SECOND EDITION

WASTEWATER TREATMENT PLANTS

PLANNING, DESIGN, AND OPERATION

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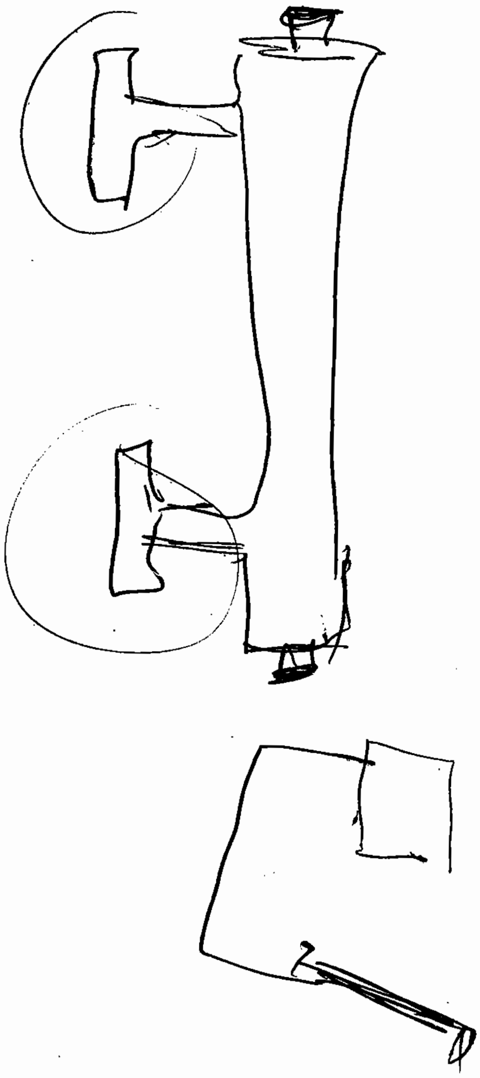
To my wife Yasmin and daughters Zeba and Saba



Contents

Preface

The first edition of this book was published in 1985 and received widespread use in colleges and universities. The book was also used extensively by practicing engineers in both public and private sectors. The widespread acceptance was because of the extensive coverage of planning, design, and operation of wastewater treatment plants with a *practical design approach.* The second edition of this book is prepared with the same *theme,* but extensive updating of the subject matter, illustrations, and design examples has been done to incorporate recent concepts and issues related to wastewater treatment plant design.

To meet the objectives established for this second edition, the author has attempted to (1) present updates of the technological advances in wastewater treatment technology; (2) consolidate the developments in planning and design of wastewater treatment facilities that have evolved as a result of technological advances in the field and as a result of concepts and policies promulgated by the environmental laws and the subsequent guidelines; (3) develop step-by-step procedures for planning, design, and operation for a medium-sized wastewater treatment plant; (4) revise the text and the design examples based on comments and suggestions received from readers to make the book more useful for students, teachers, and practicing engineers; and (5) supply design equations and computational tools that will enhance the design capabilities of students and designers.

This book is divided into 27 chapters. Chapters 1 through 5 are devoted to the basic facts of wastewater engineering. Current and future trends in wastewater treatment technology, basic design factors and effluent guidelines, wastewater characteristics, treatment processes and process combinations, and requirements of predesign studies and facility planning are discussed in detail. Chapter 6 is devoted to facility planning. A model facility plan for a medium-sized wastewater treatment facility is developed. Chapters 7 through 23 are devoted to the design and operation of a medium-sized wastewater treatment facility. Step-by-step design calculations, equipment selection, engineering drawings, operation and maintenance, and plans and specifications of various treatment units are presented. Separate chapters have also been devoted to plant layout, yard piping and hydraulics, instrumentation and automatic controls, upgrading of secondary treatment facility and advanced wastewater treatment processes, small flow wastewater treatment systems, control and treatment of storm and combined sewer overflows, and avoiding errors in plant design. The most extensive revisions in this edition are in the design of bar screen, biological nutrient removal process, UV disinfection, belt filter, and biosolids utilization. Because of enhanced interest in natural systems, on-site disposal systems, and treatment of stormwater runoffs and combined sewer overflows, these topics have been addressed in Chapter 24, and two new chapters have been added. Basic properties of water and wastewater, hydraulic design information, cost equations and procedures, equipment manufacturers, and other related design data are arranged in several appendices.

Metric units are primarily used in this book. Since old plants will be upgraded in the future, the U.S. customary units will continue to be in use for some time to come. Therefore, where possible, both units are used, and proper conversion factors are provided. Complete conversion tables are given in Appendix F.

Acknowledgments

I am very grateful to those who have helped me prepare this book. First, I must thank Walter Chiang and Mike Morrison for their interest and stimulating discussion and response during the development of this book. They reviewed many chapters and made constructive suggestions in process and equipment design. Mike Morrison prepared the initial draft of the chapter on pumping stations. Walter Chiang has been a mentor to me for many years, and his encouragement and inspiration have been the driving force for both editions of this book.

I am especially grateful to many professionals for their assistance. Pete Patel and Michael Graves reviewed the chapter on instrumentation and prepared the simplified control loop diagrams. Glen Daigger, Danna Rippon, and Raj Bhattarai reviewed the sections on BNR facility design. Ted Palit provided assistance with updates on regulations. Guang Zhu conducted material mass balance analysis and assisted with the design example on biological nutrient removal. Shih Pan reviewed the entire manuscript and did a major portion of typing and corrections. S. Chanthikul, David Coberline, and Adam Wighaman assisted with the artwork.

Many professionals, colleagues, and students also reviewed various portions of this text, conducted literature searches, checked calculations, worked out solutions to the problems, and prepared drawings. In particular, I would like to thank Robert Beleckis, Guillermo Charles, David Gudal, Thelma Box, Ernest Crosby, and Max Spindler.

Many equipment manufacturers and their local representatives provided valuable information on equipment details and specifications. Fred Willms, Bob Landry, Lee Rodgers, and Frank Clark arranged for many photographs from the equipment manufacturers. The names and addresses of many equipment suppliers are included in Appendix D. The Department of Civil and Environmental Engineering at the University of Texas at Arlington also provided assistance and support throughout the entire project.

Finally, I must acknowledge with deep appreciation the support, encouragement, and patience of my wife and daughters, who tolerated the agonies that generally accompanied long hours of work over a period of several years.

Although portions of this book have been reviewed by professionals and students, the real test will not come until it has been used in classes and by professionals as a design guide. I shall appreciate it very much if all who use this book will let me know of any errors and changes they believe would improve its usefulness.

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1

Introduction

1-1 HISTORICAL BACKGROUND

In the early 1800s, the United States was a sparsely populated, underdeveloped country. Human waste was generally disposed of by either pit privies or open drainage ditches. The dramatic increase of population and industries in urban areas and public awareness of the connection between human diseases and waste disposal needs made it necessary for municipalities to improve waste management practices. Work on the first sanitary sewer in the United States was begun in Chicago in 1855, 12 years after the world's first sanitary sewerage system was completed in Hamburg, Germany. Various types of wastewater treatment technologies were used: trickling filter in 1901, Imhoff tank in 1909, liquid chlorine for disinfection in 1914, and activated sludge in 1916. By 1948 wastewater treatment plants served some 45 million Americans out of a total population of 145 million.[1,2]

1-2 CURRENT STATUS

In 1956 Congress enacted the Federal Water Pollution Control Act, which established the construction grants program. Under the 1972 Amendments to the Federal Water Pollution Control Act (Public Law 92-500) and Clean Water Act of 1977 (Public Law 95-217), thousands of municipal wastewater treatment facilities have been constructed or expanded across the nation to control or prevent water pollution.[3,4] The law established the *National Pollutant Discharge Elimination System* (NPDES), which calls for limitation on the amount or quality of effluent and requires all municipal and industrial discharges to obtain permits. The interim goal is to achieve water quality in natural waters, which provides for the protection and propagation of fish, shellfish, and wildlife and provides recreation in and on the water. The law authorized billions of dollars for construction grants. Pursuant to the mandates of 1987 Clean Water Act Amendments, the federal construction grants program was converted to *State Revolving Fund* (SRF), which provided loans to municipalities for construction of wastewater treatment facilities. Also, regulations for stormwater discharges, combined sewer overflows, and disposal of sewage sludge have been established. The 1992 *Needs Survey Report to Congress* indicated that there were a total of 15,613 wastewater treatment facilities serving a total of 181 million people. Out of these, 1,981 facilities have no discharge; 868 facilities provide less than secondary; 9,086 facilities provide secondary; and 3,678 facilities provide higher than secondary level of treatment. It is estimated that by the year 2012, there will be 18,966 facilities serving 251 million people.[5]

Effluent treatment of liquid waste increased the quantity of sludge or residuals for handling and disposal. Residuals management practices trailed the development of liquid processing alternatives. The significant developments in sludge management technology over the past 50 years were heated anaerobic digestion and utilization of digestion gases; gravity, dissolved air flotation, and centrifuge for sludge thickening; sludge drying beds, vacuum filter, belt filter and centrifuge for dewatering; and landfilling, incineration, and composting for sludge disposal.

During the last 20 years, contemporary design has emphasized secondary treatment, primarily to control the carbonaceous and nitrogenous oxygen-demanding pollutants, and suspended solids. The treatment strategies for phosphorus and nitrogen removal were limited to highly-restricted waters. Toxicity control has emerged only recently as a design consideration.

Many innovations in screening, degritting, clarification, aeration and mixing, sludge bulking control, biological nutrient removal (both suspended and attached growth), and sludge stabilization and beneficial recycling of biosolids have taken place in the last decade and provide cost savings and system improvements. All these innovative technologies have been tested under EPA-funded programs.

1-3 FUTURE DIRECTIONS

Many technological advances have been achieved in the wastewater treatment field as a result of concepts and policies promulgated by environmental laws and guidelines. A great deal has been learned regarding process design and construction, operation and maintenance, problems associated with improper site selection, and plant design. In the next decade there will be some shift in strategy for new plants and upgrading of existing facilities. Major changes are expected in process design, wastewater reclamation and reuse, and sludge disposal and reuse. New directions and concerns are evident in several interrelated areas of wastewater treatment fields: (1) health and environmental concerns, (2) improvement in treatment processes, (3) control and treatment of stormwater and combined sewer overflows, (4) effluent disposal and reuse, (5) biosolids disposal and reuse, and (6) on-site treatment and disposal of small flows.

1-3-1 Health and Environmental Concerns

Wastewater treatment works must not be the ugly duckling in the community but, rather, a good neighbor. In the past, odors, dust, noise, erosion, and unsightly conditions created public doubts and uneasiness about the nearby municipal wastewater treatment works. It is unacceptable to create new environmental problems.[6] Therefore, future planning and designs of wastewater treatment facilities will emphasize techniques to minimize adverse environmental impacts and objections by neighborhood residents. Furthermore, active public participation programs will be an integral part of the decision-making process at all stages of the planning, design, construction and operation of wastewater treatment facilities.[7]

Currently, the release of volatile organic compounds (VOCs) and volatile toxic organic compounds (VTOCs) found in wastewater are of great concern in the operation of a collection system; wastewater treatment process selection, design, and operation; and sludge processing and disposal. Also, odors are one of the most serious environmental concerns for the public. Furthermore, in addition to being odorous, the hydrogen sulfide can cause accelerated corrosion of sewers and preliminary and primary treatment facilities. Great emphasis is therefore being placed on plant site selection, process selection, compact layout, covered and multistory process design, and even underground facilities. Such concepts reduce the atmospheric exposure of the wastewater. The air from the treatment facilities is collected and treated prior to discharge into the atmosphere. Concepts like these in plant design address VOCs, odors, and other environmental issues.

