×10^{-5}	7	T.B.	14.96 h	11	8.73×10^6	1.8
Sr-85m	e^-, γ, D	3×10^{-5}	7×10^{-3}	50	T.B.	70 m	13,000	3.2×10^7	0.11
Sr-85	e^-, γ	1×10^{-7}	1×10^{-4}	60	T.B.	64.0 d	13,000	2.40×10^4	0.29
Sr-89	β^-	3×10^{-8}	3×10^{-6}	4	Bone	52.7 d	18,000	2.78×10^4	<0.001
Sr-90	β^-, D	1×10^{-9}	3×10^{-7}	2	Bone	27.7 y	18,000	143	0
Sr-91	β^-, γ, D	3×10^{-7}	5×10^{-5}	3	Bone	9.67 h	18,000	3.56×10^6	0.38
Sr-92	β^-, γ, D	3×10^{-7}	6×10^{-5}	2	Bone	2.71 h	18,000	1.26×10^7	0.64
S-35	β^-	3×10^{-7}	6×10^{-5}	90	Tes.	87.9 d	623	4.24×10^4	0
Ta-182	β^-, γ	2×10^{-8}	4×10^{-5}	7	Liver	115.1 d	400	6230	0.6 \ddagger
Tc-96m	e^-, x, D	3×10^{-5}	1×10^{-2}	60	Kid.	52 m	20	3.8×10^7	0
Tc-96	e^-, γ	2×10^{-7}	5×10^{-5}	10	T.B.	4.35 d	1	3.13×10^5	1.4
Tc-97m	e^-, x	2×10^{-7}	2×10^{-4}	20	Kid.	91 d	20	1.5×10^4	0
Tc-97	x	3×10^{-7}	8×10^{-4}	60	Kid.	2.6×10^6 y	20	1.4×10^{-3}	0
Tc-99m	e^-, γ	1×10^{-5}	3×10^{-3}	200	T.B.	6.049 h	1	5.23×10^6	0.059
Tc-99	β^-	6×10^{-8}	2×10^{-4}	10	Kid.	2.12×10^5 y	20	0.0170	0
Te-125m	e^-, γ	1×10^{-7}	1×10^{-4}	20	Kid.	58 d	30	1.8×10^4	0.004

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Te-127m	$e^{-}, \beta^{-}, \gamma, D$	4×10^{-8}	5×10^{-5}	7	Kid.	109 d	30	9430	<0.001
Te-127	β^{-}, γ	9×10^{-7}	2×10^{-4}	20	Kid.	9.4 h	30	2.6×10^6	<0.001
Te-129m	β^{-}, γ, D	3×10^{-8}	2×10^{-5}	3	Kid.	34.1 d	30	2.97×10^4	0.023
Te-129	β^{-}, γ	4×10^{-6}	8×10^{-4}	5	Kid.	68.7 m	30	2.12×10^7	0.070
Te-131m	β^{-}, γ, D	2×10^{-7}	4×10^{-5}	4	Kid.	30 h	30	8.0×10^5	0.64
Te-132	β^{-}, γ, D	1×10^{-7}	2×10^{-5}	3	Kid.	77.7 h	30	3.05×10^5	0.11
Tl-b-160	β^{-}, γ	3×10^{-8}	4×10^{-5}	20	Bone	72.1 d	1000	1.13×10^4	0.51
Tl-200	e^{-}, β^{+}, γ	1×10^{-6}	2×10^{-4}	40	Kid.	26.1 h	7	6.00×10^5	—
Tl-201	e^{-}, γ	9×10^{-7}	2×10^{-4}	40	Kid.	74 h	7	2.1×10^5	0.008
Tl-202	e^{-}, γ	2×10^{-7}	7×10^{-5}	20	Kid.	12.0d	7	5.39×10^4	0.23
Tl-204	β^{-}, x	3×10^{-8}	6×10^{-5}	10	Kid.	3.81 y	7	460	—
Th-228	α, γ, D	6×10^{-12}	7×10^{-6}	0.02	Bone	1.910 y	73,000	821	—
Th-230	α, γ	2×10^{-12}	2×10^{-6}	0.05	Bone	8.0×10^4 y	73,000	0.019	—
Th-232	α, x, D	3×10^{-11}	2×10^{-6}	0.04	Bone	1.41×10^{10} y	73,000	1.09×10^{-7}	—
Th-234	β^{-}, γ, D	3×10^{-8}	2×10^{-5}	4	Bone	24.10 d	73,000	2.32×10^4	—
Th-nat	$\alpha, \beta^{-}, \gamma$	3×10^{-11}	1×10^{-6}	0.01	Bone	73,000	—	—	—
Tm-170	β^{-}, γ	3×10^{-8}	5×10^{-5}	9	Bone	134 d	1000	5730	0.004 ^s
Tm-171	β^{-}, γ	1×10^{-7}	5×10^{-4}	90	Bone	1.92 y	1000	1090	—
Sn-113	γ, D	5×10^{-8}	8×10^{-5}	30	Bone	115 d	100	1.00×10^4	0.002
Sn-125	β^{-}, γ, D	8×1^{-8}	2×10^{-5}	7	Bone	9.4 d	100	1.1×10^5	0.045
W-181	e^{-}, γ	1×10^{-7}	3×10^{-4}	70	Liver	140 d	4	5200	0.001
W-185	β^{-}	1×10^{-7}	1×10^{-4}	30	Bone	75 d	9	9400	0
W-187	β^{-}, γ	3×10^{-7}	6×10^{-5}	30	T. B.	23.9 h	1	7.01×10^5	0.23
U-230	α, γ, D	1×10^{-10}	5×10^{-6}	0.01	Kid.	20.8 d	15	2.73×10^4	—
U-232	α, γ, D	3×10^{-11}	3×10^{-5}	0.01	Bone	72 y	300	21—	—
U-233	α, γ	1×10^{-10}	3×10^{-5}	0.05	Bone	1.62×10^5 y	300	9.48×10^{-3}	—

(continues)

TABLE 9.51 Nuclide Information (97) (continued)

Isotope	Radiations	MPC air workers ($\mu\text{Ci/cc}$)	MPC water public, $\mu\text{Ci/cc}$	MPBB, μCi	C.O.	Half-life		Rhm per Ci^*
						Radioactive	Biological days	
U-234	α, γ	1×10^{-10}	3×10^{-5}	0.05	Bone	2.47×10^5 y	300	6.19×10^{-3}
U-235	α, γ, D	1×10^{-10}	3×10^{-5}	0.03	Kid.	7.1×10^8 y	15	2.1×10^{-6}
U-236	α, x	1×10^{-10}	3×10^{-5}	0.06	Bone	2.39×10^7 y	300	6.34×10^{-5}
U-238	$\alpha, \text{x}, \text{D}$	7×10^{-11}	4×10^{-5}	0.005	Kid.	4.51×10^9 y	15	3.33×10^{-7}
U-240	$\beta^-, \text{x}, \text{D}$	2×10^{-7}	3×10^{-5}	4	Kid.	14.1 h	15	9.26×10^5
U-nat	α, β^-, γ	6×10^{-11}	2×10^{-5}	0.005	Kid.		15	
D-38	α, β, γ	7×10^{-11}						4.38×10^{-7}
V-48	β^+, γ	6×10^{-8}	3×10^{-5}	8	Kid.	16.0 d	74	1.70×10^5
Xe-131m	e^-, γ	2×10^{-5}				11.8 d		8.45×10^4
Xe-133m	e^-, γ, D	1×10^{-5}				2.26 d		4.34×10^5
Xe-133	β^-, γ	1×10^{-5}				5.270 d		1.86×10^5
Xe-135	β^-, γ	4×10^{-6}				9.14 h		2.54×10^6
Yb-175	β^-, γ	6×10^{-7}	1×10^{-4}	30	Bone	101 h	1000	1.77×10^5
Y-88	β^+, γ	$1 \times 10^{-7\dagger}$	$1 \times 10^{-5\dagger}$	50 [†]	Bone [†]	108.1 d	18,000	1.37×10^4
Y-90	β^-	1×10^{-7}	2×10^{-5}	3	Bone	64.0 h	18,000	5.44×10^5
Y-91m	e^-, γ, D	2×10^{-5}	3×10^{-5}	5	Bone	50.3 m	18,000	4.11×10^7
Y-91	β^-, γ	3×10^{-8}	3×10^{-5}	5	Bone	58.8 d	18,000	2.44×10^4
Y-92	β^-, γ	3×10^{-7}	6×10^{-5}	2	Bone	3.53 h	18,000	9.65×10^6
Y-93	β^-, γ	1×10^{-7}	3×10^{-5}	2	Bone	10.3 h	18,000	3.27×10^6
Zn-65	β^+, γ	6×10^{-8}	1×10^{-4}	60	T. B.	245 d	933	8200
Zn-69m	e^-, γ, D	3×10^{-7}	6×10^{-5}	0.7	Pros.	13.8 h	14	3.29×10^6
Zn-69	β^-	7×10^{-6}	2×10^{-3}	0.8	Pros.	57 m	14	4.8×10^7
Zr-93	$\beta^-, \gamma, \text{D}$	1×10^{-7}	8×10^{-4}	100	Bone	1.5×10^6 y	1000	2.6×10^{-3}
Zr-95	$\beta^-, \gamma, \text{D}$	3×10^{-8}	6×10^{-5}	20	T. B.	65.5 d	450	2.10×10^4
Zr-97	$\beta^-, \gamma, \text{D}$	9×10^{-8}	2×10^{-5}	5	Bone	17.0 h	1000	1.90×10^6

*Remember that bremsstrahlung and x-rays from daughters are not included, but may significantly increase the dose rate when multicurie quantities of material are handled.
[†]Estimated.

[‡]Includes daughters.

[§]Values from NCRP Report No. 24 (r06). This value is conservative for mixed fission products up to 2 years old. Higher numbers can be used if the mixture is analyzed by gamma-ray spectrometry or other means.

$$T_{\text{eff}} = \frac{(T_{\text{RAD}} T_{\text{BIO}})}{(T_{\text{RAD}} + T_{\text{BIO}})}$$

Ci/g: The specific activity of an isotope is expressed in curies per gram. The values listed are for the pure elemental species, not the usual chemical forms.

Rhm per Ci: This column provides approximations of the roentgens per hour expected at one meter (Rhm) from one curie of the isotope. Many of the isotopes listed do not have values identified because they emit alpha or low-energy beta particles. It should be noted that these values are for a point source and do not include secondary radiation (bremsstrahlung) caused by excitation of other materials to give off x-rays or radiation from decay daughters. These secondary effects may generate substantially more radiation than the values listed.

Low-Level Waste Regulations

The principal regulations for LLW in the United States are administered by the Nuclear Regulatory Commission (NRC) and the Department of Transportation (DOT).

Waste Classification. The NRC has established regulatory requirements for disposal of LLW in 10 CFR 61 (94). This regulation is based on a classification system and a set of corresponding waste-form-acceptance criteria for licensed commercial facilities (107–109). This system is based on concentration limits (expressed in curies per cubic meter or microcuries per cubic centimeter) and establishes three categories of waste:

Class A: Consists primarily of shorter-lived isotopes and limited concentrations of longer-lived isotopes. Wastes can be disposed of with only minimum requirements for waste form and characteristics.

Class B: Increased concentration limits for certain isotopes and additional requirements for waste form and characteristics to ensure physical stability after disposal.

Class C: Greater concentration and stability requirements for physical form, packaging, and characteristics than Class B.

Radioactive wastes with concentrations exceeding Class C values are not acceptable for disposal at commercial facilities without specific approval by the NRC. Figure 9.30 presents charts showing the steps to be taken to classify the wastes under 10 CFR 61. Radioactive wastes containing nuclides not identified in these charts are considered Class A.

The NRC requires waste mixture to be evaluated using the “sums of the fractions method.” Each isotope present in the waste mixture must be divided by the appropriate limit from the charts in Figure 9.30. The sums of these fractions must not exceed 1 to remain within the classification category. If the value exceeds one then the next higher classification is required. All values must be taken from same column of the same table.

NRC Waste Package Requirements. The minimum waste packaging acceptance criteria in 10 CFR 61 place the following restrictions on all classes of waste (94):

- Cardboard or fiberboard boxes may not be used for packaging
- Liquids must either be solidified or packed with materials capable of absorbing twice the liquid volume
- Wastes must contain less than 1% of the volume as free liquid
- Wastes must not be capable of detonation, explosive decomposition or reaction, or reacting explosively

with water

- Wastes must not contain or generate toxic or harmful gases, vapors, or fumes
- Wastes must not be pyrophoric
- Gaseous wastes must have less than 100 Ci per container and not exceed 1.5 atm at 20°C
- Hazardous, biological, pathogenic, or infectious waste material must be extracted to minimize nonradiological hazard.

Class B and C wastes must meet additional stability requirements that include: (1) the waste must remain in a stable and recognizable form or container for 300 years, (2) waste packages must contain less than 1% free-standing water in containers designed to ensure stability, and (3) less than 0.5% of the package volume can be free-standing water for solidified waste.

TABLE 9.52 DOT Requirements for LLW Shipment (110)

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- Radioactive material definition. Radioactive material is defined for shipping purposes as any material having a specific activity greater than 2 nCi/g. Although this criterion implies that waste with radionuclide concentrations of less than 2 nCi/g could be considered nonradioactive, it is emphasized that the de minimis level or limit of no regulatory concern has not been established for disposal of LLW.
 - Excepted radioactive materials. Certain types of radioactive material are excepted from the DOT packaging, marking, and labeling requirements. Even though these exceptions usually cover manufactured articles, instruments, and devices common to the commercial and industrial sectors, generators should examine the specific criteria to determine if their wastes are excepted from DOT requirements.
 - DOT forms of waste. DOT regulations recognize two forms of radioactive materials, normal and special. Most LLWs suitable for transport are normal-form materials. The special form of radioactive material usually consists of either a solid waste or an encapsulated waste not readily subject to degradation after disposal (49 CFR 173A69). The DOT transportation criteria for normal-form material are more stringent than are those for special-form material.
 - Physical form. Liquids cannot be transported unless the container in which the liquids are placed contains sufficient absorbent material to absorb twice the volume of liquids being shipped (see 49 CFR 173.412). DOT requirements for liquids packaging apply to only a minor portion of the LLW that is shipped (e.g., liquid-scintillation fluid). There are other restrictions on the disposal of liquids. DOT requirements also pertain to off-site shipment of small samples for analysis and to common carrier shipment of radioactive liquids between NRC-licensed and DOE facilities.
 - Radiation levels. LLW is usually transported in one of the following shipping configurations with the radiation level as noted: Unshielded, closed vans with 1000 mrem/h at the outer surface of the package, 200 mrem/h at any point on the outer surface of the vehicle, and 10 mrem/h at 2 m from the outer surface of the vehicle. Unshielded, open flatbed trucks with 200 mrem/h at any point on the vertical planes projected from the outer edges of the vehicle, and 10 mrem/h at 2 m from the outer surface of the vehicle. Shielded shipments with 200 mrem/h on the surface of the shipping cask or shield, and 10 mrem/h at 2 m from the vertical planes projected by the outer surfaces of the vehicle.
 - Package classification. DOT's method of classifying packages is based on the potential inhalation dose to the public in the event of an accident where 0.1% of the package contents would be released. The inhalation dose varies according to the radionuclide in question; therefore, DOT has assigned dose-related values for 250 radionuclides. These values are used to classify individual packages. Because of the complex requirements, each generator should evaluate shipping needs and package classification regulations with the permitting agency before final selection of a waste-processing system.
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TABLE 9.53 Technologies Used for Processing Liquid LLW (110)

Type of technology	Regenerative solutions	Decontamination solutions	Oils	Other organics	Other inorganics	Scintillation vials and biological wastes
Transfer technologies						
Filtration	X	X	X	—	—	X
Ion exchange	X	—	—	—	—	X
Ultrafiltration	X	—	—	—	—	X
Reverse osmosis	X	—	—	—	—	X
Concentration technologies						
Evaporation	X	X	—	—	—	X
Distillation	X	X	—	—	—	X
Crystallization	X	X	—	—	—	X
Flocculation	X	X	—	—	—	X
Precipitation	X	X	—	—	—	X
Sedimentation	X	X	—	—	—	X
Centrifugation	X	X	—	—	—	X
Integrated system	X	X	—	—	—	X
Transformation technologies						
Incineration	—	—	X	X	X	—
Calcination	—	—	X	X	X	—
Conditioning technologies						
Solidification	—	—	X	X	—	X
Absorption	—	—	X	X	X	X

TABLE 9.54 Technologies Used for Processing Wet Solids LLW (110)

Type of technology	Evaporator bottoms and miscellaneous sludges	Spent ion-exchange resins	Filter sludges	Filter cartridges
Transfer technologies				
Filtration	X	—	—	—
Chemical regeneration	—	X	—	—
Concentration technologies				
Sedimentation	X	—	—	—
Centrifugation	X	—	—	—
Drying	X	—	X	—
Dewatering	X	X	X	—
Dehydration	X	X	X	—
Integrated systems	X	—	—	—
Transformation technologies				
Incineration		X	X	—
Calcination	X	X	X	X
Conditioning technologies				
Solidification	X	X	X	X
High-integrity containers	—	X	X	X
Absorption	X	—	—	—

*Applicable to miscellaneous sludges only.

TABLE 9.55 Technologies Used for Processing Dry Solids LLW (110)

Type of technology	Trash	Contaminated equipment	Irradiated hardware
Transfer technologies			
Decontamination	—	X	—
Concentration technologies			
Compaction	X	—	—
Shredding or sectioning	X	X	X
Baling	X	—	—
Transformation technologies			
Incineration	X	—	—
Conditioning technologies			
High-integrity containers	—	—	X
Solidification (ash)	X	—	—

Transportation Regulations. The majority of LLWs are transported by truck and the shipper or generator is responsible for complying with the regulations governing the transportation of radioactive material. Two federal agencies, the DOT and the NRC, have established regulations for the safe transport of radioactive materials. DOT regulates packaging, labeling, and transport conditions for commercial carriers, and the NRC reviews and approves its licensees' package design for commercial shipments. The major regulations issued by these agencies are 49 CFR 170–178 and 10 CFR 71, respectively (*112, 113*).

Individual states and disposal sites have additional regulations and requirements which control other details of packaging, waste form, and transportation routes and methods.

The applicable packaging regulations are complex and specific to each type of waste. Therefore, the complete regulations are not elaborated on in this document. Before finalizing waste-processing techniques, the shipper or generator should review the referenced regulations and discuss specific packaging problems with the permitting agencies.

DOT Regulations. The DOT regulations (49 CFR 170–178) establish criteria for the safe transport of radioactive material on public highways (*112*). Many of these requirements are based on accident scenarios and limits for inhalation pathways (*114*). Table 9.52 (page 9.116) summarizes the DOT requirements for shipping of LLW. It should be noted that transportation regulations and requirements are complex and must be thoroughly evaluated prior to selection of waste form or package.

NRC Regulations. The NRC regulations (10 CFR 71) establish criteria for packaging, preparing for shipment, and transporting radioactive material (*113*). Generators must conform to these regulations except in cases when the waste package contains radioactive materials with a specific activity less than 2 nCi/g.

The NRC regulations are supplemental to those of DOT and primarily pertain to materials with higher activities than covered by the DOT regulations. Shippers and generators must consider both sets of regulatory requirements prior to packaging and transporting.

There are two requirements in the NRC regulations which are not addressed by DOT. First, licensed materials exceeding specified activity levels must be shipped in NRC-approved packages having an NRC certificate of compliance. Second, NRC requires advance notification of shipments of wastes across state lines when shipments contain radioactive materials.

Treatment and Processing Methods

Treatment and processing of radioactive wastes are similar to those for hazardous wastes. Many of the technologies are directly applicable or require only minor modification (usually off-gas treatment systems).

As previously discussed, the end waste form will determine requirements for transport, handling, and disposal. The ultimate goal in LLW processing is to achieve a solid, stable waste form which meets the disposal classification requirements of NRC (*110, 111, 132*). While this does not seem any different than the guidelines established for chemical waste management, it should be remembered that the radiation hazards of the LLW are not eliminated by treatment. In fact, some treatment methods concentrate the radioactivity, making the end product more hazardous than the starting material. Also, waste processing may in fact generate more LLW if processes, equipment, materials, and residues become contaminated. At present, there are no *de minimis* levels for LLW, and therefore all materials that become contaminated in processing or treatment automatically become LLW.

Selecting appropriate treatment technologies for wastes must carefully consider all aspects of the process method. Tables 9.53 through 9.55 and Figures 9.30 through 9.33 show similarities between LLW and chemical waste processing and treatment systems.

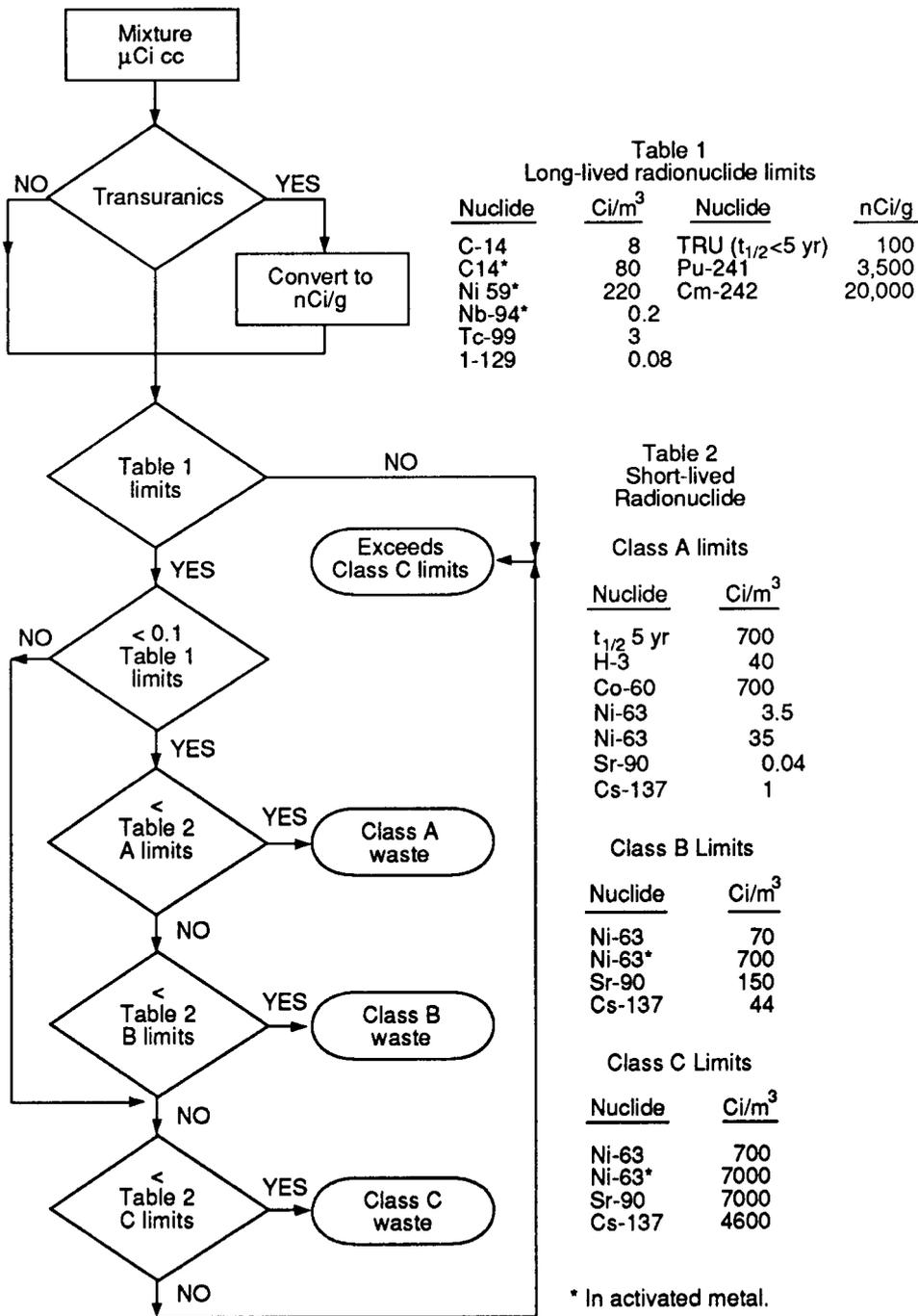


FIGURE 9.30 NRC waste classification system (110, 111).

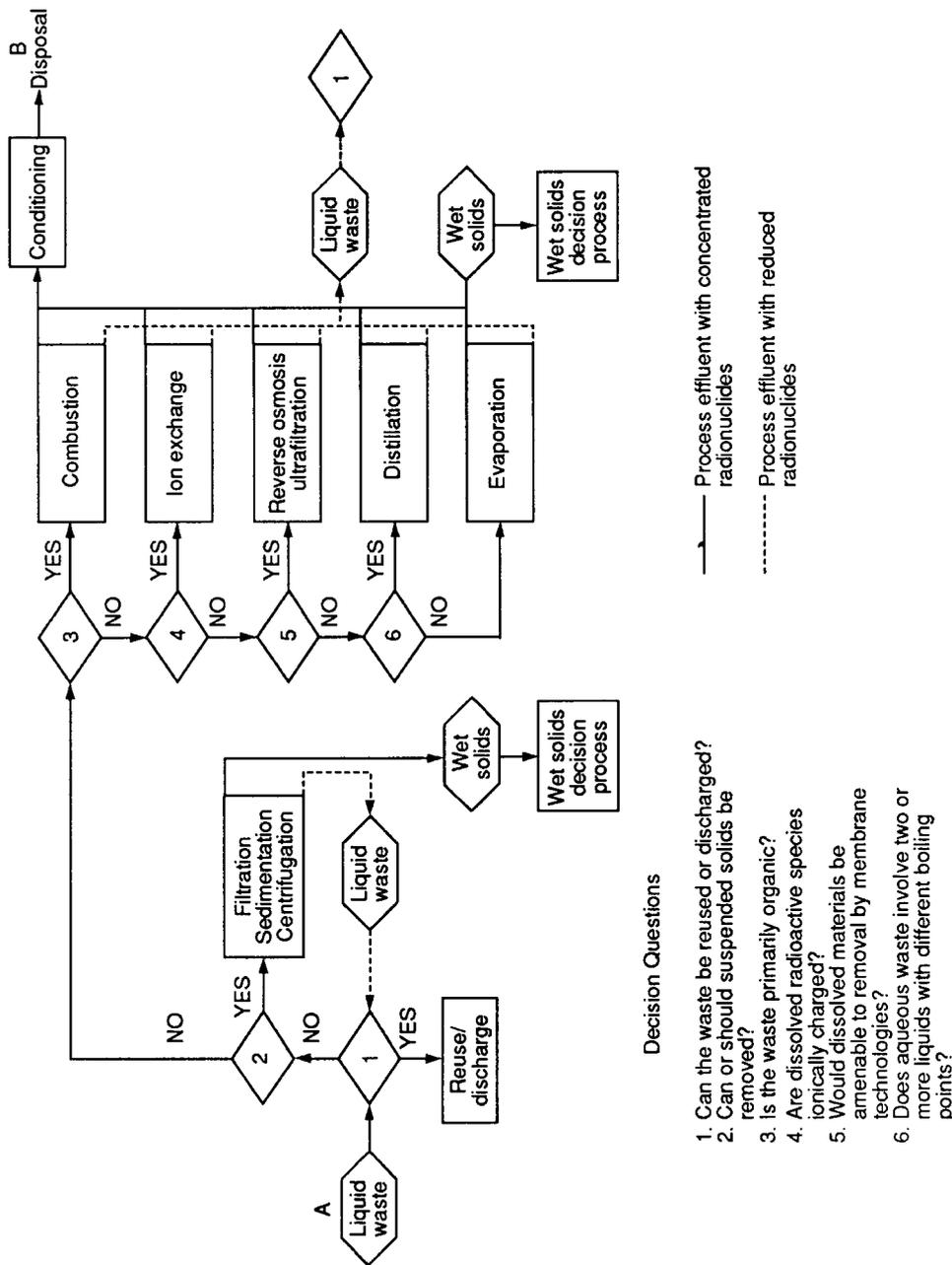
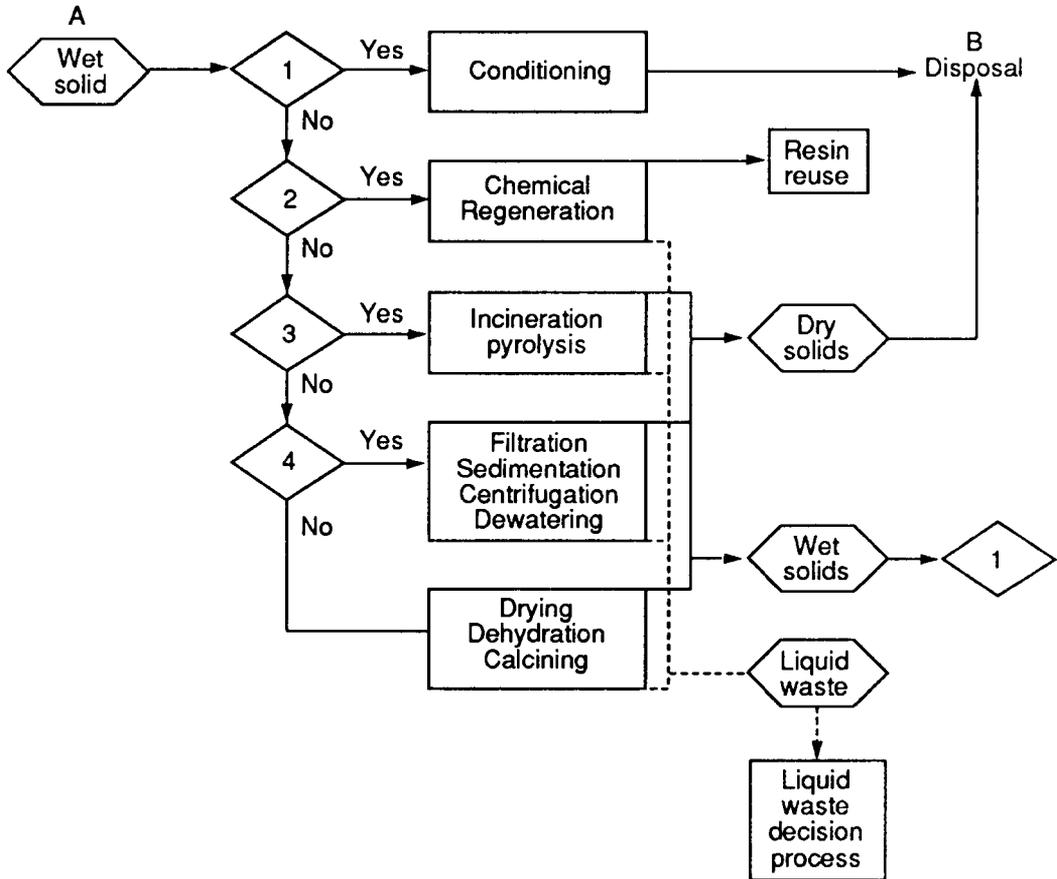