1-3-2 Improvements in Treatment Processes

In the past, extensive research activity in the search for improved processes for wastewater treatment and sludge management has resulted in a rapid growth in technology. Enhanced knowledge of fundamentals has permitted the engineers to adapt to innovative alternative technologies for improved process designs. The empirical methods commonly used in the past are inadequate for interpreting data and optimizing the processes. Laboratory and pilot plant studies will be utilized to develop process design parameters and kinetic coefficients for industrial and joint industrial–municipal wastewater treatment facilities. Future process designs may incorporate energy conservation, compact degritter, fine screens to replace primary sedimentation, sludge bulking and foaming control, biological nutrient removal, natural systems, improved methods of sludge thickening and dewatering and biosolids utilization, incineration, and codisposal with municipal solid wastes.

In addition to improved process design, great emphasis will be given to plant operation and maintenance to optimize the treatment costs. Energy saving measures and value engineering reviews are being emphasized to reduce capital and operation costs of the treatment facility.

1-3-3 Control and Treatment of Stormwater and Combined Sewer Overflows

The discharge of stormwater runoff and other nonpoint sources and combined sewer overflows (CSOs) to the receiving waters have resulted in contamination problems that often have prevented the attainment of water quality standards. The contaminants found in stormwater and CSOs include bacteria, nutrients, solids, biochemical oxygen demand (BOD), metals, and other potentially toxic organic constituents. Implementation of pollution control measures for stormwater runoff and CSOs will require management of stormwater drainage area, combined wastewater collection and detention basins, and wastewater treatment plants. In response to the need for solving these problems, engineers have used a variety of stormwater runoff and CSO control methods, including construction of new separate sewers and stormwater drainage systems, treatment at the combined sewer outlet, and storage followed by treatment under dry-weather conditions.

Approaches for reducing pollution from stormwater and CSOs are now in the development and evaluation phases. It is anticipated, however, that in many cases the benefits obtained from construction of treatment works for these purposes will be small compared with the costs. On the other hand, there are techniques for control and prevention that can be more cost-effective. The policy of the EPA is, therefore, not to use construction grants for treatment works to control pollution from stormwater and CSOs, except under unusual conditions, where the project clearly has been demonstrated to meet the planning requirements and criteria developed for CSOs. Chapter 26 is devoted exclusively to these issues.

1-3-4 Effluent Disposal and Reuse

Great emphasis is being given to the opportunities for effluent reuse. In many cases treatment plants have been located so that a portion of effluent is reused for irrigation of golf courses, highway medians, city parks, and landscape watering; effluent reused as industrial cooling water; groundwater recharge; and an indirect water supply through discharge into rivers, lakes, and reservoirs. Great attention is given to the environmental effects of constituents such as nutrients, refractory organics, toxic compounds, and microorganisms and how these constituents are safely assimilated into the aquatic, as well as terrestrial, environment. Mathematical modeling techniques are used to assess the assimilative capacity of these systems and thus to predict the impacts of the proposed reuse or discharge. A higher level of treatment may often be necessary to achieve the desired reuse objectives. Many effluent reuse and disposal options are covered in Chapter 15.

1-3-5 Biosolids Disposal and Reuse

Large quantities of sludge are produced with utilization of advanced wastewater treatment technology. There is an urgent need to find better methods of solids processing, reuse, and disposal. Landfilling and incineration of dewatered sludge is of great concern because of potential groundwater and air pollution problems. New regulations will restrict land application because of lower limits that will be imposed on certain heavy metals. This will mean tighter industrial pretreatment regulations to reduce certain heavy metals in the sludge. As a result of new regulations on biosolids utilization, solids processing has become one of the most significant challenges to the wastewater treatment plant designers and managers. Land application of biosolids is presented in Chapter 19.

1-3-6 On-site Treatment and Disposal of Small Flows

During the 1970s great emphasis was given to areawide wastewater planning and management in an effort to (1) use best practicable waste treatment technology and (2) produce revenues through use of effluent and sludge. Experience has shown that centralized facilities may require pumping of wastewater long distances from different portions of the service area and thus may be in general energy- and resource-intensive. Also serious odors, dust, noise, and other environmental problems developed in the community, as a result of processing large quantities of wastewater and sludge at one location.

In recent years, the concept of satellite wastewater treatment has been reevaluated in the overall context of economics, innovative energy-efficient technology, and reuse of effluent and sludge locally, or sludge alone being pumped to a central location for processing. Also, individual on-site treatment and disposal systems have received greater attention. This topic is covered in Chapter 25.

1-4 PLANT DESIGN

The task of planning and designing wastewater treatment works is not simple. It involves understanding of service area, sources of wastewater and the resulting characteristics, plant site, conveyance system, and treatment processes for the liquid and residues. Many nontechnical factors such as legal issues, regulatory constraints, public participation, effluent, and sludge disposal and reuse may influence planning and design. Furthermore, most of the facilities are designed to provide service over the plant's life expectancy (20 years or more). During this extended time span, technology may improve, new laws may be passed, new regulations may be issued, and economic factors may change. The engineers must consider these possibilities and should favor processes that are sufficiently flexible to remain useful in the face of changing technology, regulations, economics, and wastewater characteristics.

During the design phase of a plant, it is important to recognize that the overall performance of a wastewater treatment facility is the result of combined performances of many components utilized in the overall process train. The designers must understand the design implications and performance of the individual processes and how these processes may affect one another under normal and adverse operational conditions. As an example, failure to remove solids produced within the treatment processes will eventually cause degradation of effluent quality. Likewise, hydraulic overload to the sludge-processing systems may increase the solids in the sidestream, which is returned to the plant as a recirculating load, thus adversely affecting the influent quality to be treated.

In spite of the multitude of regulations and standards that treatment plants must comply with, the theory and design principles of wastewater treatment processes such as screening, sedimentation, biological waste treatment, nutrient removal, filtration, demineralization, and sludge processing systems have not changed over half a century. What has changed, however, are many tools that both designers and operators have at their disposal. New equipment has improved efficiency and reliability. Computers have bestowed the gifts of alacrity and accuracy in design and operation. Now, the engineers can compare the alternative processes and process trains with a speed that was not possible with pencil and graph paper. Likewise, a supervisory control and data acquisition (SCADA) system, and expert system, can provide operators and managers with accurate process control variables and operation and maintenance records. In addition to being able to look at the various options on the computer screen, engineers can conduct pilot plant studies to develop the multiple variables that are inherent to wastewater treatment plant design. Likewise, operators and managers can utilize an ongoing pilot plant facility to optimize variables and develop important information needed for future expansion and upgrading.

Wastewater treatment plants should be designed so that the effluent standards and reuse objectives and biosolids regulations can be met with reasonable ease and cost. The design should incorporate flexibility for dealing with seasonal changes, as well as long-term changes in wastewater quality and future regulations. Good planning and design, therefore, must be based on five major steps: (1) characterization of the raw wastewater quality and effluent, (2) predesign studies to develop alternative processes and selection of the final process train, (3) detailed design of the selected alternative, (4) construction, and (5) operation and maintenance of the completed facility.

Engineers, scientists, and financial analysts must utilize principles from a wide range of disciplines, such as engineering, chemistry, microbiology, geology, architecture, and economies, to carry out the responsibilities of designing a wastewater treatment plant.

1-5 SCOPE OF THE BOOK

The objective of this book is to present the technical and nontechnical issues that are most commonly addressed in the planning and design reports for wastewater treatment facilities prepared by practicing engineers. Topics discussed include facility planning, process description, process selection logic, mass balance calculations, design calculations, and concepts for equipment sizing. Theory, design, operation and maintenance, troubleshooting, equipment selection, and specifications are integrated for each treatment process. Thus, delineation of such information for use by students and practicing engineers is the main purpose of this book.

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2

Basic Design Considerations

2-1 INTRODUCTION

The planning, design, construction, and operation of wastewater treatment facilities is a complex problem. It involves political, social, and technical issues. Therefore, besides meeting the effluent quality requirements, a wastewater treatment and disposal system must also prevent many adverse environmental conditions. Some of these environmental conditions are (1) unsightliness, nuisance, and obnoxious odors at treatment and disposal sites; (2) contamination of water supplies from physical, chemical, and biological agents; (3) destruction of fish, shellfish, and other aquatic life; (4) degradation of water quality of receiving waters from overfertilization; (5) impairment of beneficial uses of natural waters (recreation, agriculture, commerce, or industry, etc.); (6) spread of diseases from crops grown on sewage irrigation or biosolids utilization; and (7) decline in land values and, therefore, restriction in community growth and development. Ideally, a wastewater treatment plant must encourage the beneficial uses of effluent and residuals.

The purpose of this chapter is to discuss many important design factors that must be considered during the initial planning and design stages of a wastewater treatment project. Basic design factors are

1. Initial and design years
2. Service area
3. Site selection
4. Design population
5. Regulatory control and effluent limitations
6. Characteristics of wastewater
7. Degree of treatment
8. Selection of treatment processes
9. Equipment selection
10. Plant layout and hydraulic profile
11. Energy and resource requirements

12. Plant economics
13. Environmental impact assessment

Although most of these design factors are covered in greater detail in subsequent chapters, the information presented below is an introduction to the plant design.

2-2 INITIAL AND DESIGN YEARS

It generally takes several years to plan, design, and construct a wastewater treatment facility. Accordingly, most of the plant's components are made large enough to satisfy the community needs for several years in the future. The initial year is the year when the construction is completed and the initial operation begins. The design or planning year is the year when the facility is expected to reach its full designed capacity. Selecting the design year is not a simple task. It requires sound judgments and skills in developing future growth estimates from the past social and economic trends of a community. Design periods[a] are generally chosen with the following factors in mind:[1,2]

1. Useful life of treatment units, taking into account wear-and-tear and process obsolescence
2. Ease or difficulty in expansion
3. Performance of the treatment facility during the initial years when it is oversized
4. Future growth in population (including shifts in community), service area, commercial and industrial developments, water demands, and wastewater characteristics
5. Interest rates, cost of present and future construction, and availability of funds

The design periods of different components of a treatment facility may also vary. As an example, main conduits, channels, and appurtenances that cannot be expanded readily are designed for periods up to 50 years in the future. On the other hand, treatment units, process equipment, pumps, and sludge-handling and disposal facilities are constructed for shorter periods to avoid construction of oversized units. In such cases, adequate space is left at the plant site for expansion of the facility at different staging periods.[b] According to the guidelines of the construction grants program, the design period may be divided into staging periods (10, 15, and 20 years), depending on the ratio of wastewater flow expected at the design and initial years.[3] These staging periods are summarized in Table 2-1. Additional discussion on staging period and procedure to calculate staging period are provided in Chapter 6. Readers may review Sec. 6-6-4 for development of staging period of 15 years calculated for the Design Example.

2-3 SERVICE AREA

Service area (also called sewer district) is defined as the total land area that will be eventually served by the proposed wastewater treatment facility. The area may be based on

[a]Design or planning period is defined as the period from the initial year to the design year.

[b]Staging periods are time intervals when plant expansions are made.

TABLE 2-1 Staging Periods for Plant Expansion over the Design Period

Flow Growth Factor (Ratio of Flows at Design and Initial Years)	Staging Period (Years)
Less than 1.3	20
1.3–1.8	15
Greater than 1.8	10

natural drainage, political boundaries, or both. Areas outside the city limit but draining toward the proposed wastewater treatment plant and the area that may become incorporated into the city at future dates should also be considered into the service area. On the other hand, areas requiring sewage pumping may also be included into the service area after careful evaluation of economics and other environmental constraints.