FIGURE 9.31 Technology selection chart for liquid LLW (110).



Decision Questions

1. Can the wet solid be conditioned for disposal?
2. Can wet solids be processed for reuse?
3. Are the wet solids primarily organic?
4. Is the aqueous wet solid pumpable?

————— Process effluent with concentrated radionuclides

----- Process effluent with reduced radionuclides

FIGURE 9.32 Technology selection chart for wet solids LLW (110).

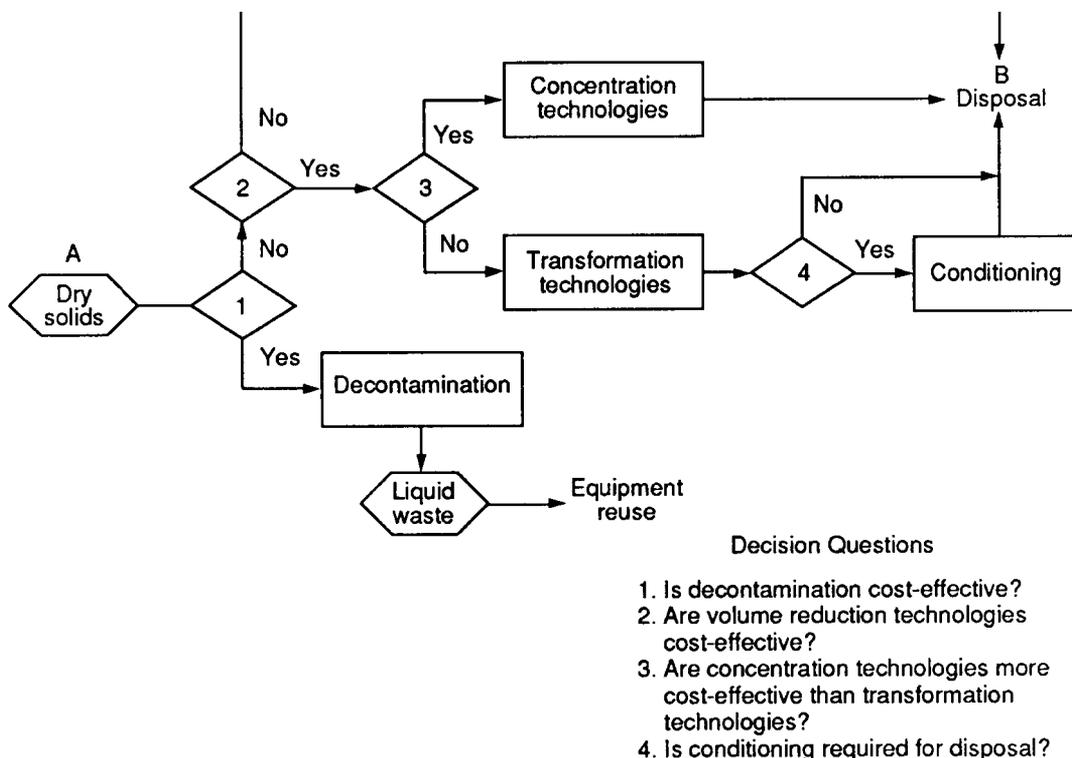


FIGURE 9.33 Technology selection chart for dry solids LLW (110).

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HAZARDOUS WASTE

CHAPTER 10

ENVIRONMENTAL ASSESSMENT

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During the decision-making process and development of a project, the environmental engineer should incorporate considerations of the proposed actions on the total environment: air, water, and land. Also, the primary and secondary impacts on neighbors and the general public are important. In the United States, the National Environmental Policy Act (*1*) and subsequent regulations form the basis for environmental assessments (EAs) and environmental impact statements (EISs). An overview of the first twenty-five years of these regulations is included in this chapter.

The environmental assessment process documents the awareness of the owner, designer, and public of the potential impacts resulting from the construction any new, modified, or expanded facilities. The documentation is concise, and the degree of detail varies with the size and location of the project. The outcome of the EA process will be a regulatory agency finding of no significant impact or a requirement for an EIS. An environmental impact statement documents in great detail real and potential direct and secondary impacts of the proposed project. Alternative evaluations and mitigation actions may be included. Public participation is key in the EA/EIS process.*

LEGISLATIVE REQUIREMENTS

The environmental assessment process is couched in federal law, with auxiliary requirements evolved through executive orders, regulations, agency response, state legal requirements, and other legal mechanisms. The principal federal legislation is the National Environmental Policy Act of 1969 (PL 91-190), typically referred to as NEPA (see Figure 10.1).

When compared with most environmental legislation, NEPA has four distinguishing features (3).

1. NEPA is a policy act setting a course for public action and is not a regulatory statute.
2. NEPA's goals and principles are not self-enforcing but require action by Congress and the President. Observance of the policy intent is mandated by the environmental impact statement and is enforceable through the courts.
3. NEPA is integrative, crossing and affecting a broad range of public issues—economic, demographic, ecological, aesthetic, and ethical.
4. NEPA is oriented toward the future by recognizing the worldwide and long-range character of environmental problems.

*Material for portions of this chapter has been drawn from Ref. 2.

PURPOSE

Sec. 2. The purposes of this Act are: to declare a national policy which will encourage productive and enjoyable harmony between man and his environment; to promote efforts which will prevent or eliminate damage to the environment and biosphere and stimulate the health and welfare of man; to enrich the understanding of the ecological systems and natural resources important to the Nation; and to establish a Council on Environmental Quality.

Title I
Declaration of National
Environmental Policy

Sec. 101. (a) The Congress, recognizing the profound impact of man's activity on the interrelations of all components of the natural environment, particularly the profound influences of population growth, high-density urbanization, industrial expansion, resource exploitation, and new and expanding technological advances and recognizing further the critical importance of restoring and maintaining environmental quality to the overall welfare and development of man, declares that it is the continuing policy of the Federal Government, in cooperation with State and local governments, and other concerned public and private organizations, to use all practicable means and measures, including financial and technical assistance, in a manner calculated to foster and promote the general welfare, to create and maintain conditions under which man and nature can exist in productive harmony, and fulfill the social, economic, and other requirements of present and future generations of Americans.

(b) In order to carry out the policy set forth in this Act, it is the continuing responsibility of the Federal Government to use all practicable means, consistent with other essential considerations of national policy, to improve and coordinate Federal plans, functions, programs, and resources to the end that the Nation may—

- (1) fulfill the responsibilities of each generation as trustee of the environment for succeeding generations;
- (2) assure for all Americans safe, healthful, productive, and esthetically and culturally pleasing surroundings;
- (3) attain the widest range of beneficial uses of the environment without degradation, risk to health or safety, or other undesirable and unintended consequences;
- (4) preserve important historic, cultural, and natural aspects of our national heritage, and maintain, wherever possible, an environment which supports diversity and variety of individual choice;
- (5) achieve a balance between population and resource use which will permit high standards of living and a wide sharing of life's amenities; and
- (6) enhance the quality of renewable resources and approach the maximum attainable recycling of depletable resources.

(c) The Congress recognizes that each person should enjoy a healthful environment and that each person has a responsibility to contribute to the preservation and enhancement of the environment.

Sec. 102. The Congress authorizes and directs that, to the fullest extent possible: (1) the policies, regulations, and public laws of the United States shall be interpreted and administered in accordance with the policies set forth in this Act, and (2) all agencies of the Federal Government shall—

FIGURE 10.1 The National Environmental Policy Act—Public Law 91-190 (as amended by PL 94-83).

(A) utilize a systematic, interdisciplinary approach which will ensure the integrated use of the natural and social sciences and the environmental design arts in planning and in decision making which may have an impact on man's environment;

(B) identify and develop methods and procedures, in consultation with the Council on Environmental Quality established by title II of this Act, which will ensure that presently unquantified environmental amenities and values may be given appropriate consideration in decision-making along with economic and technical considerations;

(C) include in every recommendation or report on proposals for legislation and other major Federal actions significantly affecting the quality of the human environment, a detailed statement by the responsible official on—

- (i) the environmental impact of the proposed action,
- (ii) any adverse environmental effects which cannot be avoided should the proposal be implemented,
- (iii) alternatives to the proposed action,
- (iv) the relationship between local short-term uses of man's environment and the maintenance and enhancement of long-term productivity, and
- (v) any irreversible and irretrievable commitments of resources which would be involved in the proposed action should it be implemented.

Prior to making any detailed statement, the responsible Federal official shall consult with and obtain the comments of any Federal agency which has jurisdiction by law or special expertise with respect to any environmental impact involved. Copies of such statement and the comments and views of the appropriate Federal, State, and local agencies, which are authorized to develop and enforce environmental standards, shall be made available to the President, the Council on Environmental Quality and to the public as provided by section 552 of title 5, United States Code, and shall accompany the proposal through the existing agency review processes.

(D) Any detailed statement required under subparagraph (C) after January 1, 1970, for any major Federal action funded under a program of grants to States shall not be deemed to be legally insufficient solely by reason of having been prepared by a State agency or official, if:

- (i) the State agency official has statewide jurisdiction and has the responsibility for such action,
- (ii) the responsible Federal official furnishes guidance and participates in such preparation.
- (iii) the responsible Federal official independently evaluates such statement prior to its approval and adoption, and
- (iv) after January 1, 1976, the responsible Federal official provides early notification to, and solicits the views of, any other State or any Federal land management entity of any action or any alternative thereto which may have significant impacts upon such State or affected Federal land management entity and, if there is any disagreement on such impacts, prepared a written assessment of such impacts and views for incorporation into such detailed statement.

The procedures in this subparagraph shall not relieve the Federal official of his responsibilities for the scope, objectivity, and content of the entire statement or of any other responsibility under this Act; and further, this subparagraph does not affect the legal sufficiency of statements prepared by State agencies with less than statewide jurisdiction.

(E) study, develop, and describe appropriate alternatives to recommended courses of action in any proposal which involves unresolved conflicts concerning alternative uses of available resources;

FIGURE 10.1 The National Environmental Policy Act—Public Law 91-190 (as amended by PL 94-83) (*continued*).

(F) recognize the worldwide and long-range character of environmental problems and, where consistent with the foreign policy of the United States, lend appropriate support to initiatives, resolutions, and programs designed to maximize international cooperation in anticipating and preventing a decline in the quality of mankind's world environment;

(G) make available to States, counties, municipalities, institutions, and individuals, advice and information useful in restoring, maintaining and enhancing the quality of the environment;

(H) initiate and utilize ecological information in the planning and development of resource-oriented projects; and

(I) assist the Council on Environmental Quality established by title II of this Act.

Sec. 103. All agencies of the Federal Government shall review their present statutory authority, administrative regulations, and current policies and procedures for the purpose of determining whether there are any deficiencies or inconsistencies therein which prohibit full compliance with the purposes and provisions of this Act and shall propose to the President not later than July 1, 1971, such measures as may be necessary to bring their authority and policies into conformity with the intent, purposes, and procedures set forth in this Act.

Sec. 104. Nothing in Section 102 or 103 shall in any way affect the specific statutory obligations of any Federal agency (1) to comply with criteria or standards of environmental quality; (2) to coordinate or consult with any other Federal or State agency, or (3) to act, or refrain from acting contingent upon the recommendations or certification of any other Federal or State agency.

Sec. 105. The policies and goals set forth in this Act are supplementary to those set forth in existing authorizations of Federal agencies.

Title II Council on Environmental Quality

Sec. 201. The President shall transmit to the Congress annually beginning July 1, 1970, an Environmental Quality Report (hereinafter referred to as the "report") which shall set forth (1) the status and condition of the major natural, manmade, or altered environmental classes of the Nation, including, but not limited to, the air, the aquatic, including marine, estuarine, and fresh water, and the terrestrial environment, including, but not limited to, the forest dryland, wetland, range, urban, suburban, and rural environment; (2) current and foreseeable trends in the quality management and utilization of such environments and the effects of those trends on the social, economic, and other requirements of the Nation; (3) the adequacy of available natural resources for fulfilling human and economic requirements of the Nation in the light of expected population pressures; (4) a review of the programs and activities (including regulatory activities) of the Federal Government, the State and local governments, and nongovernmental entities or individuals, with particular reference to their effect on the environment and on the conservation, development and utilization of natural resources; and (5) a program for remedying the deficiencies of existing programs and activities, together with recommendations for legislation.