It is important that the design engineer and the project team become familiar with the service area of the proposed project. Site visits and review of engineering data on topography, geology, hydrology, climate, ecological elements, and social and economic conditions may be necessary. Existing zoning regulations and land use or zoning plans and probable future changes that may affect both developed and undeveloped lands should be studied. Such efforts should be carefully coordinated with the state, regional, and local planning agencies and should be in conformance with the development and implementation of area-wide waste management plans.[3]

2-4 SITE SELECTION

Site selection of a wastewater treatment facility should be based on careful consideration of regions' land use and development patterns as well as social, environmental, and engineering constraints. It is important to remember that the selection of a site for a wastewater treatment plant will have long-lasting social, economic, and political repercussions on the affected community and neighborhood. Therefore, public involvement in decision making is crucial.

All possible sites for a wastewater treatment plant should be fully evaluated on the basis of topography, environmental impacts, and economics of wastewater collection and treatment. Often an interdisciplinary[c] team approach may be necessary to cover all these aspects. The following is a list of some basic principles that must be considered during site evaluation:

1. A wastewater treatment plant should be located at a low elevation in order to permit gravity flow.
2. The site should be fairly isolated from presently built-up areas or areas that have potential for future developments. All plants should be designed with aesthetic consid-

[c]The interdisciplinary team may include professionals from sanitary and environmental engineering, urban planning, architecture, biology, government, and civic groups.

erations and odors in mind. For instance, the drying beds for sludge require large areas and are potential sources of bad odors.

3. A site on a large land area is helpful in maintaining isolation (buffer areas) and fulfills the needs for future expansion.
4. A site that may provide opportunity for local disposal of end products such as effluent, grit, screenings, sludge, and ash is highly desirable.
5. A site within a flood zone should not be selected unless proper flood protection measures are taken. Such measures include raising the units above flood level or constructing levees around the site. Storm drainage system with pumping equipment to discharge the storm water and effluent above the flood level should be provided. Auxiliary power equipment may be necessary in the event of power outages.
6. The site should have year-round, all-weather access roads. Railroads may be helpful for delivery of bulk chemicals and transport of sludge from large facilities.
7. The site should be near a large body of water or irrigable land capable of accepting the treated effluent.
8. The ability of the ground to support structures without extensive piling is important consideration in site selection. Common foundation problems are low bearing capacity, excessive settlement, differential settlement, and flotation caused by high groundwater table.
9. A site with a moderate slope will assist in locating various treatment units in their normal sequence without excavation or filling. This will provide gravity flow, least disruption to the natural topography, and least erosion control measures.
10. The sites should be evaluated and checked for presence of archaeological, historical, or other properties included in or eligible for inclusion in the *National Register of Historic Places.* The site should also be investigated for the presence of endangered or threatened species of flora or fauna or their critical habitats.
11. Site selection and planning of wastewater treatment facilities should be done with prime consideration of preservation of shorelines, particularly in urban areas. Shoreline preservation involves public pathways along shore, public parks, recreation facilities, and protection against erosion of banks, siltation of the waterway, and preservation of valuable ecological niches.

Additional information on site selection and planning for wastewater treatment plants may be found in Chapter 20. In summary, proper site selection for proposed wastewater treatment facility is important and must not be overlooked by the designers. A team approach to site selection will be helpful in future expansion cost and energy savings, and avoid future complications that may occur due to public opposition. Matters to be investigated include topography, drainage, surface and groundwater, soil type, prevailing winds, temperature, precipitation, seasonal solar angles, wildlife habitats, ecosystems, regional and local land use and zoning, transportation, archaeological and historical features, and other factors.

There is a large amount of existing information available on the above subjects at various local, state, and federal agencies, and at the universities. There are also many types of methodologies available for alternative evaluation and selection of sites. These methodologies offer analytical tools for identifying, measuring, and interpreting the data. Data sources and evaluation methodologies may be found in Refs. 4–7.

2-5 DESIGN POPULATION

The volume of wastewater generated in a community depends on the population and per capita contribution of wastewater. It is therefore important to estimate the population to be served at the design year. Accurate population prediction is quite difficult because many factors influence the growth of the city. Among important factors are industrial growth; state of development of the surrounding area; location with regard to transportation sources; availability of raw material, land, and water resources; local taxes and government activities; migration trends; and so on.

The population data can be obtained from several sources. The U.S. Bureau of Census (Department of Commerce) publishes 10-year census data. For the interim periods, reliable data can usually be obtained from local census bureaus; the state, county, or local planning commissions; the chambers of commerce; voters registration lists; the post office; newspapers; and public utilities (telephone, electric, gas and water, etc.). It is important that the design engineer become familiar with the population data sources and the type of information that can be obtained from these sources.

2-5-1 Methods of Population Forecasting

There are many mathematical and graphical methods that are used to project past population data to the design year. Widely employed methods are

1. Arithmetic growth
2. Geometric growth
3. Decreasing rate of increase
4. Mathematical or logistic curve fitting
5. Graphical comparison with similar cities
6. Ratio method
7. Employment forecast
8. Birth cohort

All these methods utilize different assumptions and therefore give different results. Selection of any method depends on the amount and type of data available and whether the projections are made for the short or long term. The arithmetic, geometric, decreasing rate of increase, and logistic curve-fitting methods are summarized in Table 2-2. The remaining methods are presented in Table 2-3 and Figures 2-1 to 2-3. Several excellent references may be consulted on population forecasting.[8-11]

2-5-2 Population Density

The average population density for the entire city rarely exceeds 7500–10,000 per km^2 (30–40 per acre). Often it is important to know the population density in different parts

TABLE 2-2 Population Projections by Using Arithmetic, Geometric, Decreasing Rate of Increase, and Mathematical Curve Fitting

Method	Description	Basic Equations or Procedure[a]	Calculated Values[b]
Arithmetic method	Population is assumed to increase at a constant rate. The method is used for short-term estimates (1–5 yr).	$\frac{dY}{dt} = K_a;\ Y_t = Y_2 + K_a(T - T_2)$ $K_a = \frac{Y_2 - Y_1}{T_2 - T_1}$	$K_a = 300/\text{yr}$ $Y_t = 21{,}000$
Geometric method	Population is assumed to increase in proportion to the number present. The method is commonly used for short-term estimates (1–5 yr).	$\frac{dY}{dt} = K_pY;\ \ln Y_t = \ln Y_2 + K_p(T - T_2)$ $K_p = \frac{\ln Y_2 - \ln Y_1}{T_2 - T_1}$	$K_p = 0.0182$ $Y_t = 21{,}600$
Decreasing rate of increase	Population is assumed to reach some limiting value or saturation point.	$\frac{dY}{dt} = K_d(Z - Y),\ Y_t = Y_2 + (Z - Y_2)(1 - e^{-K_d(T - T_2)})$ $Z = \frac{2Y_0Y_1Y_2 - Y_1^2(Y_0 + Y_2)}{Y_0Y_2 - Y_1^2}$ $K_d = \frac{-\ln[(Z - Y_2)/(Z - Y_1)]}{T_2 - T_1}$	$Z = 20{,}000$ $K_d = 0.09163$ $Y_t = 19{,}200$
Mathematical or logistic curve fitting	It is assumed that the population growth follows a logistical mathematical relationship. Most common relationship is an S-shaped curve.	$Y_t = \frac{Z}{1 + ae^{b(T - T_0)}}$ $a = \frac{Z - Y_0}{Y_0}$ $b = \frac{1}{n}\ln\left[\frac{Y_0(Z - Y_1)}{Y_1(Z - Y_0)}\right]$	$Z = 20{,}000$ $a = 1.00$ $b = -0.1099$ $n = 10$ $Y_t = 19{,}287$

[a] dY/dt = rate of change of population with time. Y_0, Y_1, and Y_2 = populations at time T_0, T_1, and T_2. Y_t = estimated population at the year of interest. Z = saturation population. K_a, K_p, and K_d are proportionality constants. a and b = constant. n = constant interval between T_0, T_1, and T_2 (generally 10 yr).
[b] Population for the year 2000 is estimated using the following census results: $T_0 = 1970$, $Y_0 = 10{,}000$; $T_1 = 1980$, $Y_1 = 15{,}000$; $T_2 = 1990$, $Y_2 = 18{,}000$. $T = 2000$.

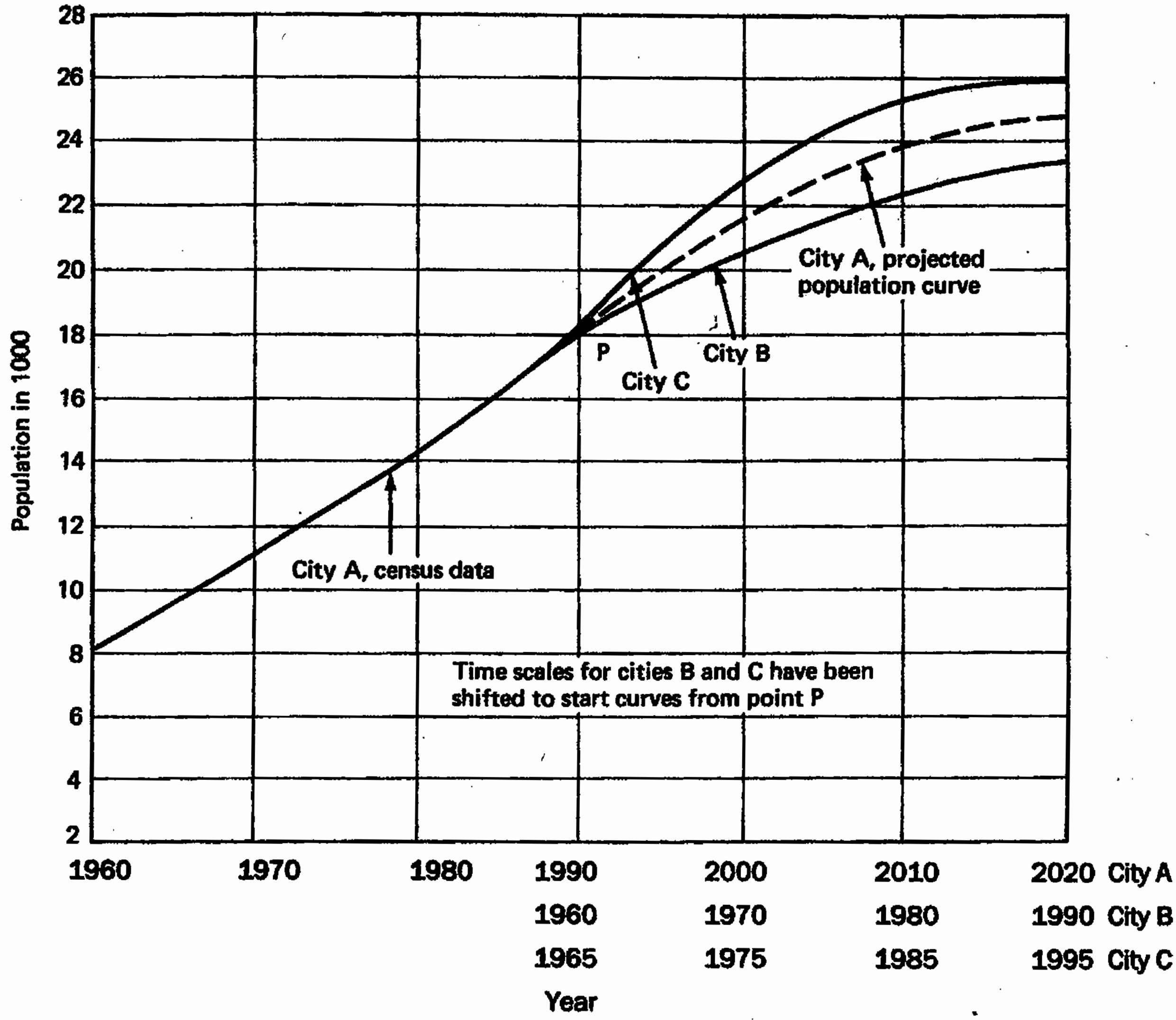


Figure 2-1 Population Estimate by Graphical Comparison.

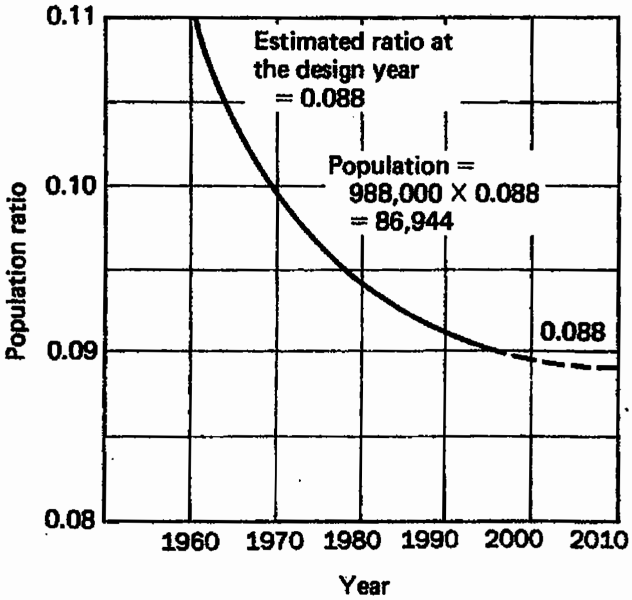


Figure 2-2 Population Estimate by Ratio Method.

TABLE 2-3 *Population Projections by Using Graphical Comparison, Ratio Method, Employment Forecast, and Birth Cohort*

<table>
<tr><th>Method</th><th>Description</th><th>Problem Definition</th><th>Estimated Population</th></tr>
<tr><td>Graphical comparison</td><td>The procedure involves the graphical projection of the past population data for the city being studied. The population data of other similar but larger cities are also plotted in such a manner that all the curves are coincident at the present population of the city being studied. These curves are used as guides in future projections.</td><td>Estimate the population of City A by using graphical comparison with cities B and C. The design year is 2020.
<table>
<tr><th></th><th colspan="3">Population in 1000s</th></tr>
<tr><th>Year</th><th>City A</th><th>City B</th><th>City C</th></tr>
<tr><td>1960</td><td>8.0</td><td>18.0</td><td>16.0</td></tr>
<tr><td>1970</td><td>11.0</td><td>20.3</td><td>20.0</td></tr>
<tr><td>1980</td><td>14.2</td><td>22.3</td><td>25.0</td></tr>
<tr><td>1990</td><td>18.0</td><td>23.2</td><td>25.6</td></tr>
</table>
The present population of the city A is 18,000. The procedure is illustrated in Figure 2-1.</td><td>$Y_{2020} = 24{,}800$</td></tr>
<tr><td>Ratio and correlation</td><td>In this method the population of the city in question is assumed to follow the same trends as that of the region, county, or state. From the population records of a series of census years, the ratio is plotted and then projected to the year of interest. From the estimated population of the region, county, or state, and the projected ratio, the population of the concerned city is obtained.</td><td>Estimate the population of a city using the ratio method. The design year is 2010. The estimated population of the region in the year 2010 is 988,000.
<table>
<tr><th></th><th colspan="3">Population in 1000s</th></tr>
<tr><th>Year</th><th>City</th><th>Region</th><th>Ratio</th></tr>
<tr><td>1960</td><td>50</td><td>455</td><td>0.110</td></tr>
<tr><td>1970</td><td>61</td><td>623</td><td>0.098</td></tr>
<tr><td>1980</td><td>72</td><td>766</td><td>0.094</td></tr>
<tr><td>1990</td><td>77</td><td>850</td><td>0.091</td></tr>
</table>
</td><td>$Y_{2010} = 86{,}944$</td></tr>
</table>

Employment forecast or other utility connections forecast

The population is estimated using the employment forecast. From the past data of population and employment, the ratio is plotted and population is obtained from the projected employment forecast. Procedure is similar to that of the ratio method. Similar procedure can be utilized from the forecast of various utility service connections such as telephone, electric, gas, water and sewers, etc. Utility companies conduct studies and develop reliable forecasts on the future connections. Forecasts of postal and newspaper service points have also been used in population estimates.

Estimate the population of a city using employment forecast. The design year is 2010. Use the following data. Employment forecast for the year 2010 is 21,300.

Year	Population in 1000s	Employment in 1000s	Ratio Population and Employment
1960	20	6.80	2.94
1970	30	10.79	2.78
1980	39	14.77	2.64
1990	46	17.83	2.58

The procedure is illustrated in Figure 2-3.

$Y_{2010} = 54{,}102$

Birth cohort

A birth cohort is defined by demographers as a group of people born in a given year or period.[a] The existing populations of males and females in different age groups are determined from the past records. From birth and death rates of each group and population migration data, the net increase in each group is calculated. The population data are then shifted from one group to the other until the design period is reached. This procedure is discussed in Refs. 8, 10 and 11.

[a]*Demography* is that branch of anthropology that deals with the statistical study of the characteristics of human population with reference to total size, density, number of deaths, births, migration, etc.

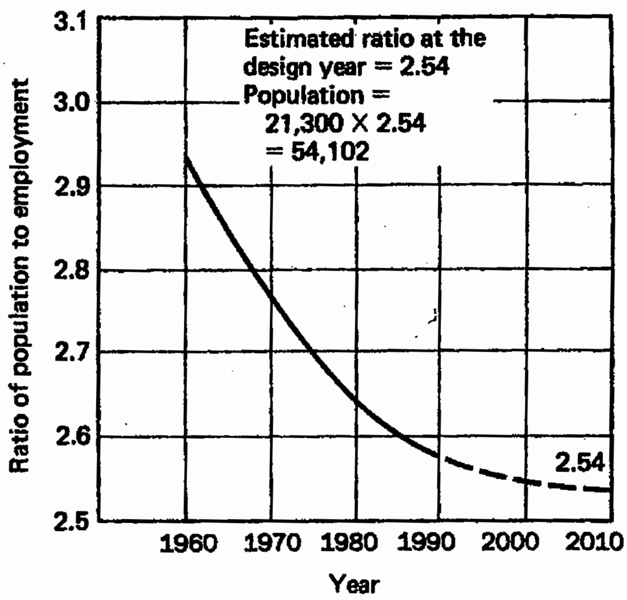


Figure 2-3 Population Estimate by Employment Forecast.

of the city in order to estimate the flows and to design the collection network. Density varies widely within a city depending on the land use. The average population densities based on land use characteristics are summarized in Table 2-4.

2-6 REGULATORY CONTROLS AND EFFLUENT LIMITATIONS

2-6-1 Water Pollution Control Legislation

Virtually all of the major water pollution control legislation in the United States has been passed and written in the past 35 years. However, much progress has been made during this period. Following is a summary of water pollution control legislation in the United States:

- Federal Water Pollution Control Act of 1948
- Federal Water Pollution Control Act of 1956
- Federal Water Pollution Control Act Amendments of 1961
- Water Quality Act of 1965
- Clean Water Restoration Act of 1966
- Water Quality Improvement Act of 1970
- Federal Water Pollution Control Act Amendments of 1972
- Clean Water Act of 1977
- Clean Water Act Amendments of 1981
- Clean Water Act Amendments of 1987

The 1972 and 1977 laws are collectively known as the Clean Water Acts. The Clean Water Act requires the U.S. Environmental Protection Agency to report to Congress *(Needs Survey)* about the progress and status of the program and a detailed capital cost invest-

TABLE 2-4 Range of Population Densities in Various Sections of a City

	Population Range	
Land Use	**Persons per km²**	**Persons per Acre**
Residential areas		
Single-family dwellings, large lots	1250–3700	5–15
Single-family dwellings, small lots	3700–8700	15–35
Multiple-family dwellings, small lots	8700–25,000	35–100
Apartment or tenement houses	25,000–250,000	100–1000 or more
Commercial areas	3700–7500	15–30
Industrial areas	1250–3700	5–15
Total, exclusive of parks, playgrounds, and cemeteries	2500–12,500	10–50

Note: $km^2 \times 247.1 =$ acres.
Source: Adapted from *Water and Wastewater Engineering, Vol. 1. Water Supply and Wastewater Removal* by G. M. Fair, J. C. Geyer, and D. A. Okun, 1966. Used with permission of John Wiley & Sons.

ment additionally needed to comply with the requirements of the Act. The ultimate goal of these Acts is to eliminate the discharge of pollutants into all surface waters.

The 1992 Needs Survey indicated that considerable progress in municipal pollution abatement from publicly owned treatment works, both nationwide and in urban areas, has been made. Existing secondary treatment capacities have been expanded, and less efficient facilities have been upgraded. This progress has been both significant and measurable.[12]

2-6-2 Effluent Standards

Under the Clean Water Acts each state and the area agencies were required to develop a *water quality management* (WQM) plan that identified sources and severity of pollution, and needed programs to control pollution. Once completed and approved, the plan became the foundation of the WQM process. Each state annually assesses current water quality problems, updates its strategy to solve problems, prepares and carries out a work program to implement solutions, evaluates performance, and revises the plan. As part of the WQM process, the state agency establishes total maximum daily wasteloads for all surface waters throughout the state. The agency classifies bodies of state waters as either *effluent-limited* or *water quality-limited.* If state's water quality standards can be met by uniform national discharge limits, the water body is classified as effluent-limited, and all municipal treatment plants need only achieve secondary treatment. (The secondary level of treatment as defined by EPA is summarized in Table 2-5.[13]) Where stricter limits are needed to meet the state's standards, the water body is classified as water quality-limited. The resulting wasteload allocations are generally incorporated into the effluent limita-

TABLE 2-5 Minimum Level of Effluent Quality Attainable by Secondary Treatment as Defined by EPA

	30-Day Average		
Effluent Parameter	**Maximum Concentration, mg/L**	**Minimum Removal, %**	**7-Day Average, Maximum Concentration, mg/L**
BOD_5	30	85	45
$CBOD_5$[a]	25	85	40
TSS[b]	30	85	45
PH[c]		Between 6.0 and 9.0	

[a]CBOD = carbonaceous biochemical oxygen demand.
[b]TSS = total suspended solids.
[c]The pH limit may be exceeded if POTW demonstrates that (1) inorganic chemicals are not added to waste stream as part of treatment process, and (2) contributions from industrial sources do not cause the pH of the effluent to be less than 6.0 and greater than 9.0.
Note: Special considerations may apply to combined sewers, certain industrial categories, waste stabilization ponds, and less concentrated influent wastewater for separate sewers.

tions, and in the compliance schedule of the National Pollutant Discharge Elimination System (NPDES) permit.

Pursuant to mandates of the 1987 Clean Water Act Amendments, regulations for stormwater discharges and disposal of sewage sludge have been established. To manage the stormwater discharges under the NPDES permitting process, regulations were established in 1990 for municipalities and industrial activities related to manufacturing processes and raw material storage areas.[14] Regulations for disposal of sewage sludge by land application, surface disposal, and incineration became effective in 1993. These sludge regulations are self-implementing and apply to all facilities disposing municipal sludge by the above three methods, independent of NPDES permitting.[15]

2-6-3 Enforcement

The Clean Water Act requirements are backed up by a permit program which describe the effluent limitation requirement of the point source discharge and other conditions to be imposed on individual discharges. Among these conditions are monitoring and schedules for compliance. These permits are part of the National Pollutant Discharge Elimination System (NPDES) and are issued and enforced either by an EPA regional office or by a state water quality agency (most of the states have been approved by EPA to operate the NPDES permit program).