Sec. 202. There is created in the Executive Office of the President a Council on Environmental Quality (hereinafter referred to as the "Council"). The Council shall be composed of three members who shall be appointed by the President to serve at his pleasure, by and with the advice and consent of the Senate. The President shall designate one of the members of the Council to serve as Chairman. Each member shall be a person who as a result of his training, experience, and attainments, is exceptionally well qualified to analyze and interpret environmental trends and information of all kinds; to appraise programs and activities of the Federal Government in the light of the policy set forth in Title I of this Act; to be conscious of and responsive to

FIGURE 10.1 The National Environmental Policy Act—Public Law 91-190 (as amended by PL 94-83) (*continued*).

the scientific, economic, social, esthetic, and cultural needs and interests of the Nation; and to formulate and recommend national policies to promote the improvement of the quality of the environment.

Sec. 203. The Council may employ such officers and employees as may be necessary to carry out its functions under this Act. In addition, the Council may employ and fix the compensation of such experts and consultants as may be necessary for the carrying out of its functions under this Act, in accordance with section 3109 of title 5, United States Code (but without regard to the last sentence thereof).

Sec. 204. It shall be the duty and function of the Council—

(1) to assist and advise the President in the preparation of the Environmental Quality Report required by section 201;

(2) to gather timely and authoritative information concerning the conditions and trends in the quality of the environment both current and prospective, to analyze and interpret such information for the purpose of determining whether such conditions and trends are interfering, or are likely to interfere, with the achievement of the policy set forth in title I of this Act, and to compile and submit to the President studies relating to such conditions and trends;

(3) to review and appraise the various programs and activities of the Federal Government in the light of the policy set forth in title I of this Act for the purpose of determining the extent to which such programs and activities are contributing to the achievement of such policy, and to make recommendations to the President with respect thereto;

(4) to develop and recommend to the President national policies to foster and promote the improvement of environmental quality to meet the conservation, social, economic, health, and other requirements and goals of the Nation;

(5) to conduct investigations, studies, surveys, research, and analyses relating to ecological systems and environmental quality;

(6) to document and define changes in the natural environment, including the plant and animal systems, and to accumulate necessary data and other information for a continuing analysis of these changes or trends and an interpretation of their underlying causes;

(7) to report at least once each year to the President on the state and condition of the environment; and

(8) to make and furnish such studies, reports thereon, and recommendations with respect to matters of policy and legislation as the President may request.

Sec. 205. In exercising its powers, functions, and duties under this Act, the Council shall—

(1) consult with the Citizen's Advisory Committee on Environmental Quality established by Executive Order numbered 11472, dated May 29, 1969, and with such representatives of science, industry, agriculture, labor, conservation organizations, State and local governments, and other groups as it deems advisable; and

(2) utilize, to the fullest extent possible, the services, facilities, and information (including statistical information) of public and private agencies and organizations, and individuals in order that duplication of effort and expense may be avoided, thus assuring that the Council's activities will not unnecessarily overlap or conflict with similar activities authorized by law and performed by established agencies.

Sec. 206. Members of the Council shall serve full time and the Chairman of the Council shall be compensated at the rate provided for Level II of the Executive Schedule Pay Rates (5 U.S.C. 5313). The other members of the Council shall be compensated at the rate provided for Level IV of the Executive Schedule Pay Rates (5 U.S.C. 5315).

Sec. 207. There are authorized to be appropriated to carry out the provisions of this Act not to exceed \$300,000 for fiscal year 1970, \$700,000 for fiscal year 1971, and \$1,000,000 for each fiscal year thereafter.

FIGURE 10.1 The National Environmental Policy Act—Public Law 91-190 (as amended by PL 94-83) (*continued*).

National Environmental Policy Act

The enactment of NEPA added a new dimension to the planning and decision-making process of federal and state agencies in the United States. NEPA requires federal agencies to assess the environmental impact of implementing their major programs and actions as early in the planning process as possible and, in all cases, prior to the undertaking of the project or the action (1). For those projects or actions that are either expected to have a significant effect on the quality of the human environment or are expected to be controversial on environmental grounds, the proponent agency is required to file a formal environmental impact statement (EIS) (1). This environmental assessment process (also referred to as the “NEPA process”) has had a profound effect on the way major projects with significant federal agency involvement are planned and implemented.

In order to incorporate environmental concerns into the decision-making process, the environmental impact analysis of major undertakings is a logical first step. In the normal market exchange of goods and services, these environmental concerns cannot be, and are not, properly accounted for. Consequently, in the United States, provisions of NEPA were designed to create a vehicle for considering environmental amenities consistent with other national needs, such as economic development, in a systematic manner. This systematic procedure is called the environmental assessment process, and it is documented as an environmental assessment or an impact statement.

Purpose. The main purposes of this legislation, as set forth in the act, are

To declare a national policy which will encourage productive and enjoyable harmony between man and his environment; to promote efforts which will prevent or eliminate damage to the environment and biosphere and stimulate the health and welfare of man; to enrich the understanding of the ecological systems and natural resources important to the Nation; and to establish a Council on Environmental Quality (1).

There are two titles under the Act: Title I, Declaration of National Environmental Policy, and Title II, Council on Environmental Quality (CEQ).

Title I. Title I sets forth the national policy on restoration and protection of environmental quality. The relevant sections under this title are summarized as follows:

Section 101. Requirements of Section 101 are of a substantive nature. Under this section, the federal government has a continuing responsibility “consistent with other essential considerations of national policy to minimize adverse environmental impact and to preserve and enhance the environment as a result of implementing federal plans and programs.

Section 102. Section 102 requirements are of a procedural nature. Under this section, the proponent federal agency is required to make a full and adequate analysis of all environmental effects of implementing its programs or actions.

In Section 102(1), Congress directs that policies, regulations, and public laws shall be interpreted and administered in accordance with the policies of NEPA. Section 102(2) directs all federal agencies to follow a series of steps to ensure that the goals of the act will be met.

The first requirement is found in Section 102(2)(A), where it is required that “a systematic and interdisciplinary approach” be used to ensure the integrated use of social, natural, and environmental sciences in planning and decision making.

Section 102(2)(B) states that federal agencies shall, in consultation with CEQ, identify and develop procedures and methods such that “presently unquantified environmental amenities and values may be given appropriate consideration in decision-making” along with traditional economic and technical considerations.

Section 102(2)(C) sets forth the requirements and guidelines for preparing environmental impact statements. This section requires all federal agencies to include in every recommendation or report on legislative

proposals and other major federal actions significantly affecting the quality of the human environment, a detailed statement by the responsible official covering the following elements:

1. The environmental impact of the proposed actions
2. Any adverse environmental effects that cannot be avoided should the proposal be implemented
3. The alternatives to the proposed action
4. The relationship between local short-term uses of the environment and the maintenance and enhancement of long-term productivity
5. Any irreversible and irretrievable commitments of resources that would be involved in the proposed action should it be implemented

Specific EIS format, coordination, and instructions for approval and review hierarchy are established by CEQ regulations (4) and each federal agency. Persons preparing an EIS should follow the instructions of the appropriate organization.

Section 103. This section requires all federal agencies to review their regulations and procedures “for the purpose of determining whether there are any deficiencies or inconsistencies therein which prohibit full compliance with the purposes and provisions of this Act and [they] shall propose to the President . . . such measures as may be necessary to bring their authority and policies into conformity with . . . this Act.”

Title II. Title II establishes the CEQ as an environmental advisory body for the executive office. In addition, the President is required to submit to the Congress an annual “Environmental Quality Report.” This yearly summary sets forth: (1) the status and condition of the major natural, man-made, or altered environmental classes of the nation; (2) current and foreseeable trends in the quality, management, and utilization of such environments and socioeconomic impacts of these trends; (3) the adequacy of available natural resources; (4) a review of the effect of governmental and nongovernmental activities on the environment and natural resources; and (5) a program for remedying the deficiencies and recommending appropriate legislation.

The CEQ is composed of three members who are appointed by the President with the advice and consent of the Senate. The President designates one of the members of the CEQ to serve as chairman. In addition, the CEQ employs environmental lawyers, professional scientists, and other employees to carry out its functions as required under NEPA. Duties and functions of CEQ may be summarized as follows:

1. Assist and advise the President in the preparation of the Environmental Quality Report as required by NEPA
2. Gather, analyze, and interpret, on a timely basis, information concerning the conditions and trends in the quality of the environment, both current and prospective
3. Review and appraise the various programs and activities of the federal government in light of the policy of environmental protection and enhancement, as set forth under Title I of NEPA
4. Develop and recommend to the President national policies to foster and promote improvement of environmental quality to meet many goals of the nation
5. Conduct research and investigations related to ecological systems and environmental quality
6. Accumulate necessary data and other information for a continuing analysis of changes in the national environment and interpretation of the underlying causes
7. Report at least once a year to the President on the state and condition of the environment
8. Conduct such studies and furnish such reports and recommendations as the President may request

A significant feature to note is that both the charter assigned and the responsibilities delegated to CEQ are quite extensive. However, although CEQ does not have any regulatory or policing responsibilities, it is highly influential in its advisory capacity.

CEQ REGULATIONS AND ENVIRONMENTAL DOCUMENTS

In the United States, the CEQ is responsible for overseeing federal efforts to comply with NEPA. In 1970, the council issued guidelines for the preparation of EISs under Executive Order 11514 (1970). Until recently, the 1973 revised guidelines were in effect. But under Executive Order 11991, the President directed the CEQ to issue regulations to supersede the 1973 guidelines. Issued in 1977, the regulations became effective on July 30, 1979. In the executive order, the President directed that the regulations should be (5):

Designed to make the Environmental Impact Statement process more useful to decision-makers and the public and to reduce paperwork and the accumulation of extraneous background data, in order to emphasize the need to focus on real environmental issues and alternatives.

The new regulations were developed to achieve three principal goals: reduction of paperwork; reduction of delays; and, most importantly, production of better decisions which further the national policy to protect and enhance the quality of the human environment.

These regulations are designed to ensure that the action-forcing procedures of Section 102(2) of NEPA are used by agencies to fulfill the requirement of the congressionally mandated policy set forth in Section 101 of the act. Since these regulations are applied uniformly to all federal agencies, this will minimize misinterpretation, redundancy, and misapplication. Also, time required to learn these regulations and review these documents will be minimized.

Various documents required to comply with the NEPA process and CEQ regulations are described herein.

Environmental Documents

The CEQ regulations require that a number of environmental documents be prepared during the project planning and development process. These documents include the following (3):

1. Notice of intent
2. Environmental assessment
3. Draft environmental impact statement
4. Final environmental impact statement
5. Finding of no significant impact
6. Record of decision

This section will identify each of these documents and discuss briefly the differences among them.

Notice of Intent. This refers to a notice that an EIS will be prepared and considered. This notice is published in the *Federal Register*; and all timing for document processing is tied to this publication date. This notice shall briefly

1. Describe the proposed action and possible alternatives
2. Describe the agency's proposed scoping process, including whether, when, and where any scoping meeting will be held
3. State the name and address of the person within the agency who can answer questions about the proposed action and the EIS

Environmental Assessment. An environmental assessment is also referred to as an environmental impact assessment or EIA. This is a concise public document prepared by a federal agency that serves to

1. Provide sufficient analysis for determining whether an EIS or a “finding of no significant impact” is needed
2. Aid in the agency’s compliance with the NEPA when no EIS is necessary
3. Facilitate preparation of an EIS when one is necessary.

Its format is similar to that of an EIS, except that its summary, lists of preparers, and appendix may be eliminated, and the level of detail required is less than that of the EIS. In the long run, it would be desirable to use the same general format for an EIA as is used for an EIS.

Draft Environmental Impact Statement. The draft EIS (DEIS) is a detailed written statement required by Section 102(2)(C) of NEPA. The DEIS undergoes a formal review process outlined by CEQ regulations.

Final Environmental Impact Statement. The final EIS is a detailed written statement as required by Section 102(2)(C) of NEPA, and it incorporates the results of a formal review of the draft EIS.

Finding of No Significant Impact. This document, referred to as a FONSI, is prepared by a federal agency and briefly presents the reasons why an action, not otherwise excluded, will not have a significant effect on the human environment and for which an EIS, therefore, would not be prepared. It should include either the environmental assessment or its summary and other related environmental documents.

The relationship between the documents discussed in the preceding sections is described in Figure 10.2.

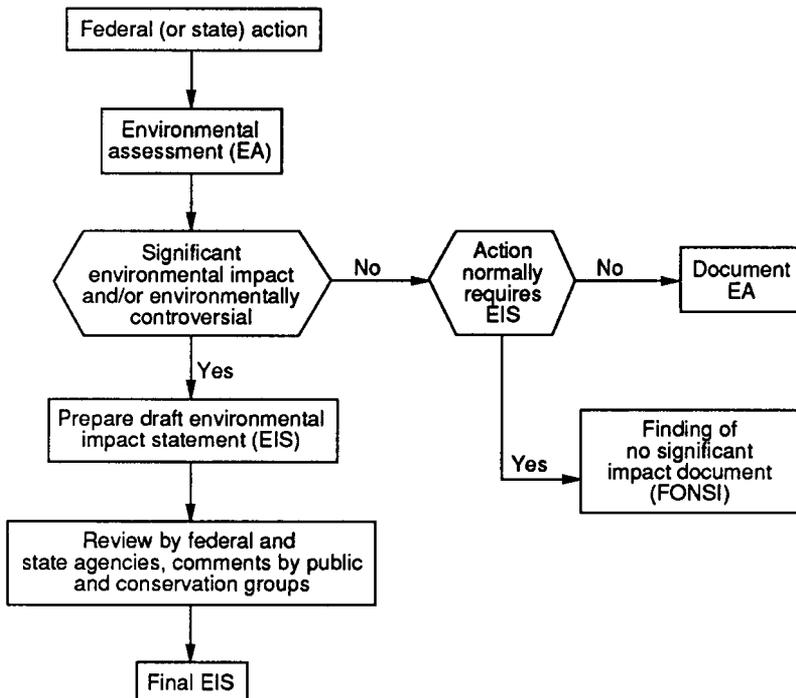


FIGURE 10.2 EA/EIS and FONSI relationship.