Under the NPDES permitting program, effluent limitations are established for conventional, nonconventional, toxic, and hazardous pollutants based upon their levels of concentration in the effluent. In a similar manner, industrial pretreatment standards[d] are being developed for all pollutants that "interfere with, pass through, concentrate in

[d]Pretreatment standards apply to the industrial discharges into the POTWs.

sludge, or otherwise are incompatible" with the publicly owned treatment works (POTWs). Under the pretreatment regulations, two types of federal pretreatment standards are established: (1) prohibited discharges and (2) categorical standards.[16,17] The prohibited discharges are those that cause fire or explosion hazard, corrosion, obstruction, slug discharges, and heat discharge to sewers or POTWs. The categorical standards are developed for those pollutants that are incompatible, that is, those that interfere with the operation of, pass through, or contaminate the sludge and other residues from POTWs.

The substances considered for categorical standards are those for which there is substantial evidence of carcinogenicity, mutagenicity, and/or teratogenicity; substances structurally similar to aforementioned compounds; and substances known to have toxic effects on human beings or aquatic organisms at sufficiently high concentrations and that are present in the industrial effluents. There are many specific elements or compounds that have been identified as priority pollutants. These include heavy metals, organics, cyanides, phenols, and asbestos.[17]

On March 9, 1984, the U.S. EPA published a policy of biomonitoring to determine the toxicity in the wastewater effluent to control the discharge of toxic pollutants in toxic amounts.[18] Live organisms are used. If a toxic level is found and verified, toxicity reduction evaluation (TRE) is required for corrective action. This topic is discussed in more detail in Sec. 3-4-3.

2-6-4 Federal Assistance to Communities

To provide incentives, the Clean Water Act offers federal funds to cover part of the cost of construction of publicly owned wastewater treatment works. In the past, in order to receive construction grants, communities had to meet a series of conditions. Grants were awarded in three steps:[19] (a) Step 1: the facility planning phase, when mostly major decisions leading toward construction of POTWs were made; (b) Step 2: the design plans and specifications for the facility were completed; and (c) Step 3: actual construction work was performed. Under the Act, grantees were required to provide a minimum of secondary treatment to be eligible for a federal grant. New concepts were introduced as part of the planning process. Some of these concepts included facility plan, infiltration/inflow (I/I) analysis, environmental impact assessment, user charge (UC) system, industrial cost recovery, cost-effectiveness, best practicable waste treatment technology (BPWTT), public involvement, etc. The Act also authorized $18 billion over a 5-year period to support the construction grants program and to provide for a continuity of funding. The federal share was 75 percent.

The Clean Water Act of 1977 authorized $24.5 billion over a 5-year period in support of the construction grants program. Several significant changes were introduced, including evaluation of innovative and alternative (I/A) technologies, into the planning process.[20] Technologies that qualified under the program included treatment processes or components that were not fully proven and could be used to achieve reuse and recycle of wastewater and sludge; to reduce cost and energy compared to conventional treatment methods; or to provide simple and economical treatment for small communities. For approved I/A projects, the federal grant share was increased to 85 percent.

The major changes in the construction grants program that occurred in 1981 were the elimination of grants for Step 1 (planning) and Step 2 (design) of treatment plants and a reduction in the percentage of funding from 75 to 55 percent beginning October 1, 1984.[21] In February 1987, a significant provision of the Amendments provided for the gradual changing of the method of assistance from the federal government to the state as they will be the eventual sole manager of the operations of the Construction Grants Program. Under Title VI of the 1987 Amendments, state allotments of federally appropriated funds could be used for establishing a *State Revolving Fund* (SRF) in each state. This fund would make loans to municipalities for construction of wastewater treatment facilities.[22] Repayments of principal and interest would return to the SRF for use in making other loans. The Act requires the state to deposit state monies in the SRF in an amount equaling at least 20 percent of each federal grant payment.

The SRF provides loans at interest rates lower than the rate available elsewhere to any political subdivision with the authority to own and operate a sewer system. Only POTWs are eligible to receive assistance from the SRF. Loans can be used for planning, design, and construction of wastewater treatment facilities, wastewater recycling and reuse facilities, collection systems, and non-point-source pollution control projects, including stormwater management.[22]

2-6-5 Public Participation

The planning procedure for the construction grants program includes the broad-based requirements for public participation specified for all Clean Water Act Programs. Public participation is encouraged in the development, revision, effluent limitation, alternatives selection, and other decision-making processes. Public participation is discussed in detail in Chapters 5 and 6.

2-7 CHARACTERISTICS OF WASTEWATER

The characteristics of wastewater are developed in terms of flow conditions and chemical quality. The characteristics depend largely on the water usage in the community and the industrial and commercial contributions. During wet weather, a significant quantity of infiltration/inflow may also enter the collection system. This will change the characteristics of wastewater. The quantity of infiltration/inflow depends on the condition of sewer system (age and cracks in the pipes, and defective pipe joints and manholes), illegal roof or drain connections, groundwater table relative to sewer position, and the like.

If there is an existing wastewater treatment plant, the characteristics are obtained from the flow records and laboratory data. In the absence of the existing facilities, the data on wastewater characteristics are developed from the population estimates, water usage, and industrial waste discharges.

Wastewater characteristic data are needed for the initial year and for the design year. The data include minimum, average, and maximum dry weather flows; peak wet weather flows; sustained maximum flows; and chemical parameters such as BOD_5, total suspended solids, pH, total dissolved solids, ammonia and total nitrogen, phosphorus, and toxic chemicals.

It is important that reliable estimates on wastewater characteristics be made, because this is what the designed facilities will be treating. Chapter 3 is devoted exclusively to development of wastewater characteristics. Methods of estimating dry weather flows, infiltration and inflow allowances, and various wastewater quality parameters are fully covered in Chapter 3. Detailed procedures for determining infiltration/inflow and wastewater characteristics are presented in Chapter 6.

2-8 DEGREE OF TREATMENT

The degree of treatment required is based on the influent characteristics of the plant and the effluent quality. If the effluent is discharged into the natural water, it should comply with NPDES permit requirements. If used for land irrigation, the plant effluent must also satisfy the health regulations governing the types of crops that are irrigated. Other effluent uses, such as recreational lakes, agriculture, industrial, and municipal, may dictate the effluent quality and thus the degree of treatment.

2-9 CHOICE OF TREATMENT PROCESSES, PROCESS TRAIN, AND COMPARISON OF ALTERNATIVES

Wastewater treatment plants utilize a number of treatment processes to achieve the desired degree of treatment. In addition to this, the design engineer must evaluate numerous other important factors in selection of the treatment processes. These factors include constituents treated, effluent limitations, proximity to buildup areas, hydraulic requirements, sludge disposal, energy requirements, and plant economics. The collective arrangement of various treatment processes is called a flow scheme, a flow sheet, a process diagram, or a process train.

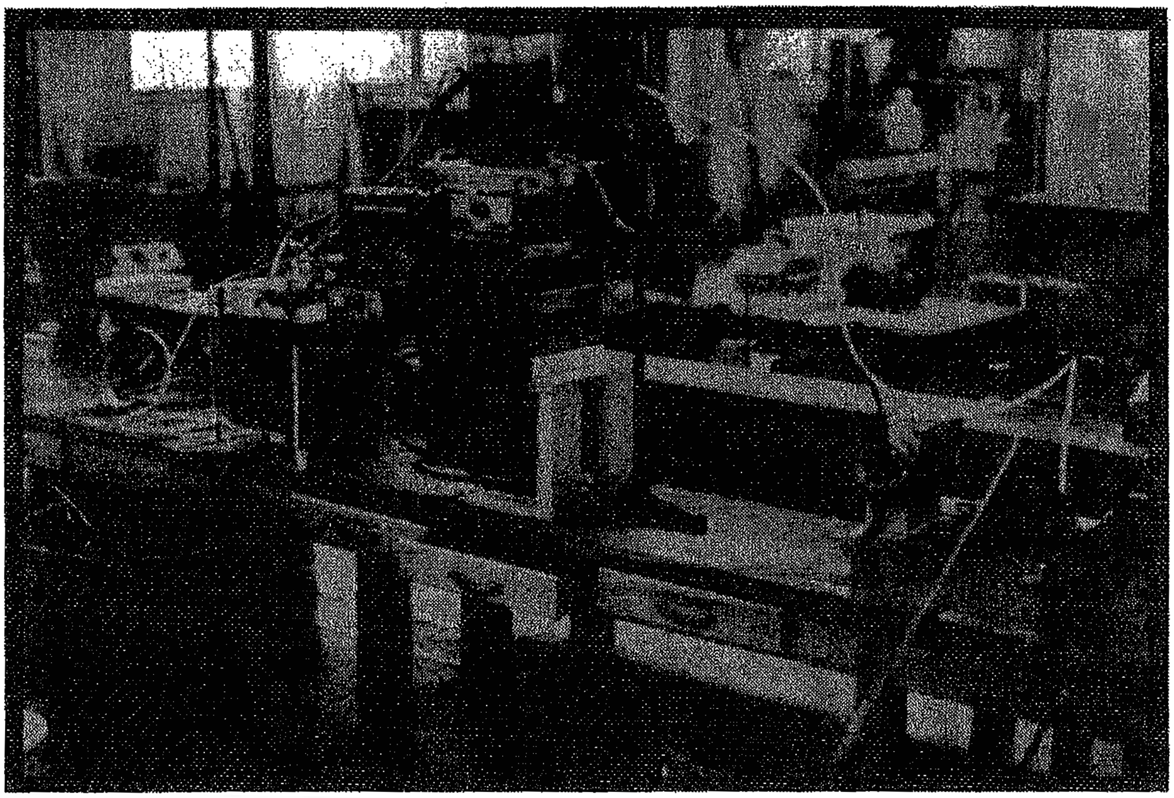
Choice of proper treatment processes and development of the flow scheme is not a simple task. It requires understanding of the unit operations and processes, operational capabilities, and environmental effects of various treatment components that are arranged to develop the process train for a desired application.

Laboratory and pilot plant studies are often necessary to develop design parameters for physical, chemical, and biological treatment processes used to treat industrial wastewater. The laboratory studies include batch and/or continuous flow reactor studies. Procedures to conduct treatability studies on the industrial wastes are given in Refs. 23–26. Figure 2-4 shows the photographs of continuous flow, bench-scale and pilot plant facilities used for wastewater treatment studies.