Record of Decision. At the time of its decision or, if appropriate, its recommendation to Congress, each agency should prepare a concise public record of decision. This record may be integrated into any other documentation that is prepared by the agency for a similar purpose. This record should include

1. A statement of what the decision was.
2. Identification of all alternatives considered by the agency in reaching its decision, including specification of alternatives that were considered environmentally preferable. An agency may discuss preferences among alternatives based on factors related to economic and technical considerations and to agency statutory missions. The agency should identify and discuss all such factors, including any other essential considerations of national policy that were balanced by the agency in making its decision; the agency should state how much these considerations entered into the decision-making process.
3. A statement of what practicable means to mitigate environmental damage under the selected alternatives were included in the implementation of the action. If some practicable mitigation techniques were not included, the reasons for their exclusion should be stated. A monitoring and enforcement program designed to carry out the mitigation techniques identified should be summarized.

EIS Format and Content

The CEQ has provided a streamlined standard format for all agencies to follow unless the agency has determined that there is a compelling reason to do otherwise. An outline for the CEQ-prescribed EIS format and content is provided in Figure 10.3. The following discussion outlines the content of each section.

Cover Sheet. The cover sheet should not exceed one page. Information required for the cover sheet is outlined in Figure 10.3. An example cover sheet is also included in Figure 10.4.

Summary. Each EIS is required to contain a summary that adequately and accurately summarizes the statement; it is suggested that the EIS format be used for organizing the summary. The summary should stress major conclusions, areas of controversy (including issues raised by agencies and the public), and the issues to be resolved (including the choice among alternatives). In order to stress and highlight these points, it might be desirable to cover these topics as distinct subsections of the summary.

Table of Contents. Extra effort in organizing a table of contents can help the reviewers of the document considerably. All headings and subheadings should be identified with the appropriate page numbers. Lists of figures and tables with appropriate page numbers can also be included. While it is common practice to define each abbreviation, scientific notation, or Greek symbol used for the first time in the text, many readers reviewing or analyzing the document have difficulty with these abbreviations and symbols because they are not familiar with them. It is suggested that these items be listed and defined as an inclusion in the table of contents section.

Purpose and Need. The EIS should contain a brief description of the need or requirement to which this action is responding. The purpose or objective for the action should relate directly to the need or requirement discussed.

Alternatives, Including the Proposed Action. CEQ considers this section to be the heart of the EIS. Some information for this section may have to be prepared after other sections are completed (for example, Section 7). In fact, many parts of the EIS are interdependent, and the preparers may have to work on more than one section at a time and make changes as different sections are analyzed and completed.

A general outline for this section is given in Figure 10.3. In describing the various alternatives consid-

- 1 COVER SHEET
 - Title of the action.
 - Its Location.
 - EIS designation: Final __ , Draft __ , Supplement __ .
 - Lead agency and cooperating agencies.
 - Agency point of contact (name, address, and telephone no.).
 - Date by which comments must be received.
 - Abstract (one paragraph).
- 2 SUMMARY (NTE 15 pages). (NTE = not to exceed)
 - Summarize EIS; use EIS format.
 - Conclusions.
 - Areas of controversy.
 - Issues to be resolved.
- 3 TABLE OF CONTENTS (suggest NTE 6 pages).
 - Cover all headings and subheadings.
 - List of figures.
 - List of tables.
 - List of abbreviations.
 - List of scientific or Greek symbols.
- 4 PURPOSE OF AND NEED FOR THE ACTION (sec 4–7, NTE 150 pages, 300 in special cases).
 - Need or requirement.
 - Purpose or objective.
- 5 ALTERNATIVES INCLUDING THE PROPOSED ACTION
 - Describe each alternative considered (including alternatives not under lead agency jurisdiction and no action alternative).
 - Alternatives not rigorously explored and reasons.
 - Environmental consequences of alternatives in a comparative form.
 - Preferred alternative.
 - Mitigation measures.
 - Sec 101 and 102 (1) requirements of NEPA.
- 6 AFFECTED ENVIRONMENT
 - Describe affected environment.
 - Necessary description relevant to impacts.
 - Summarize, consolidate, or reference information to minimize bulk.
- 7 ENVIRONMENTAL CONSEQUENCES
 - Direct effects—significance.
 - Indirect effects—significance.
 - Conflicts with other Federal, State, Local, or Native American tribe plans.
 - Energy requirements and conservation potential.
 - Natural or depletable resource requirements and conservation potential.
 - Urban quality, historic and cultural resources, and the design of the built environment—reuse/conservation potential.
 - Mitigation measures not covered under “Alternatives” section.
- 8 LIST OF PREPARERS (NTE 2 pages).
 - Name and qualifications of preparers.
 - Reference preparers of sections where possible.
- 9 AGENCIES, ORGANIZATIONS, AND PERSONS TO WHOM COPIES SENT
- 10 APPENDIX
 - Material prepared in support of EIS.
 - Analysis to support effects.
 - Analytic computations relevant to the decision.
- 11 INDEX

FIGURE 10.3 Outline for CEQ-prescribed EIS content.

PROPOSED ACTION TITLE: _____ RELATED ACTIONS: _____ LOCATION: _____ _____ ENVIRONMENTAL IMPACT STATEMENT: FINAL _____ ; DRAFT _____ ; SUPPLEMENT _____ . LEAD AGENCY: _____ COOPERATING AGENCIES: _____ _____ POINT OF CONTACT: _____ DATE OF RECEIPT OF COMMENTS: _____ ABSTRACT: _____ _____ _____ _____ _____ _____ DATE: _____

FIGURE 10.4 Example EIS cover sheet.

ered, all alternatives not under lead agency jurisdiction should also be discussed. Also, the alternative of “no action” is specifically required to be analyzed. Since those alternatives which are considered by the decision maker are supposed to be included in the EIS, there might be some difficulty in providing sufficient detail regarding an alternative which is not under the jurisdiction of the lead agency decision maker.

Usually, there are many alternatives which are considered initially, but not explored vigorously. Reasons for eliminating such alternatives must be included.

From the information developed in Section 7 (Environmental Consequences), environmental consequences of alternatives considered should be displayed in a comparative form. The purpose of this display is

to sharply define the issues involved and to provide a clear basis for choosing among alternatives. The agency and public decision makers can quickly compare and analyze the alternatives from this comparative display. The agency can identify the preferred alternative or alternatives in this section unless another law prohibits the agency from expressing such a preference.

Means to mitigate adverse environmental impacts can be included here; however, it would seem appropriate to include mitigation measures only for preferred alternatives. A summary of this information on mitigation measures must be included in the record of decision. Also, for mitigation measures to have any meaning, a plan and a framework for including these measures in the actual project or action must be developed.

For the implementation of these regulations, the CEQ has required an analysis that should show how alternatives were considered and whether decisions based on these alternatives will or will not achieve the requirements of Sections 101 and 102(1) of NEPA and of other environmental laws and policies. Since the CEQ considers this section to be the heart of the EIS, it would be desirable to include this information here.

Affected Environment. This section is what is commonly referred to as the environmental baseline. It is important to note that the baseline is not necessarily static. The affected environment, or the baseline, should be described succinctly, and the information included should be relevant to the impacts discussed. In the past, many EISs were prepared using verbose descriptions of the baseline; consequently, the CEQ has emphasized in its new regulations that this information be consolidated and, if necessary, other documents referenced in order to minimize EIS bulk.

Environmental Consequences. This section forms the scientific and analytic basis for the comparison of alternatives included in the alternatives section. Environmental consequences or impacts to be considered should include direct, indirect, and induced impacts in the biological, physical, social, economic, and cultural environments. The discussion should include adverse environmental effects that cannot be avoided, relationships between short-term uses of the environment and the maintenance and enhancement of long-term productivity, and any irreversible or irretrievable commitments of resources that would be involved should the action be implemented. This section should not duplicate the discussion included in the alternative section. Environmental consequences resulting from the proposed action may be organized as shown in the outline (Figure 10.3). Any mitigation measures not fully covered in the discussion of alternatives may be further described here.

List of Preparers. Names and qualifications (expertise, experience, professional disciplines) of persons who were primarily responsible for preparing the EIS or developing background papers or basic analysis components of the EIS should be included. Where possible, persons responsible for a particular analysis or section of the EIS should be so identified. This list of preparers will

1. Provide a basis for evaluating whether a systematic, interdisciplinary approach was used in preparing the EIS
2. Increase the accountability and professional responsibility of those who were, in fact, responsible for preparing any part of the EIS
3. Give due credit to and enhance the professional standing of the preparers

Agencies, Organizations, and Persons to Whom Copies Are Sent. The list may serve as a check for both preparers and reviewers in determining the range and adequacy of assessment coordination and solicitation of comments.

Index. Particularly for lengthy or complex statements, an index may facilitate the review and evaluation process. Since the indexing process can become a quite extensive undertaking, many preparers choose to omit this section in favor of expanded depth in the table of contents.

Appendix. The appendix should not become a dumping ground for inclusion of irrelevant or unnecessary paperwork. Only the material that is prepared in support of the EIS and material substantiating analysis in the main body of the EIS should be included. It is often necessary to perform extensive computations to determine some impacts. It would be appropriate to include such analytic computations and the scientific basis for the computation in the appendix. Any material that is generally available in another document can be referenced and need not be included in the EIS. Among other things, the CEQ regulations differ in this regard from the old guidelines. The old guidelines required that the EIS document be an independent, complete document, and not require undue reference to other documents. Also, the appendix may be circulated with the EIS or be made available upon request.

Processing Environmental Documents

After an environmental document has been prepared, processing depends on the type of document and on specific agency requirements. General processing requirements are outlined as follows.

Notice of Intent. This document should be published in the *Federal Register* “as soon as practicable” after the decision to prepare an EIS and before the scoping process. (Exception: When there is a lengthy period between the agency’s decision to prepare an EIS and the time of actual preparation, the notice may be published at a “reasonable time” in advance of draft DEIS preparations.)

Environmental Assessment. An EA is an “intermediate” document. If it is determined that no statement is required, then a FONSI results and the EA presents the substantiating evidence to support that conclusion. The EA or a summary is included in the FONSI. If a statement is required, the EA forms the basis for structuring the EIS.

Draft EIS. Draft statements shall be filed with the EPA (Attention: Office of Federal Activities [A-104], 401 M Street SW, Washington, D.C. 20406). At the same time, they are transmitted to commenting agencies and made available to the public. EPA shall deliver one copy of each statement to the CEQ, thus satisfying the requirement of availability to the President. The EPA will publish a notice of this filing in the *Federal Register* and will review the document as described in a later section of this chapter.

Office of Management and Budget Circular A-95, through a system of clearinghouses, provides a means of securing the views of state and local environmental agencies. After preparing a draft EIS and before preparing the final EIS, the agency should obtain comments from the following groups:

- Federal agencies that have jurisdiction by law or special expertise
- State and local agencies authorized to develop and enforce environmental standards
- Native American tribes, when the action affects them or their reservations
- Any agency that has requested the draft EIS
- The applicant, if the federal agency is issuing a lease, license, permit, or entitlement
- Public and conservation groups affected by or interested in the action

Federal agencies with jurisdiction by law or special expertise with respect to any environmental impact involved are required to comment on the draft EIS. A federal agency or a cooperating agency may reply that it has no comment. In order to reduce unnecessary delays, agencies are required to provide their comments within the time period specified in the regulations.

The sequence of processing activities for draft and final EIS documents is shown in Figure 10.5.

Final EIS. The agency preparing the final EIS should assess and consider comments received on the draft statement both individually and collectively. The agency should take advantage of this input and make the

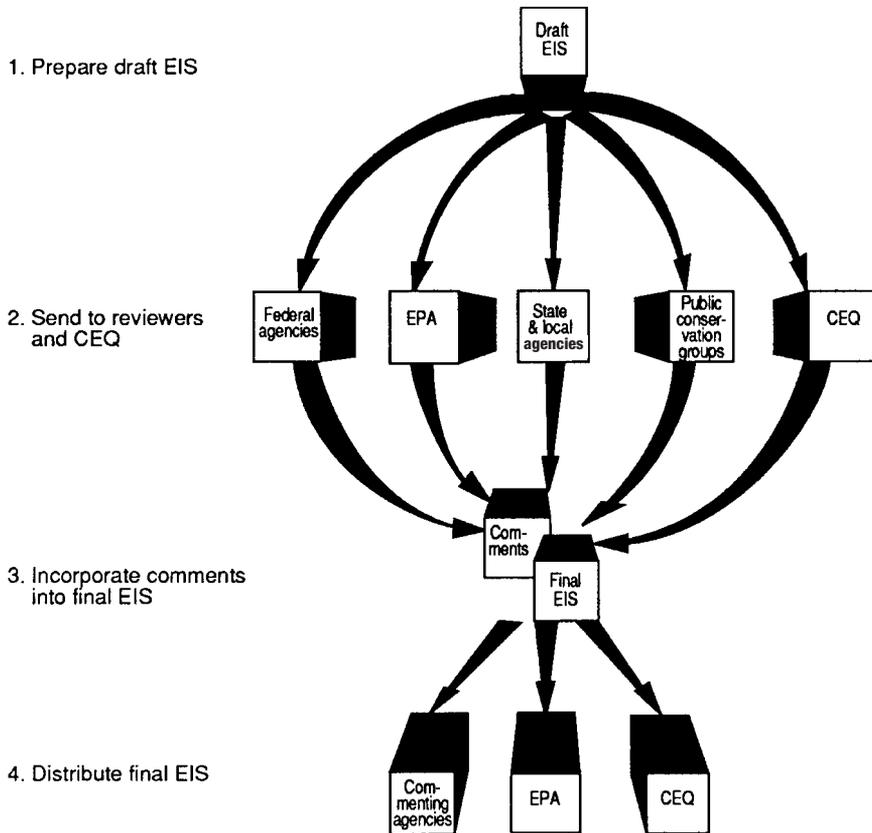


FIGURE 10.5 EIS processing.

necessary changes. When the comments do not warrant further agency response, an explanation should be provided.

All substantive comments received on the draft EIS should be attached to the final EIS. Voluminous comments may be summarized. If changes in response to the comments are minor, only the comments, responses, and changes made need to be circulated. In other words, there may be no need to rewrite the draft EIS; instead, the draft EIS, along with this supplement prepared in response to the comments received, would constitute the final EIS.

The CEQ regulations require that the final EIS, together with comments and responses, or summaries thereof, be filed with the EPA, Office of Federal Activities, in Washington, D.C. The EIS should be made available to commenting agencies and the public at the same time it is filed with the EPA (see Figure 10.4). To comply with the requirements of making the EIS available to the President, the EPA again will deliver one copy of the statement to the CEQ.

Finding of No Significant Impact. The agency should make this document available to the affected public upon request. In certain limited circumstances, which the agency may cover in its NEPA procedures, the agency should make this document available for public review (including state- and areawide clearinghouses) for 30 days before the action begins.

Record of Decision. This document may be integrated into any other record prepared by the agency, including that required by OMB Circular A-95 (Revised), Part I, Section 6(c) and (d), and Part II, Section 5(b)(4). Specific documentation, processing, and other follow-up procedures may vary, but the record of decision must be made available to the public.

Timing of Agency Action

Each week, the EPA will publish a notice in the *Federal Register* of the EISs filed during the preceding week. The minimum time periods set forth by the CEQ should be calculated from the date of publication of this notice. No decision on the proposed action should be made or recorded by a federal agency except as follows:

1. Ninety days after publication of the notice for a draft EIS.
2. Thirty days after publication of the notice for a final EIS. However, there is an exception for those rule-making actions where an agency may announce a preliminary decision at the same time an EIS is filed. In such cases there must be a real opportunity to alter the decision. This means that the period for appeal of the decision and the 30-day period required for the EIS process may be concurrent. An agency engaged in rulemaking under the Administrative Procedures Act or other statute for the purpose of protecting the public health or safety may waive this 30-day period.
3. The minimum 90 days required between the draft EIS and the final action (or recording of the decision) and the 30-day waiting period after the EIS may be concurrent. However, a minimum of 45 days must be provided for comments by other agencies and the public.
4. The lead agency may extend these prescribed time periods. The EPA may reduce the prescribed time periods at the request of the lead agency due to compelling reasons of national policy. Also, the EPA may extend the time periods at the request of other federal agencies (other than the lead agency) in consultation with the lead agency. The EPA is required to notify CEQ of any such extension or reduction of the time periods. There are some other restrictions as well, and these are further described in the CEQ regulations.

Ordinarily, until an agency issues a record of decision, no action concerning the proposed action may be taken that would have an adverse environmental impact or limit the choice of reasonable alternatives.

PUBLIC PARTICIPATION

CEQ regulations state that agencies should make diligent efforts to involve the public in preparing and implementing their NEPA procedures. The council encourages federal agencies to use innovative techniques beyond the public hearing process to provide for effective public participation early in the planning stages of federal projects and actions. Presented in the CEQ regulations are many administrative techniques that federal agencies can use to obtain such participation (3). The extent of public involvement would naturally depend on the scope of the project. For example, in the case of an action with effects of national concern, notice should include publication in the *Federal Register* and notice by mail to national organizations reasonably expected to be interested in the action; in cases of actions having effects that are primarily of local concern, state- and areawide clearinghouses, as described in OMB Circular A-95, can be used.

The notice of intent required in the CEQ regulations serves as an early warning or early notice system for informing the public of upcoming federal actions to which they may want to provide input. This is followed by the "scoping process," which takes place early in the planning stages, into which input by federal, state,

and local agencies and other interested persons can be provided. This input assists in determining the scope of significant issues to be addressed in the EIS. In addition, comments are specifically solicited by the federal agencies on a draft EIS, and these comments are carefully evaluated and considered in the decision-making process. This provides another positive vehicle for public participation.

Effective Public Participation

Effective public or community participation has been defined as occurring when a community acts with full information, equal access to decision-making institutions, and is able to implement its jointly articulated objectives (2). Based on this definition, several important requirements should be met to attain effective public participation.

First, the public should have easy access to all project-related information. There is often considerable misinformation about the nature of most proposed projects, even when the withholding of information is not practiced deliberately. This lack of communication precludes effective citizen participation in many cases.

Second, the community members must have access to the decision process. Allowing or encouraging community involvement in problem identification and discussion, without influence on the ultimate decision, is not very useful to the community or the agency.

Third, for community participation to be effective, the input provided by the citizenry should result in a course of action consistent with the desires and needs of their fellow community members. The agency must have the power to act on behalf of the citizens, and the decisions must reflect the joint objectives of the agency and the community.

The elements of an effective participation system are

1. Information exchange
2. Access to decision making
3. Implementation powers

Various types of communication exchange provide the elements of an effective program. For a communication technique to function as a public participation tool, it must allow for citizens to become involved in decision making. This definition means that techniques that allow only one-way communication, such as newspaper articles, are not very useful. Newspaper articles may, however, be one prerequisite communication step in a public participation program that includes other forms of interaction with a well-informed public. A wide range of techniques contain some or all of the characteristics necessary for a public participation program.

Public Information and Public Involvement

Public information techniques and the level of public involvement vary depending upon the stage of development of the project, significance of the project to the public, and potential for environmental impacts. In the study initiation stage, public information techniques include

1. News releases, advertisements, and announcements through the news media
2. Mailings to individuals or groups
3. Information briefings to elected representatives and governmental agencies

During the planning stages, information and involvement techniques include the notice of intent announcement followed by the “scoping process” as required by the CEQ regulations. These activities can be

supplemented by brochures and information sheets on the project, and site visits and field trips for the interested groups.

During the design stages of a construction project, and during final execution plan development of other types of projects and actions, the information techniques are more specific and are focused on the critical issues identified during the scoping process. Information and involvement techniques include

1. Public hearings
2. Public meetings
3. Presentations to community groups
4. Presentations to governmental agencies representing public interest in areas of their expertise or jurisdictional responsibility
5. Mailings to groups and individuals requesting specific information
6. Workshops and seminars
7. Focused meetings with public interest groups

Participation as a Group Member

Because of the complexity of projects, reviewing and intelligently commenting on an EIS can be a difficult undertaking for an individual. Participation as a group member can provide the necessary knowledge base to comment meaningfully on an EIS and the necessary organizational strength to influence the agency's decision-making process (6). For technical aspects of the project, persons trained in the field should be consulted to assist the public interest group. General statements without proper scientific foundation are not likely to influence the agency or help the group's interests.

Benefits from an Effective Public Participation Program

The catalog of reasons why decision making should not occur in a public forum, but should reside in a central locus, is extensive. Centralized decision making leads to more rapid, cost-effective, conclusive decisions, thus permitting effective and efficient leadership. The military is built on this mode of decision making. Congress is often perceived to act on issues lethargically, appearing to be inefficient and ineffective in comparison with the executive branch. However, this slow action has a distinct benefit; it provides an opportunity for diverse views to be accommodated. This perspective on the value of public participation suggests that decisions made on behalf of the public by centralized agencies can be substantially enhanced by providing channels for public input.

There is a greater likelihood that more viable or innovative alternatives to a project will be identified by opening up the process to the public. Community members are well aware of their own resources, limitations (most often), and problems. The diverse perspectives of the community's citizens provide input that could otherwise be obtained only through extensive fieldwork by the agency sponsoring the project. There is, further, the possibility that there might be a closer integration of planning and development with existing area planning efforts in which major input has already been made by the public. A community may be expected to react unfavorably when previous input to other pertinent plans is summarily disregarded by agency planners and decision makers.

Active public involvement can also ensure that the final product, which the community helps to develop, will be successfully implemented. Implementation is much more likely where the community has taken an active concern in planning problems and has played an important role in generating and evaluating alternative solutions.

ENVIRONMENTAL IMPACTS

An environmental impact may be defined as a change in one or more of various socioeconomic and biophysical characteristics of the environment. Environmental assessment is, therefore, a prediction of the probable changes which may result from a proposed or impending action. The following discussion describes some of the considerations to be made in the projection of environmental impacts.

Impact Types

With environmental impacts defined as changes in any component of the biophysical or socioeconomic environment, an impact may be positive, negative, or it may have both beneficial and detrimental aspects simultaneously. Impacts may be classified as direct, indirect, or cumulative.

Direct Impacts. Direct impacts are caused by a specific action and occur at the same time and location as the action. As an example, erosion resulting from runoff from a newly cleared construction site is a direct impact. Direct impacts are also referred to as primary impacts.

Indirect Impacts. Indirect impacts are caused by an action but are usually either (1) later in time or (2) further removed in distance. Indirect impacts, also called secondary impacts, may include growth-inducing effects and other effects related to induced changes in the pattern of land use, population density, or growth rate and related effects on air, water, and other natural systems including ecosystems. The adverse effects on the aquatic ecosystem caused by the erosion and soil loss described in the preceding paragraph are an example of an indirect impact.

Cumulative impacts. A cumulative impact is one that results from the incremental impact of the action when added to other past, present, and reasonably foreseeable future actions regardless of what agency or person undertakes such other actions. Cumulative impacts might result from individually minor or innocuous actions which may be collectively significant when taking place over a long period of time. Reservoir siltation and ultimate reduction in storage volume resulting from the erosion described in the preceding paragraphs is an example of a cumulative impact.

Identification of Impacts

The identification of environmental impacts is the essence of the environmental assessment process. In order for impact identification to be accomplished, it is necessary to follow a sequence of steps as outlined below

1. It is first necessary to develop a complete understanding of the proposed action incorporated in this portion of the analysis. A detailed description of what needs to be done or accomplished is required. What kinds of materials, workers, and/or resources are involved? Where will the action take place? When will the various activities associated with the project occur?
2. It is then necessary to gain a complete understanding of the affected environment. What resource factors in the biophysical and/or socioeconomic environments exist and are likely to be changed or affected by the action?
3. Finally, it is necessary to superimpose the proposed action on the affected environment and project possible impacts on the environmental characteristics, quantifying changes whenever possible and aggregating the impacts in order to prepare an environmental document describing the anticipated changes.

Several methods have been developed that may be utilized to accomplish the analysis in a systematic fashion. These methodologies will be discussed in a later section. It should be noted that identification of environmental impact must be accomplished with the understanding that the environment is in a dynamic state. That is, changes are occurring in virtually all aspects of the environment on a continuing basis. It is the function of the environmental assessment process to consider only those changes or impacts that would be expected to result from the project.

Measurement of Impact

It is desirable to quantify impacts wherever possible. This may be accomplished through the use of instrumentation, data acquisition, application of models or other acceptable quantification techniques. In situations where quantitative measurements are not possible, impacts may be described through the use of qualitative measurements. Relative changes and general comparisons, based upon expert judgment, may be utilized in the absence of adequate measurable parameters.

Aggregation

Prior to reporting results of an environmental assessment in an environmental document, it is necessary to aggregate or combine the impacts identified with the various aspects of the proposed project. In this aggregation two problems normally occur. The first involves the wide range of environmental characteristics or resource factors that are utilized to describe the biophysical and socioeconomic environment. The traditional problem of comparing “apples and alligators” results. The second problem involves the need to aggregate impacts of various activities on a single resource factor. Finally, the combination of aggregative effects must be utilized to develop a comparative discussion for the decision-making process in the evaluation of different alternatives.

RESOURCE FACTORS

In order to facilitate the identification and description of impacts of the environment, it is useful to describe the environment in specific terms. Resource factors or “attributes” provide a mechanism for impact identification, measurement, and aggregation. In most assessment methodologies, an initial, basic list of environmental categories is utilized to develop an expanded list of detailed attributes that are specific and relevant to the proposed project and its alternatives in a given range of environmental settings. The discussion that follows provides a basic listing of formal resource factor categories and a higher level of attributes in each category. These listings may be expanded and/or reduced as particular circumstances warrant.

Categories of Environmental Resource Factors

Basic environmental resource factors include both biophysical and socioeconomic considerations. For convenience, the following discussion is broken into seven areas, with a short description of detailed attributes which may be considered in each category. The seven areas are

1. Atmosphere
2. Water
3. Land
4. Biological environment

5. Sound
6. Human aspects
7. Economic aspects

It needs to be emphasized that the listing is an example set. Other resource factors may be added depending on the complexity of the project and level of detail necessary for impact analysis.

To make a list of attributes useful, it is desirable to further define them. For most of these attributes, a writeup covering the following items is available in Ref. 2:

1. Definition of the attribute
2. Activities that affect the attribute
3. Source of effect
4. Variables to be measured
5. How variables are measured
6. Data sources for the attribute baseline information
7. Skills and instruments required to collect the data
8. Evaluation and interpretation of data
9. Geographical and temporal limitations
10. Mitigation of impacts
11. Secondary effects
12. Other comments and additional references

Atmosphere. Impacts to air quality frequently head the list of concerns associated with environmental assessment. This is particularly so when such impacts are visible and/or of a toxic nature, leading to potential health effects.

Diffusion Factor. The diffusion factor is related to the structural aspects of the air environment. It reflects such components as vertical temperature structure, wind speed, stability, topography, and climatic conditions. An understanding of the diffusion factor is necessary in determining the extent to which air pollution can build up in a given region.

Particulates. Particulates, one of the most prevalent air pollutants, exist in the form of individual particles suspended in the air. Ranging in size from 0.01 to 100 μm , particulates can produce health effects, visibility problems, structural and material deterioration, and property value reductions.

Sulfur Oxides. Generated primarily by the combustion of fossil fuels, sulfur oxides are usually a combination of sulfur dioxide, sulfur trioxide, sulfuric acid, and sulfurous acid. Undesirable results of sulfur oxides include health effects, increased corrosion of metals, plant injury, and production of acid rain.

Hydrocarbons. Hydrocarbons include several organic compounds that are produced when petroleum fuels are burned. Automobile exhausts produce approximately one-half the hydrocarbons emitted to the atmosphere. Other hydrocarbons are produced from natural sources, such as decomposition of organic matter on land and in swamps and marshes. The primary concern associated with hydrocarbons lies in their role in the formation of photochemical oxidants and smog.

Nitrogen Oxides. Nitrogen oxides are produced as a result of high-temperature combustion. In combination with hydrocarbons, they produce photochemical smog. Although direct health effects are uncertain, nitrogen oxides can affect vegetation, produce corrosion, and damage synthetic fibers and clothing. Furthermore, nitrogen oxides may contribute substantially to the acid rain problem.

Carbon Monoxide. Carbon monoxide is produced by the incomplete combustion of carbonaceous materials in fuels, particularly in space heating and burning of refuse. Carbon monoxide is one of the most widely distributed and most commonly occurring air pollutants. Adverse health effects are related to carbon monoxide poisoning. However, vegetative or material effects usually are minimal.

Photochemical Oxidants. Products of atmospheric reactions between hydrocarbons and nitrogen oxides

which are initiated by sunlight are called photochemical oxidants. Ozone is a product of these reactions, as is peroxyacetyl nitrate (PAN). Although the toxicological potential of photochemical oxidants on animal and human receptors is uncertain, acute and chronic injury may occur to vegetation. Also, photochemical oxidants may lead to the deterioration of materials such as polymers and rubber.

Hazardous Toxicants. The release of toxic substances such as elements or compounds of arsenic, asbestos, barium, chromium, lead, nickel, tungsten, radioactive wastes, mercury, and phenols may cause serious damage to the health and welfare of an exposed community. These materials can create serious health hazards and diseases of a chronic nature.