2-10 EQUIPMENT SELECTION

Every wastewater treatment facility will involve manufactured equipment or materials. In fact, many design details are often governed by the dimensions and installation requirements of the selected equipment. It is the responsibility of the design engineer to select the treatment processes and the corresponding types of equipment for achieving the desired results. To do this, a review of the design standards, design procedure, and design assumptions; preliminary design calculations; and careful study of the manufac-

(a)



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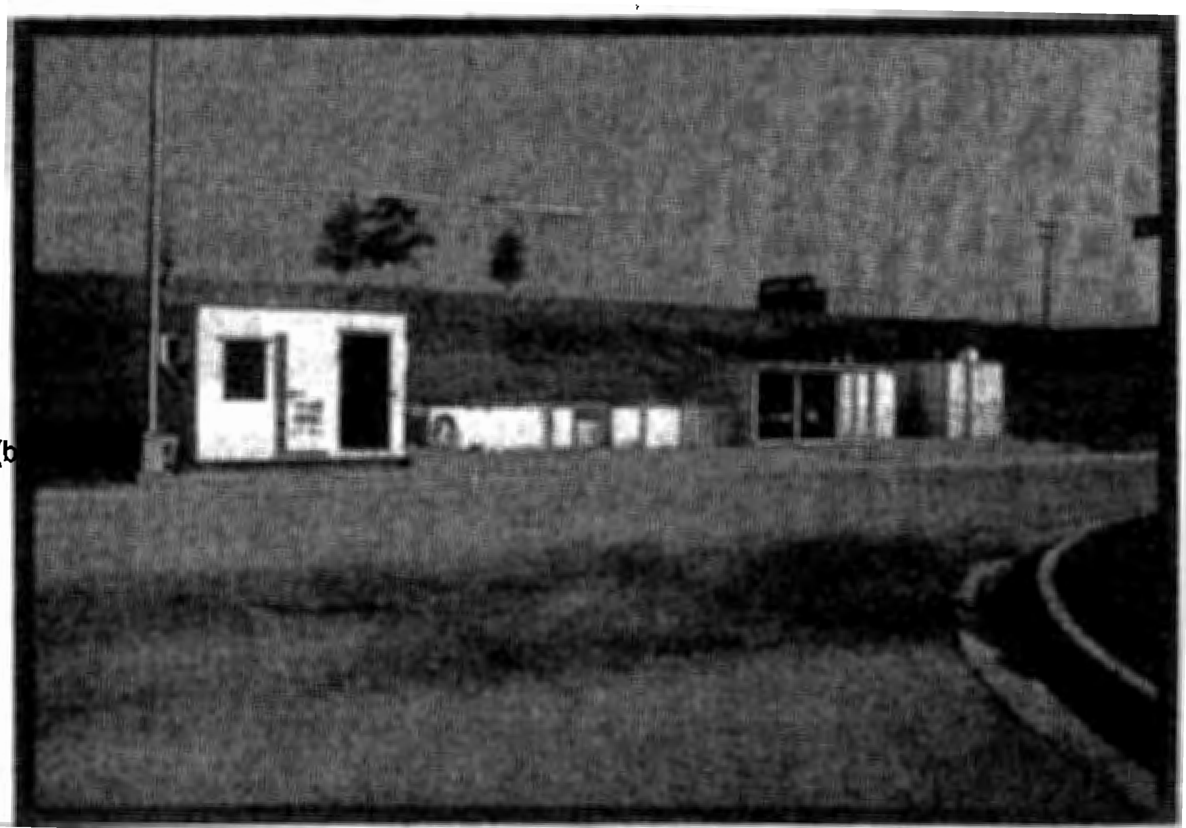

Figure 2-4 Photographs of Laboratory and Pilot Plant Facilities Used to Develop Design Parameter for Biological Wastewater Treatment System: (a) continuous flow, bench-scale laboratory units, (b) pilot plant facility for field testing.

turers' catalogs may be necessary in advance. Then the manufacturers of the appropriate equipment and their local representatives should be asked to furnish the needed data. It may often be necessary for the engineer to work closely with the equipment supplier and provide as much data and general information as possible to ensure that the equipment selection is best for the specific application.

The request for information on equipment should not impose any obligation on the engineer to use the equipment or even to include it in the specifications.[27] However, the equipment data should not be requested unless there is some specific application in the design. It may often be necessary to obtain equipment data from more than one supplier. In no case, however, should the request be sent as a routine to all equipment manufacturers. Preparing the needed information on equipment costs money, and supplying such information without obligation can be justified only if there is a prospect for sale.

2-11 PLANT LAYOUT AND HYDRAULIC PROFILE

During early planning and design stages, careful consideration must be given to the existing conditions at the selected site of the proposed wastewater treatment plant. Condition such as topography, available land area, proximity to the developed areas, access roads, flood conditions, need for future expansion, available head, and so on should all be considered in unit selection and layout. Chapters 20 and 21 are devoted exclusively to the topics of plant layout and hydraulic profile. Design engineers should study these chapters in order to include the basic considerations relative to plant layout and hydraulic profile during the preliminary design and process selection.

2-12 ENERGY AND RESOURCE REQUIREMENTS

Because of the recently increased concern about the limited resources available to meet our energy needs, the project planning and design must also include energy conservation. Primary energy is the energy used in the operation of the facility, while secondary energy is needed to manufacture chemicals, other consumable materials, and construction material such as concrete and steel. Under the Clean Water Act of 1977, it is required that the designers encourage wastewater treatment techniques that would reduce total energy requirements. Waste treatment alternatives that substantially conserve energy are considered innovative. Therefore, process energy utilization and conservation should be of particular value throughout the planning, project formulation, and preliminary engineering design. Basic energy needs and energy conservation techniques in municipal wastewater treatment plants may be found in Refs. 5, 7, and 28.

2-13 PLANT ECONOMICS

As an integral part of the wastewater treatment plant planning and design, a cost-effective analysis must be performed to ensure that the construction and the operation and maintenance (O&M) are reasonable and appropriate for the planned level of treatment and process train. A cost-effective solution is one that will minimize total costs of the resources over the life of the treatment facility. Resources costs include capital (land plus construction), operation, maintenance and replacements, and social and environmental costs. Benefits from sludge and effluent sale or reuse will partly offset the resources costs.

Many publications and computer programs are available that are extensively used in making the preliminary cost estimates. When preparing cost estimates, proper assumptions, cost curves, and cost indexes should be used. Discussion on cost estimates and cost indexes is given in Appendix C. Procedures for cost estimation and cost-effective evaluation are given in Chapter 6.

2-14 ENVIRONMENTAL IMPACT ASSESSMENT

The National Environmental Policy Act of 1969 (NEPA) was enacted to ensure that federal agencies consider environmental factors in the decision-making process and utilize an interdisciplinary approach in evaluating these issues.[3] The environmental impact assessment must evaluate all impacts—beneficial and adverse, primary and secondary—that may result from the construction of a wastewater treatment facility. The primary impacts are those directly associated with construction and operation of the treatment works. For example, changes in water quality and odors resulting from the plant are the primary impacts. The secondary impacts are indirect, resulting from the growth or change in land use induced or facilitated by the construction of the plant or its associated sewers. Environmental impact assessment for wastewater treatment facilities is covered in Chapters 5 and 6. In-depth coverage on environmental impact assessment may be found in Refs. 4–7.

It is important that the design engineer work closely with the federal, state, and local regulatory agencies that have responsibilities for planning, design, and operation of the wastewater treatment facilities.[10,11,28,29] The regulation of the state agency (state department of health or state water pollution control agency) usually establishes many basic design considerations and standards. The most widely used standards in the past were the *Ten-States Standards*.[30] Now most of the states have developed their own standards. It may often be necessary for the design engineer to deviate from the standards. However, most state officials will take a reasonable approach on these issues if deviations from the state's standards may truly be necessary. Such issues are incorporation of new technology, relative cost savings, or other constraints that may be specific to a particular project.

2-15 PROBLEMS AND DISCUSSION TOPICS

2-1 List the major items of concern for the site selection of a wastewater treatment facility. What agencies would you contact to develop the needed information?

2-2 Visit the wastewater treatment facility in your community. Mark on a map the service area and the plant location. Draw the treatment process train. Briefly summarize the history of wastewater treatment, including major events that helped to bring about improvements. If federal funds were involved under the construction grants program, review Step 1, Step 2, and Step 3 applications.

2-3 Was an environmental impact report prepared for the most recent expansion of the wastewater treatment facility in your community? List the major environmental issues addressed in this report. Have other major issues been identified since the construction and operation of the facility?

2-4 Discuss various factors that may influence the population growth in a community.

2-5 Estimate the year 2000 population of a community by using arithmetic, geometric, decreasing rate of increase, and logistic curve-fitting methods. Use the following census data:

Year	Population in Thousands
1970	31.6
1980	36.9
1990	42.3

2-6 The population and employment data for a city are given below. Estimate the year 2000 population if the employment projection for the year 2000 is 9200.

Year	Population	Employment
1970	20,000	7500
1980	21,000	8000
1990	23,000	8800

2-7 The initial and design years of a wastewater treatment facility are the years 1995 and 2015. The estimated populations for these years are 35,000 and 76,000, respectively. The expected flow rates during these periods are 600 Lpcd. What should the staging period be?

2-8 Obtain the following information for the wastewater treatment facility in your community:
(a) Design year, estimated population, and average flow
(b) Initial year, population, and average flow
(c) Service area in km^2
Determine (1) design period, (2) population density at the design year (person/km^2), (3) ratio of average design flow to initial flow, (4) does staging period correspond to the value given in Table 2-1? and (5) is the estimated population growth pattern similar to the actual population trends of the community?

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3

Wastewater Characteristics

3-1 INTRODUCTION

Municipal wastewater is the general term applied to the liquid wastes collected from residential, commercial, and industrial areas and conveyed by means of a sewerage system to a central location for treatment.

It is important that reliable estimates on wastewater characteristics are made, as this is what the treatment plant will be treating in the future. The purpose of this chapter is to develop information that can be utilized in assessing the characteristics of wastewater for the design of treatment facilities. Since under dry weather conditions, municipal wastewater is derived largely from the water supply, the water usage data under present and anticipated future conditions are essential. Therefore, discussion in this chapter is provided in three general categories: (1) municipal water demands, (2) wastewater flow rates, and (3) chemical quality of municipal wastewater.

3-2 MUNICIPAL WATER DEMAND

3-2-1 Components of Municipal Water Demand

Water demand data are very useful in estimating the wastewater characteristics. The average amount of municipal water withdrawn in the United States is approximately 628 liters per capita per day (Lpcd[a]).[1,2] This amount includes residential, commercial, light industrial, fire fighting, public uses, and water lost or unaccounted for. There is, however, a wide variation in the municipal water withdrawal rate. Factors affecting water withdrawal rates are (1) climate; (2) geographic location; (3) size, density, and economic conditions of the community; (4) degree of industrialization; (5) metered water supply; (6) conservation effort; (7) quality, dependability, and cost of water; and (8) supply pressure. The average water demand by states in the United States is summarized in Table 3-1. Various components of municipal water demand are discussed below.

[a]628 Lpcd = 166 gallons per capita per day (gpcd).

TABLE 3-1 *Average Municipal Water Demand by State*

State	Lpcd	gpcd[a]	State	Lpcd	gpcd	State	Lpcd	gpcd
Alabama	806	213	Maine	553	146	Oregon	712	188
Alaska	1790	473	Maryland	515	136	Pennsylvania	685	181
Arizona	787	208	Massachusetts	530	140	Rhode Island	462	122
Arkansas	503	133	Michigan	636	168	South Carolina	916	242
California	685	181	Minnesota	473	125	South Dakota	549	145
Colorado	746	197	Mississippi	507	134	Tennessee	488	129
Connecticut	541	143	Missouri	485	128	Texas	587	155
Delaware	700	185	Montana	829	219	Utah	1113	294
Florida	617	163	Nebraska	636	168	Vermont	553	146
Georgia	946	250	Nevada	1154	305	Virginia	420	111
Hawaii	746	197	New Hampshire	485	128	Washington	1200	317
Idaho	897	237	New Jersey	526	139	West Virginia	568	150
Illinois	772	204	New Mexico	772	204	Wisconsin	587	155
Indiana	534	141	New York	609	161	Wyoming	746	197
Iowa	466	123	North Carolina	644	170	District of Columbia	799	211
Kansas	587	155	North Dakota	477	126	Puerto Rico	326	86
Kentucky	314	83	Ohio	594	157	United States	628	166[b]
Louisiana	545	144	Oklahoma	492	130			

[a]1 gal = 3.785 L.
[b]Average is based on total municipal water demand for the United States divided by the population served.
Source: Adapted from Ref. 1.

TABLE 3-2 Typical Breakdown of Residential Water Uses

Types of Water Use		Nonconserving Home Usage, Percent
Toilet flush, including toilet leakage		33
Shower and bathing		28
Wash basin		11
Kitchen		9[a]
Drinking, cooking	(2–6%)	
Dishwashing	(3–5%)	
Garbage disposal	(0–6%)	
Laundry and washing machine		16[b]
Lawn sprinkling and miscellaneous		3

[a]Dishwasher is 3%, and remaining is faucet use.
[b]Higher percentage because increased use of washing machines and reduced total demand.
Source: Adapted in part from Refs. 3–8.

Residential Water Use. The residential or domestic water demand is the portion of municipal water supply that is used in homes. It may vary between 30 to 40 percent of the total municipal water demand. Typical breakdown of residential water uses is given in Table 3-2.[3-6] It includes toilet flush, cooking, drinking, washing, bathing, watering lawn, and other uses. The average residential water demand varies from 228 to 456 Lpcd, while most commonly used numbers are 300–380 Lpcd.[2,9,10] In recent years, with flow reduction devices, the total domestic water consumption can drop to less than 210 Lpcd.[7,8] Typical water uses for various household devices are summarized in Table 3-3.[3-8]

TABLE 3-3 Typical Rates of Water Use from Various Devices

Device	Range of Flow
Household faucet	10–20 L/min
Wash basin	4–8 L/use
Shower head	90–110 L/use; 19–40 L/min
Tub bath	60–90 L/use
Toilet flush, tank-type	19–27 L/use
Toilet flush, valve-type	90–110 L/min
Dishwasher	15–30 L/load
Washing machine	110–200 L/load
Lawn sprinkler	6–8 L/min
Continuous flowing drinking fountain	4–5 L/min
Garbage disposal	6000–7500 L/wk, 4–8 L/person per day
Dripping or leaky faucet	10–1000 L/d

Source: Adapted in part from Refs. 3–8.

TABLE 3-4 Average Water Demand in Residential, Institutional, and Commercial Establishments

Source	Unit	Flow (L/unit · d)
Residential		
Single-family		
Low-income	Person	270
Medium-income	Person	310
High-income	Person	380
Summer cottage	Person	190
Trailer park	Person	150
Apartment	Person	230
Hotel, motel	Unit	380
Camps	Person	133
Resort (day or night)	Person	190
Institutional		
Hospital	Bed	950
Rest homes	Bed	380
Prison	Inmate	450
Schools		
Boarding	Student	300
Day	Student	76
Commercial		
Country clubs		
Resident	Member	380
Nonresident	Member	95
Restaurant	Customer	30
Cafeteria	Customer	6
	Employee	40
Bar	Customer	8
	Employee	50
Coffee shop	Customer	20
	Employee	40
Dance hall	Person	8
Store	Toilet room	1520
	Employee	40
Department store	m^2 floor area	8
	Employee	40
Shopping center	m^2 floor area	6
	Employee	40
Office building	Employee	65

Source	Unit	Flow (L/unit · d)
Barber shop	Chair	210
Beauty salons	Station	1026
Laundries		
Laundromat	Machine	2200
Commercial	Machine	3000
Service station	First bay	3800
	Additional bays	1900
	Employee	190
Theaters[a]		
Drive-in	Car space	19
Movie	Seat	8
Airport[a]	Passenger	10
Car wash[a]	Car washed	209
Industrial building	Employee	55
Factories		
With showers	Employee-shift	133
Without showers	Employee-shift	95

[a]Does not contain per day unit.
Source: Adapted in part from Refs. 5, 11, and 12.

Commercial Water Use. Commercial establishments include motels, hotels, office buildings, shopping centers, service stations, movie houses, airports, and the like. The commercial water demand depends on the type and the number of commercial establishments. In cities of over 25,000 population, the commercial water demand is about 10–20 percent of total water demand. Water demands in various types of commercial establishments are given in Table 3-4.[5,11,12]

Industrial Water Use. Industrial water demands are very large in the United States. Generally, large industries develop their own water supply systems. Only small industries purchase water from the cities and therefore impose demand on local municipal systems. Industrial water demand may be estimated on the basis of proposed industrial zoning and type of industries most likely to develop within the city. The average water demand may vary from 9 to 14 m^3/ha · d (1000 to 1500 gal/acre · d) for light industrial and 14 to 28 m^3/ha · d (1500 to 3000 gal/acre · d) for medium industrial developments.[2,9] Typical water demand with small industries in town is 20–35 percent of total municipal water demand.[9] Some industrial water demand data are summarized in Table 3-5.[13,14] The values in this table should be used to estimate the industrial flow rates when data on size and type of industries are available.

TABLE 3-5 Typical Industrial Water Demand

Industrial Use	Quantity
Canning	30–60 m^3/metric ton
Milk, dairy	2–3 m^3/metric ton
Meat packaging	15–25 m^3/metric ton
Cattle	40–50 L/head · d
Dairy	70–80 L/head · d
Chicken	30–40 L/100 · d
Pulp and paper	200–800 m^3/metric ton
Steel	260–300 m^3/metric ton
Tanning	60–70 m^3/metric ton raw hides processed

Source: Adapted in part from Refs. 13 and 14.

Public Water Use. Water used in public buildings (city halls, jails, schools, etc.), as well as water used for public services including fire protection, street washing, and park irrigation, is considered public water use. Public water use accounts for 5–10 percent of total municipal water demand.[9,10]

Water Unaccounted For. In a water supply system there is a certain amount of water that is lost or unaccounted for because of meter and pump slippage, leaks in mains, faulty meters, and unauthorized water connections. In municipal supply systems this may be 8–24 percent of the total water demand.[9] Newer distribution systems in general have less losses than older systems. Faulty meters often are responsible for large errors in water quantities that are unaccounted for. In many developing nations where individual water connections are unmetered and the distribution system is old and leaky, a large portion of supplied water is lost or unaccounted for. To reduce these losses, water is supplied only for a few hours in the mornings and evenings. This practice may save water but may cause serious contamination of water supply due to entry of surface and groundwater into the distribution system.

3-2-2 Variations in Municipal Water Demand

The municipal water demand discussed above is based on annual average daily demand. There are wide variations in seasonal, daily, and hourly water demands. Some of the general observations of municipal water demands are summarized below:

- Working days have higher demand than holidays.
- Hot and dry days have more demand than wet or cold days.
- Maximum months are typically July or August (summer).

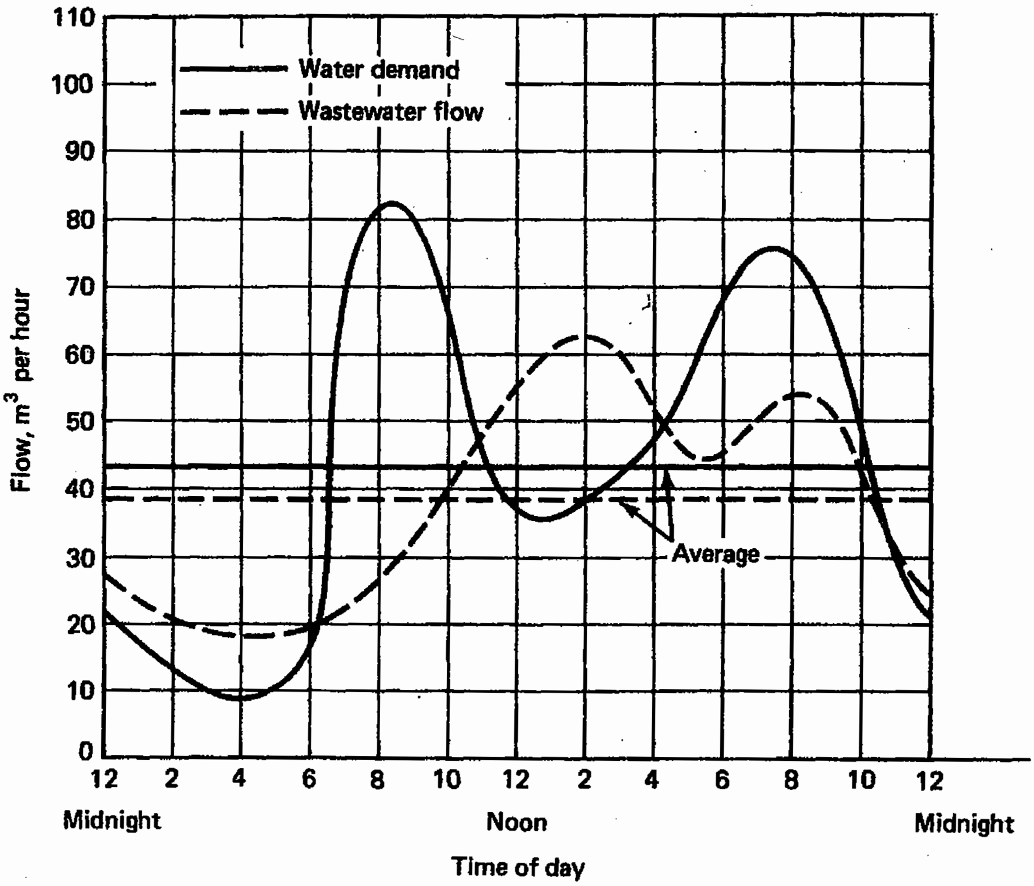

Figure 3-1 Typical Variation in Municipal Water Demand and Wastewater Flow.

- Within a day there are two peak demands. One peak is in the morning as the day's activities start, while the other peak is in the evening.
- The minimum water demand occurs normally around 4 A.M.
- Fluctuations in water use in municipal systems can be estimated using the procedures given in Refs. 7–9. Typical hourly variations in a municipal water demand are shown in Figure 3-1. Typical fluctuations are summarized in Table 3-6.

TABLE 3-6 Typical Fluctuations in Water Demand in Municipal Water Supply Systems

	Ratio of Annual and Average Day Demand	
Condition	**Range**	**Typical**
Maximum day in a year	1.5–2.2	1.80
Daily average in maximum week	1.3–1.6	1.40
Daily average in maximum month	1.1–1.4	1.20
Peak hour within a day	2.7–4.0	2.70[a]

[a]1.5 times the maximum day.

3-3 WASTEWATER FLOW

3-3-1 Relation between Water Supply and Wastewater Flow Rates

Municipal wastewater is derived largely from the water supply. A considerable portion of the water supply, however, does not reach the sewers. This includes water used for street washing, lawn sprinkling, fire fighting, and leakages from water mains and service pipes. A small portion of water may also be consumed in products and manufacturing processes. Also, many homes and other establishments that are not served by a sewerage system may use the city water supply but utilize on-site wastewater treatment and disposal. On the other hand, infiltration/inflow, and water used by industries and residences that is obtained from privately owned sources may make the quantity of wastewater larger than the public water supply. In general, the average wastewater flow may vary from 60 to 130 percent of the water used in the community. Many designers frequently assume that the average rate of wastewater flow, including a moderate allowance for infiltration/inflow, equals the average rate of water consumption. Average wastewater flows from residential, commercial, industrial, institutional, and other sources may be obtained from careful consideration of local water consumption data or by using wastewater generation rates published in the literature.

3-3-2 Infiltration/Inflow

Infiltration is the groundwater that enters the sewers through service connections, cracked pipes, defective joints, and defective pipe and manhole walls. Inflow is the surface runnoff that may enter through a manhole cover, roof and area drains, and cross-connections from storm sewers and combined sewers. This is called direct inflow. Flow from cellar and foundation drains, cooling water discharges, and drain from springs are *steady inflow* and are included with infiltration.

The amount of infiltration/inflow reaching a sewer system depends on the length and age of the sewers; the construction material, methods, and workmanship; the number of illegal roof or drain connections; the groundwater table relative to the sewer position; and the type of soil, ground cover, and topographic conditions. As part of the Federal Construction Grants Program (Chapters 5 and 6), extensive studies of sewer system evaluation and rehabilitation must be undertaken by the municipalities in order to demonstrate that the proposed wastewater treatment facility will not be subject to excessive infiltration/inflow. Procedures for conducting infiltration/inflow evaluation surveys and determining the infiltration/inflow quantities are presented in Chapter 6.