Odors. Although most malodors are considered harmless, they may cause loss of personnel and community pride, loss of social and economic status, discomfort, nausea, loss of appetite, and insomnia.

Water. The impacts on water quality, which are of particular importance in many projects, are of greatest concern when the results reduce the beneficial use or the aesthetic appeal of water supplies.

Aquifer Safe Yield. The dependability of groundwater from a quantitative standpoint is dependent upon aquifer safe yield. This term relates the consumption rates to the recharge rates. If consumption is greater than recharge, the aquifer is, in effect, being mined—an undesirable situation.

Flow Variations. Changes in the velocity of flow and in discharge rates are important from both quantitative and qualitative standpoints. Nutrient transport, silt and sediment loading, and downstream effects on aquatic life may result from changes in upstream land use.

Radioactivity. Since ionizing radiation is injurious when absorbed in living tissues in concentrations substantially above that present in natural background, it is necessary to prevent excessive levels of radiation from reaching any organism. Radioactive sources may include power development, industrial operations, medical laboratories, and material disposal.

Suspended Solids. Suspended solids contained in water are those that are not in solution but which may be removed through laboratory filtration tests. Solids may be floating, settleable, or nonsettleable, and may contain organic or inert substances. Suspended solids may lead to undesirable aesthetic conditions, difficulties in water treatment and adverse impacts on fish and other aquatic life.

Thermal Discharge. When water is released into an environment at temperatures much higher or lower than those of the ambient conditions, “thermal pollution” may result. Since water temperature is closely associated with physical, chemical, and biological conditions, the aquatic ecosystem may be significantly disrupted as a result of a thermal discharge.

Acid and Alkali. Discharge of wastes which significantly alter the pH of the aquatic environment may be extremely damaging to aquatic life. Social and economic losses in terms of reduced productivity, declining recreational activity, and treatment costs necessary to correct pH-related problems may also result.

Biochemical Oxygen Demand (BOD). BOD is an indirect measure of the amount of biological degradable organic material present in water. Dissolved oxygen will be depleted in the process of satisfying BOD requirements. As oxygen is utilized, water conditions become septic and other water quality and aesthetic problems result.

Dissolved Oxygen (DO). DO is necessary in water to maintain aerobic conditions. Absence of DO leads to the development of anaerobic conditions with their associated odor and aesthetic problems. Adequate DO levels are necessary in surface waters to maintain growth of fish and other desirable aquatic life.

Dissolved Solids. Total dissolved solids or TDS is the sum of carbonates, bicarbonates, chlorides, sulfates, phosphates, nitrates, and various salts of calcium, magnesium, sodium, potassium, and other substances. These salts in solution may alter the physical and chemical nature of water. High amounts of TDS are objectionable because of physiological effects, mineral tastes, or economic consequences.

Nutrients. Nutrients are, in essence, fertilizers; they include forms of nitrogen and phosphorus which are responsible for eutrophication of water supplies. Eutrophic conditions are typified by dense algae growth, surface water scums, foul-smelling water, periods of anaerobic conditions, and fish kills. Many activities increase the supply of nutrients to natural waters.

Toxic Compounds. Toxic compounds may occur in wastes containing heavy metals, organic chemicals, cyanides, sulfides, and pesticides. Adverse effects that can result include damage to aquatic life, reduction of beneficial use of water, and more and extensive treatment requirements.

Aquatic Life. Aquatic life refers to the population of species which exists in a state of dynamic balance in water supplies. Since any pollution tends to upset this natural balance, changes in the many previously listed attributes will influence aquatic life.

Fecal Coliforms. Fecal coliforms are single-celled bacteria found in the intestinal tracts of humans and other warm-blooded animals. Their presence in water is an indication of possible contamination by human wastes and, thus, that the water may contain bacteria, viruses, or other disease-causing organisms.

Land. Most food and fiber resources come from the land; consequently, it must be used in a properly planned and controlled manner. Many activities affect the land resource and may preclude alternative uses.

Erosion. Erosion is a process whereby soil particles are dislodged and transported to another location by wind or water action. Erosion may be highly destructive and render land unproductive and/or may pose a physical barrier to other activities. Water courses downslope from eroded areas may be adversely affected by excessive sediment loads. Areas subjected to erosive forces may become barren and aesthetically unappealing.

Natural Hazards. Natural hazards are those occurrences, brought about by the forces of nature, that result in discomfort, injury, or death to humans; damage or destruction to physical structures; detrimental changes in physical character of the land, water, and air; or damage to plant and animal life in the affected area. Natural hazards include floods, earth slides, wildfires, and volcanos. Other hazards, such as earthquakes and hurricanes, may cause great personal and physical damage when they occur in areas of concentrated human activity. In some cases, the occurrence of these hazards may be accelerated through human activity, e.g., flooding. In other situations, the occurrence is unaffected by human activity; however, the degree of damage done may be dictated by the location and magnitude of population centers and other developments.

Land Use Patterns. Land use patterns are natural or imposed configurations that result from spatial arrangement of the different uses of land at a given time. Historically, land use patterns evolve as a result of (1) changing economic considerations, (2) legal restrictions, such as zoning, and (3) changes in existing legal restrictions. Compatibility of use between one parcel of land and adjacent properties becomes a controlling issue.

Biological Environment. The biological environment and its attributes can be utilized interchangeably with the concept of ecology and the ecosystem. Convincing evidence exists that species diversity in an ecosystem is closely related to the stability of that system. Greater species diversity indicates that the ecosystem is more able to resist disturbance and stress. Although humans have made progress in providing for their needs, the results are often accomplished at the expense of the total ecological balance of the environment. Since this balance is crucial to the long-term viability of human beings, species diversity must be maintained.

Large Animals (Wild and Domestic). Large animals are those, both wild and domestic, that weigh typically more than 30 lb when fully grown. These populations may be affected by food supplies, cover, water, natural and artificial barriers, noise, and other activities.

Predatory Birds. Birds of prey are flesh eaters that typically obtain their food by hunting, killing, and eating small animals, other birds, and fish. These birds are frequently impacted by loss of habitat, poisonings, or intrusion into or near nesting areas.

Small Game. Small game includes both upland game birds and animals that typically weigh less than 30 lb as adults and which are hunted for sport. Many human activities result in habitat destruction or in other impacts to these animals.

Fish and Shellfish. Fish are cold-blooded aquatic animals that obtain oxygen through a gill system and inhabit saltwater and freshwater bodies and streams. Shellfish are aquatic animals that have an exoskeleton

shell. Since these species depend directly on water for some or all aspects of their existence, any activity which affects water quality and water level will impact on their well-being.

Waterfowl. Waterfowl are birds that frequent and often swim in water, nest and raise their young near water, and obtain at least part of their food from aquatic plants, animals, and insects. Because they depend directly on water for all or some facets of their existence, activities which affect water quality and water level will also impact waterfowl.

Field Crops. Field crops are those commercially cultivated for the primary purpose of providing food and clothing for humans or food for domestic livestock. Activities which impact field crops include those that alter land use and those which affect crop production, including the application of herbicides or other toxic materials.

Natural Land Vegetation. Natural land vegetation is vegetation that uses soil as its growth medium and that is not the subject of extensive cultural practices. Included are plants, trees, shrubs, grasses, herbs, ferns, and lichens. Native vegetation is established through a series of successional stages. Successional change is slow, and disruption frequently results in the least-desirable plant types becoming the first to be reestablished.

Aquatic Plants. Aquatic plants are those whose growth medium is primarily water, although they may be rooted in soil. They include free-floating plants, all surface and submerged rooted plants, and swamp and marsh vegetation whose roots are periodically or primarily submerged in water. Activities that cause changes in water level or water quality parameters have the greatest impact on aquatic plants.

Sound. The level of sound is an important indicator of environmental quality. Noise has been defined as unwanted sound in the environment.

Physiological Effects. Noise can affect physiology in three different ways. First, internal bodily systems may be affected; for example, extreme noises may constrict blood vessels, increase pulse and respiration rates, increase tension and fatigue, cause dizziness, and cause loss of balance. Second, the hearing threshold may be impacted with temporary or permanent hearing loss resulting. Third, sleep patterns may be disrupted, with resulting irritability and often irrational behavior.

Psychological Effects. Noise exposure can have an impact on an individual's mental stability and psychological response. This response may be manifested with signs of increased annoyance, anxiety, or fear. Important variables of noise that result in psychological effects are its loudness, duration, and frequency content.

Communication Effects. The presence of noise can adversely affect face-to-face conversation and telephone communication. Temporary interference or interruption can be annoying and occasionally harmful to personal well-being.

Performance Effects. The presence of noise can affect the ability of a human to perform mechanical and mental tasks. This is manifested through increase in muscular tension, lapse in attention, diversion of attention, masking of needed auditory signals, and startle response to high-intensity noises.

Human Aspects. This category refers to the way in which people interact with each other and the natural environment. These interactions are frequently complex and their analysis requires careful scrutiny of all activities and interrelationships.

Life-styles. The social activities of humans often take on structural characteristics which eventually cause them to become organizations. The makeup of organizations may vary, depending upon the characteristics, interests, and objectives of the organization's population. The bases for these organizations might be ethnic, political, religious, or occupational. Changes in life-styles may be reflected by activities which affect employment, standards of living, community development, and recreational opportunity.

Psychological Needs. Psychological needs of human beings are primarily those of emotional stability and security. Activities which affect these needs are essentially the same as those which impact life-styles.

Physiological Systems. These systems include anything that is a part of a person's body or that plays a

part in a bodily function. They include respiratory, circulatory, and digestive organs; skeletal and excretory systems; and all parts of the human body that contribute to its effective, efficient functioning. Any activity that can harm or threaten the efficient functioning of any part of the human body must be considered in light of its effect.

Community Needs. A community requires many services including housing; water supply; sewage disposal; utilities, such as gas, electricity and telephone; recreational facilities; and police and fire protection. Any change or impact that occurs will depend primarily on the degree to which a population is affected by a proposed activity.

Economic Aspects. In environmental impact analysis, the potential impact on the economic structure or changes resulting from project activities comes primarily from the direct effect of purchases of goods and services for project activities and from the indirect effects arising from goods and services purchased from payrolls. Effects may reflect impact on industrial and commercial activities, the local government, and the individual.

Regional Economic Stability. Stability refers to a resistance to change in the ability of a region's economy to withstand severe fluctuations or to the speed and ease an economy demonstrates in returning to an equilibrium situation after receiving a shock. The more diverse an economy and the more closely related it is to growth areas of the national economy, the more stable it is likely to be.

Public Sector Revenues and Expenditures. This attribute consists of the annual per capita revenues and expenditures of local and state governments and associated agencies in the region under study. Changes in this factor can be interpreted as a measure of a change in economic well-being of the public sector.

Per Capita Consumption. The annual per capita personal consumption of goods and services by local citizens can be interpreted as a direct measure of personal economic well-being. Increases or decreases in local employment, industrial reduction or deletion, and construction all have the potential to affect per capita consumption.

Environmentally Sensitive Areas

Several resource factors are particularly important due to their tendency to elicit strong public reaction. When impacts occur in these areas, it is essential that an environmental assessment adequately addresses projected impacts due to all project alternatives. Furthermore, the assessment should contain a detailed discussion of mitigation measures which would be implemented should the project be undertaken.

Human Health and Safety. Any activity that can adversely impact human health or increase the risk due to accidental exposure must be considered. There are many possible sources of impacts. These range from activities that might impair the safety of individuals when placed in a working environment to exposure to hazardous chemicals, gases, radiation, noise, or disease.

Threatened Species. Formerly categorized as "rare or endangered," threatened species include all forms of plants and animals whose rates of reproduction have declined to the point where their populations are so small they are in danger of disappearing. Lists of threatened animal species are published periodically by the Bureau of Sport Fisheries and Wildlife of the U.S. Department of the Interior. While animal species are the ones most often cited in the news media, there are many species of plants that also qualify as threatened. If any question exists as to the presence of a threatened species in the area of a project, knowledgeable wildlife biologists or botanists should be called on to verify their presence and to give a preliminary assessment of the impact of the activity on the population of this species.

Nonrenewable Resources. Nonrenewable resource consumption is of particular interest since the consumption or utilization represents a commitment that is potentially irreversible or irretrievable. Fossil fuels,

metals, many building materials, mineral resources, and other materials fall into this category. Impacts on nonrenewable resources may be minimized by economizing on resource requirements, development and use of substitutes, recycling, etc.

Aesthetics. Impacts on aesthetics are those that are perceived through the senses: sight, taste, smell, hearing, and touch. Although aesthetic perceptions usually require the consideration of all senses simultaneously, visual perception is perhaps the most familiar of the areas. Generally speaking, any activity that will change the quality or distinguishable characteristic of the perceived environment can be considered as having an effect on aesthetics. Aesthetic considerations may be complex to evaluate because these perceptions are individual and values for definition become difficult to quantify.

Cultural Resources. Cultural resources may include historic, archeological, and architectural resources, and may take the form of sites, artifacts, or properties. Other forms of cultural resources are associated with behavior patterns and may be associated with customs, traditions, and religious beliefs and practices.

ALTERNATIVES

The analysis of alternatives is an important consideration in the environmental assessment process. CEQ regulations, in fact, state that the required section on alternatives is the “heart” of the EIS.

Types of Alternatives

Specifically included in the alternatives to be considered are (1) the “no action” alternative, (2) all “reasonable” alternatives, (3) the agency’s “preferred” alternative(s), if one or more exist, and (4) mitigation measures not discussed with the previous alternatives.

No Action. Analysis of the “no action” alternative requires a projection of future environmental conditions without the proposed project; in other words, what will happen if “nothing” is done or if the decision is made to simply maintain the status quo.

Reasonable Alternatives. The CEQ regulations call for agencies to “rigorously explore and objectively evaluate all reasonable alternatives, and for alternatives that were eliminated from detailed study, briefly discuss reasons for their having been eliminated.” Furthermore, the regulations state that “reasonable alternatives not within the jurisdiction of the lead agency” are to be included in the analysis (1). There is no clear definition of what constitutes “reasonableness,” although the courts have rendered various decisions that may be utilized in making a determination. Options may include alternatives in terms of routes, sites, timing or scheduling, objectives, and/or methods to accomplish a single objective. Possible exclusions are those alternatives too remote or speculative to be viable, or those unavailable in the proposed time frame. Where sheer guesswork would be necessary in the evaluation, such alternatives may be listed and the reason for lack of detailed analysis included.

Preferred Alternatives. Although agencies are not to commit resources that would prejudice the selection of alternatives before making a final decision, the fact that, due to economic or other reasons, a preferred alternative may exist is recognized by the CEQ regulations. If such preference does indeed exist, it should be so stated. In this manner, the proposed action or preferred alternative can be more easily compared to the other alternatives prior to final decision making.

Mitigation Measures. The minimization of adverse impacts (mitigation) often may be accomplished in a variety of ways. If appropriate mitigation measures are identified but have not already been included in the discussion of the proposed action or alternatives, special effort to include them as alternatives is required.

Alternative Analysis

Each alternative is to be analyzed for environmental impact. The CEQ regulations require that the statement should present the environmental impact of the proposal and the alternatives in comparative form, in order that the issues may be sharply defined and result in providing a clear basis for choice among options by the decision maker and the public (4). Although no format is specified, the requirement to present alternatives in comparative form frequently results in a matrix display of alternatives and summarized impacts, as shown in Figure 10.6.

ASSESSMENT PREPARATION AND REVIEW

The preparation and review of environmental assessments, impact statements, and other documents is accomplished through an interdisciplinary effort requiring the acquisition of background or baseline data and information and development of an accurate and complete definition of the scope of the proposed project. Then, utilizing a methodology to systematically relate activities of the proposed project and the alternatives to the affected environment, the potential impacts are identified and the indicated documentation is prepared. Finally, the documents are reviewed and disseminated for use in decision making. This section provides a brief overview of each of these basic components of the NEPA process.

The Interdisciplinary Team

As specified in the act, NEPA requires the utilization of an interdisciplinary approach. This means that due consideration must be given to all facets of the environment—biophysical and socioeconomic. This is accomplished through the efforts of a group of individuals with expertise encompassing the entire range of resource factors previously discussed participating in the assessment study. Furthermore, these individuals must participate as team members, interacting and cross-communicating so that the result is interdisciplinary as opposed to multidisciplinary. As a general rule, group working sessions frequently become spirited”; however, such exchange is often highly productive and becomes essential to the identification of indirect and cumulative impacts as is required in the analysis.

Baseline Studies

To establish the baseline requires a study of the environmental setting or location and conditions under which the proposed action is to take place. This may include the assembling of previously collected or secondary data, and when necessary, the acquisition of first-hand or primary data. The background studies should incorporate existing conditions, geographic characteristics, and temporal characteristics in such manner that the baseline information serves as a filtering mechanism, eliminating consideration of impacts unrelated to a specific area or site, and aiding in the quantification of impacts where the severity of impact would vary.

Conditions Prior to the Activity. The nature of the impact is determined by the conditions of the environment prior to the activity. This information establishes the status of the attributes prior to the activity at the

project location. Because the measurement and analysis of environmental impact cannot take place without base data, identification of the characteristics of the baseline is critical.

Geographic Characteristics. There may be significant differences in impact on attributes caused by a given activity in different areas. Geographic location is, therefore, one of the factors that affects the merit or relative importance of a particular attribute. For example, the impact of similar projects on water quality in an area with abundant water supplies versus the impact in an area with scarce water resources would differ significantly. The spatial dispersion of different activities introduces an element of difficulty when comparing the impact of one activity with another.

Temporal Characteristics. Time may also pose problems for impact analysis. It is essential to ensure that all impacts are examined over the same projected time period. Furthermore, to adequately compare (or combine) activity impacts, it is necessary that the same time period (or periods) apply.

Scoping

The term “scoping” refers to a process of identification of the important issues that require full analysis and the separation of those issues from the less significant ones that do not require detailed study. It is necessary that the scoping process take place as soon as possible after an agency decides to prepare an EIS. Before completing the scoping process, the lead agency must publish a notice of intent.

The scoping process also serves as a mechanism to identify other environmental review and consultation requirements and to allocate responsibilities among lead and cooperating agencies. Affected federal, state, and local agencies, any affected Native American tribe, the proponent of the action, and other interested persons must be invited to participate in the scoping process.

In determining the scope of environmental impacts, the CEQ requires that agencies consider three types of actions, three types of alternatives, and three types of impacts (4). These include

- A. Actions (other than unconnected single actions) that may be
 1. Connected actions, by which it is meant that they are closely related and therefore should be discussed in the same impact statement. Actions are connected if they
 - a. Automatically trigger other actions that may require environmental impact statements
 - b. Cannot or will not proceed unless other actions are taken previously or simultaneously, or
 - c. Are interdependent parts of a larger action and depend on the larger action for their justification.
 2. Cumulative actions, which when viewed with other proposed actions, have cumulatively significant impacts and should therefore be discussed in the same impact statement.
 3. Similar actions, which when viewed with other reasonably foreseeable or proposed agency actions, have enough in common to provide a basis for evaluating their environmental consequences together, such as common timing or geography. An agency may wish to analyze these actions in the same impact statement. It should do so when the best way to assess adequately the combined impacts of similar actions or reasonable alternatives to such actions is to treat them in a single impact statement.
- B. Alternatives, which include
 1. No action alternative
 2. Other reasonable courses of action
 3. Mitigation measures (not in the proposed action)
- C. Impacts, which may be
 1. Direct
 2. Indirect
 3. Cumulative

Assessment Methodologies

Many methodologies have been developed which allow the user to respond in a substantive manner to CEQ requirements when preparing an EA/EIS. Presented in this section is a discussion of some of these impact assessment methodology types (2).

Ad Hoc. These methodologies provide minimal guidance for impact assessment beyond suggesting broad areas of possible impacts; e.g., impacts on flora and fauna, lakes, and forests, rather than defining specific parameters to be investigated.

Overlays. These methodologies rely on a set of maps of a project area's environmental characteristics (physical, social, ecological, and aesthetic). These maps are overlaid to produce a composite characterization of the regional environment. Impacts are identified by noting the impacted environmental characteristics within the project boundaries.

Checklists. These methodologies present a specific list of environmental parameters to be investigated for possible impacts; they do not require establishing direct cause-effect links to project activities. They may or may not include guidelines for the measurement and interpretation of parametric data.

Matrices. These methodologies incorporate a list of project activities with a checklist of potentially impacted environmental characteristics. The two lists are related in a matrix that identifies cause-effect relationships between specific activities and impacts. Matrix methodologies may either specify actions that impact certain environmental characteristics or may simply list the range of possible actions and characteristics in an open matrix that will be completed by the analyst.

Networks. These methodologies work from a list of project activities to establish cause-condition-effect relationships. They are an attempt to recognize that a series of impacts may be triggered by a project action. These approaches generally define a set of possible networks and allow the user to identify impacts by selecting and tracing out the appropriate project actions.

Combination Computer-Aided Methods. These methodologies, usually developed to assess large federal agency projects, use a combination of matrices, networks, analytical models, and a computer-aided systematic approach to: (1) identify activities associated with implementing major federal programs, (2) identify potential environmental impacts at different user levels, (3) provide guidance for abatement and mitigation techniques, (4) provide analytical models to establish cause-effect relationships to quantitatively determine potential environmental impacts, and (5) provide a methodology and a procedure to use this comprehensive information to respond to the requirements of environmental assessment documentation preparation.

There is an extensive literature that discusses and describes various specific methodologies (2, 7).

Document Preparation

Results of the impact analysis process are organized into one or more of the following documents:

1. An environmental assessment (EA)
2. A finding of no significant impact (FONSI)
3. A draft environmental impact statement (DEIS)
4. A final environmental impact statement (FEIS)

Summary of Environmental Impacts			
Alternative	Air	Water	
1. No action	_____	_____	_____
2. Preferred alternative X	_____	_____	_____
3. Alternative A	_____	_____	_____
4. Alternative B	_____	_____	_____
5. Alternative C	_____	_____	_____

FIGURE 10.7 Example impact summary sheet (6).

It may be useful to consider displaying the results in a way that makes it easy to comprehend total impact from a brief review. One suggested method for doing this is by displaying the impacts on a summary sheet, such as that shown in Figure 10.7.

Details of the specific format for environmental impact analysis documentation are given by individual agency guidelines. These guidelines should be consulted and followed for each analysis.

General comments included in the CEQ regulations regarding the preparation of EISs can be summarized as follows

1. EISs should be analytic rather than encyclopedic.
2. Impacts should be discussed in proportion to their significance.
3. EISs should be concise.
4. EISs should state how alternatives considered and decisions made, based on the EIS, will or will not achieve the requirements of Sections 101 and 102(1) of NEPA and other environmental laws and policies.
5. Alternatives discussed should be limited to those that are expected to be considered by the agency decision maker.
6. The agency should not commit resources prejudicing selection of alternatives before making a final decision.
7. EISs should be a means of assessing the environmental impact of the proposed action, rather than a means of justifying decisions already made.
8. A systematic and interdisciplinary approach should be used to prepare EISs.
9. EISs should be written in plain language and appropriate graphics may be used so that decision makers and the public can readily understand the documents.
10. Public participation should be actively sought during the scoping process and in the early planning stages of the project to identify significant issues.

Document Review

The review of NEPA-related documents takes place at many different levels, for a variety of purposes, and from different perspectives, depending on the reviewers. Five typical review situations and three common areas of concern among the different types of review have been identified (2).

Review Situations. Specific review situations include the following.

Internal Review. Intraagency or “in-house” review whereby the document is examined for glaring errors, legal compliance, style, and technical content prior to public release is standard practice.

Interagency Review. Before preparation of the final EIS, the draft statement must be sent to federal agencies.

1. Having jurisdiction by law or special expertise with respect to any environmental impact involved, or
2. That are authorized to develop and enforce environmental standards
3. In addition, the proponent agency is required to request comments of appropriate state and local agencies authorized to enforce environmental standards
4. Native American tribes when a reservation may be affected, and
5. Any agency making a request.

EPA Review. Draft and final EIS documents are reviewed by the EPA. For draft statements, EPA considers the environmental impact of the action and the adequacy of the statement. Results of the review are published in the *Federal Register*. For final statements, EPA reviews the document with primary attention directed to substantive issues related to environmental impact assessment. Again, results are published in the *Federal Register*.

Public Review. The lead agency must also request comments from the general public. Comments are usually solicited through published notices, public hearings, and contact with known interested conservation groups and individuals.

Review for Decision Making. Finally, the EIS document is reviewed at the agency decision level, and utilized in preparing the record of decision.

Types of Review. Three areas of concern are common to the diverse review situations outlined above.

Administrative Compliance Review. This concern focuses on the document from the standpoint of compliance with NEPA, CEQ regulations, and specific agency document preparation and processing requirements.

General Document Review. This aspect of review is concerned with clearness, completeness, and correctness.

Technical Review. The third aspect is concerned with the technical accuracy and adequacy of the document.

Several approaches for reviewing NEPA-related documents have been developed. Most utilize either an independent analysis or predetermined evaluation criteria to score or rate a document. Regardless of the approach taken, the overall review process is essential to the preparation of better documents, and ultimately better decisions.

NEPA AFTER 25 YEARS

Immediately after its enactment, NEPA began to alter the way federal agencies did business. Agencies, responding to NEPA, CEQ guidelines, and subsequent regulations, began developing an infrastructure to respond to the legal and procedural requirements. Legal actions, including landmark court cases, interpreted and further shaped the law. In the years immediately following the enactment of NEPA, the number of EISs filed reached over 2000 per year. That number declined to less than 400 per year in 1989, but appears to have stabilized at approximately 500 per year in the 1990s. What is now known as the “NEPA process” has become a matter of routine, championed or respected by some, tolerated or criticized by others.

On the occasion of the 25th anniversary of the act, the CEQ conducted a reflective study of NEPAs effectiveness and prospects for improvements. Participants in the study included representatives of the original framers of the act and individuals who drafted the CEQ regulations, along with some members of Congress, federal agencies with NEPA implementation experience, state and local agencies, and the public. In addition to evaluating strengths, the participants also considered the potential for improving the current implementation of NEPA, identifying areas “. . . ripe for innovation, re-invention, and improvement” (9)

The study concluded that NEPA has been a success because it (1) causes agencies to examine the environmental consequences of their actions before they make final decisions, and (2) provides a framework for collaboration between the agencies causing the actions and those who will be affected by the consequences (environmental, social, and economic impacts) of their decisions. The participants identified five elements critical to NEPA effectiveness and efficient implementation, and pointed out problem areas associated with each element.

Strategic Planning

The NEPA process provides a framework for integrating environmental concerns directly into agency internal planning, policies and programs at an early stage. Unfortunately, the process is often either initiated too late to be effective in agency planning and decision making or applied to individual projects instead of evaluating the programs that call for those projects. Agencies then encounter delays, incur increased costs or end up with fewer alternatives available for consideration, thereby denying NEPA its full strategic planning value.

Public Information and Input

NEPA significantly increased public information and input into agency decision making by opening opportunities for involvement. In many cases, agencies have actively solicited ideas and concerns from the public to improve the quality of projects and reduce adverse impacts on the environment. In others, the desired level of public involvement has not been achieved, with the result being a sense on the part of citizens of dissatisfaction, frustration, and distrust, sometimes leading to litigation.

Interagency Coordination

NEPA and many subsequent laws have required interagency coordination in an effort to reduce conflicts, reduce duplication of effort, and to improve the environmental planning process. When information is not shared or when processes are uncoordinated, agencies and often the public may be placed in adversarial positions leading to conflicts and delays. Problems are often initiated by conflicting or different timetables, requirements, and modes of public participation.

Interdisciplinary, Place-based Approach to Decision Making

NEPA calls for an interdisciplinary approach to planning and decision making. When agencies bring the full range of biophysical and socioeconomic factors affecting a single place into their management and planning processes, they are in a position to make better decisions with regard to the needs of the community and minimize adverse impacts on the environment. Difficulties include lack of quality environmental baseline data, lack of early participation by resource agencies, and costs and delays associated with data collection. New tools such as modern computers, internet communications, and geographic information systems hold considerable promise for improvements in this area.

Monitoring and Adaptive Management

Science-based and flexible management approaches are essential to the confirmation of predicted impacts. to ensure that mitigation measures are effective and to modify remediation or mitigation processes if necessary. By accepting more uncertainty in the environmental assessment process and adapting quickly to conditions actually encountered during the project operational phases, agencies may be able to decrease project startup time and reduce overall project costs.

The CEQ's goal is to use the experiences of the first 25 years to streamline and improve the NEPA process. By streamlining the process, the CEQ hopes to make it more efficient, to promote the integration of social, environmental, and economic factors into agency decision making, and to ensure openness in government. Additional information is available through its web site (www.whitehouse.gov/CEQ/).

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