No further I/I analysis will be required if domestic wastewater plus nonexcessive infiltration does not exceed 460 Lpcd (120 gpcd) during periods of high groundwater. The I/I analysis is also not required if the total daily flow during a storm does not exceed 1050 Lpcd (275 gpcd), and there are no operational problems, such as surcharges, bypasses, or poor treatment performance, resulting from hydraulic overloading of the treatment works during storm events.[15] The flow rate of 460 Lpcd for infiltration analysis contains two flow components: 270 Lpcd (70 gpcd) domestic wastewater base flow and 190 Lpcd (50 gpcd) of nonexcessive infiltration. This is a national average based on the results of a need survey of some 270 standard metropolitan statistical area cities. If I/I exceeds the above mentioned criteria, sewer evaluation and rehabilitation may be necessary. It is therefore clear that the maximum infiltration plus inflow allowance is 780 Lpcd (205 gpcd).[b]

In older sewer lines infiltration is high because of deterioration of joints and masonry mortar. Newer sewers use joints sealed with rubber gaskets or synthetic material and precast manhole sections. Therefore, the infiltration rate is significantly smaller. When designing for sewers, allowance must be made for old and new constructions. Average values for infiltration allowance used by designers are 94–9400 L/d/cm/km (100–10,000 gal/d/in./mi) or 200–28,000 L/ha/d (20–3000 gal/ac/d).[2] Higher rates may be allowed where adverse conditions may exist.

3-3-3 Flow Variations

Like water demand, wastewater flows vary according to the season of the year, weather conditions, day of the week, and time of the day. Under dry weather conditions, the daily wastewater flow shows a diurnal pattern. Figure 3-1 illustrates daily variations in water demand and wastewater flows. The wastewater curve closely parallels that of water demand with a lag of several hours. Also, the fluctuations in wastewater flows are less than that of water supply because of the storage space in the sewers and because of the time required for the wastewater to reach the treatment plant. Also, the commercial and industrial discharges tend to reduce the peak flows. The infiltration/inflow further changes the diurnal flow pattern.

It is important that the design engineers clearly understand the significance of flow variations in the overall context of planning, design, and operation of wastewater treatment plant. These terms are briefly presented below:

1. Average daily flow: The average flow (also called average annual flow) is based on annual flow rate data. It is the average flow occurring over a 24-h period under dry weather conditions. It is normally used to evaluate (1) treatment plant capacity, (2) develop flow rate ratios, (3) size many treatment units, (4) calculate organic loadings, (5) estimate sludge solids, (6) estimate chemical needs, and (7) calculate pumping and treatment unit costs.
2. Maximum dry weather flow: The maximum dry weather flow rate occurs during a dry day. This is the maximum flow on a typical dry weather diurnal flow curve (Figure 3-1). Although this flow occurs for a short time, it is important because it brings about flushing action in sewers, conduits, and channels. It is used to design and check the retention time of several critical components of a treatment facility.
3. Peak wet weather flow: The peak wet weather flow occurs during or after precipitation and includes a substantial amount of I/I. Some regulatory agencies define peak

[b]Permissible I/I allowance of 780 Lpcd = 1050 Lpcd − 270 Lpcd (domestic wastewater base flow).

wet weather flow as the highest *2-hour flow* encountered under any operational conditions, including times of highest rainfall (generally the 2-year, 24-h storm is assumed) and prolonged periods of wet weather. For municipal systems the peak flow to average annual flow ratio is normally in the range of three–five to one, although other peaking factors may be warranted.

Peak hourly flow data (or highest 2-hour flow) is needed (1) to design interceptor sewers and collection system, (2) for conduits and connecting pipings and channels in a treatment plant, (3) for pumping stations, (4) for flow meters, (5) for bar screen and grit channels, (6) to check the retention period of many critical treatment units, and (7) to design the hydraulics of the influent and effluent structures of the treatment units.

4. Minimum hourly flow: The minimum hourly flow is the lowest flow on a typical dry weather diurnal flow curve (Fig. 3-1). Low flows may cause settling of solids in pipes and channels. These flow rates are needed for sizing of flow meters, chemical feeder, and pumping equipment. Often, recirculation is needed during low flows to maintain hydraulic loading.
5. Sustained flows: The sustained flows occur over several days under special events in the community. Sustained flows could be on the low or high side. This information is needed to check the design of a wastewater treatment facility under extreme adverse conditions (see Sec. 3-5).

Procedure for developing minimum, average, and peak dry weather flows, infiltration/inflow allowances, and peak wet weather flows are presented in detail in Chapter 6. These procedures are based on long-term flow measurements at the wastewater treatment facility. In the absence of flow records, many designers use the following procedure to develop the design flows:

1. The population estimates, industrial and commercial growths, and land use patterns are developed for the initial and design years (see Sec. 2-2). Procedures for making population estimates and development of land use plans are covered in Chapters 2 and 6.
2. Based on the current per capita water demands and future trends, average water usage data for the initial and design years are developed.
3. Average wastewater flow data are developed from water usage. Considerations are given to portions of the water lost due to lawn sprinkling, street washing, and leakage. Also, the portion of wastewater that may be lost because of exfiltration is considered in the analysis. Many designers assume that the average dry weather flow is approximately 80 percent of the water demand. Another method involves determination of wastewater flow rates from different types of residential subdivisions; commercial contribution per employee or per acre; industrial flows based on per employee contribution, unit product, per acre, or industry as a whole; and institutional wastes per person.[16,17]
4. Peak and minimum dry weather flows are estimated from several equations and graphical relationships developed from case studies. The ratios of peak to average and

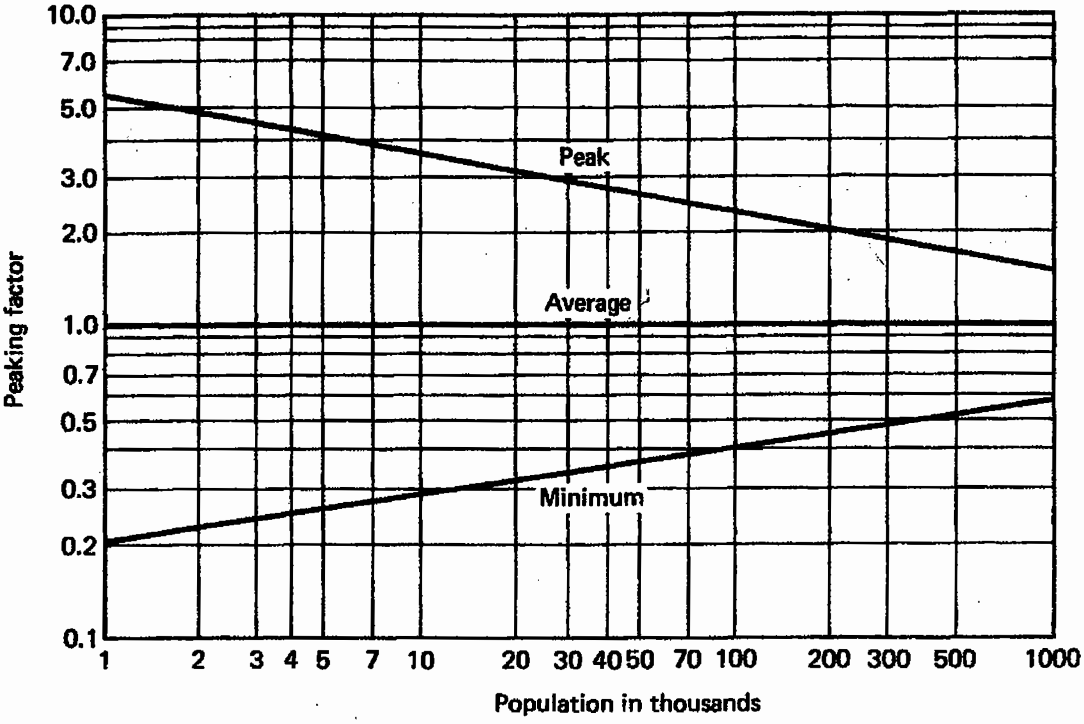

Figure 3-2 Ratios of Extreme Flows to Average Daily Flow (Peaking Factor) for Municipal Wastewater Under Dry Weather Conditions (from Ref. 16; used with permission of Water Environment Federation, and American Society of Civil Engineers).

minimum to average flows depend on the population. Larger cities have less deviations from the average than the smaller cities. Several methods for estimating ratios of peak and minimum flows to average flows are given in Refs. 9, 10, and 16–19. Commonly used equations are given below [Eqs. (3-1), (3-2), and (3-3)]. Ratios of extreme flows to average daily flow (peaking factors) developed from various sources are given in Figure 3-2.

$$M = 1 + \frac{14}{4 + \sqrt{P}} \qquad (3\text{-}1)^{9,16}$$

$$M = \frac{5}{P^{0.167}} \qquad (3\text{-}2)^{18}$$

P = population in thousands

$$Q_{\max} = 3.2 Q_{\text{avg}}^{5/6} \qquad (3\text{-}3)^{19}$$

where

M = ratio of maximum flow ($Q_{\max}$) to average flow (Q_{avg}).

Many designers use the "fixture-unit" load method for estimating peak wastewater flows for facilities such as hospitals, hotels, schools, apartment buildings, and office buildings.[20,21] A procedure for estimating peak flows from the fixture unit method is given in Ref. 16.

5. The peak hourly wastewater flow is the peak dry weather flow plus the infiltration/inflow. In relation to the water supply, this is the peak hourly rate of water demand multiplied by the proportion of the water supply reaching the collection system, plus infiltration/inflow (fire demand does not enter into these calculations).[9,22] In Chapter 6 a procedure for estimating peak hourly flow from flow measurement data is provided.
6. Minimum rates of wastewater flow are useful in the design of pumping stations and to investigate the velocity in sewers and channels during the low flows. In the absence of gauging data, minimum flow may be assumed to be 33 percent and 50 percent of average flows for small- and medium-sized communities, respectively.
7. Sustained flows are the flows that persist for various time durations. Peak and minimum flows last only for brief periods (less than 2 h), while sustained flows are those extremes that may persist over longer durations. As an example, extraordinarily dry or hot weather may cause sustained low extremes. Likewise, special events in a community such as fairs, exhibitions, conventions, games, and the like may influence large population increases causing high sustained flows. These sustained flows are important in the design of wastewater treatment facilities. A procedure for developing sustained flow-envelope curves from flow records is given in Ref. 2. The ratios of one-day sustained peak and low flows to average flow are 2.9 and 0.4, respectively.[2]
8. In the absence of flow measurement records and other pertinent data, many states specify 1500 L (400 gal) per capita per day for design of laterals and submains and 1300 L (350 gal) per capita per day for mains and trunk sewers. These design flows include normal infiltration/inflow. Additional allowance must be made where industrial wastes are also transported, and conditions favoring excessive infiltration and inflow are present.[9]

3-3-4 Flow Reduction

The construction grants program requires a close examination of various wastewater flow reduction methods as part of facility planning process. Reduction in wastewater volume is achieved by (1) water conservation, (2) reuse of water in homes, and (3) reduction of infiltration/inflow.

Water Conservation. Water conservation in homes, commercial establishments, and industries is gaining popularity. Water conservation has the beneficial effect of reducing the water and sewer bills. A number of inexpensive devices is now available as part of most faucet combinations. A community awareness program is necessary for the general public to understand the importance of water conservation and the cost and benefits of the water-saving devices. Water savings as high as 15–30 percent can be achieved by using flow reduction devices and by practicing simple water conservation measures in homes, businesses, and industries. Principal flow reduction devices and systems fall into two categories: (a) retrofit devices used with conventional fixtures and (b) water-conserving devices. Many of these devices and the percent reduction that may be achieved over conventional devices are summarized in Table 3-7.[4,23-25] Many water-saving ideas for homes are listed in Table 3-8.[6,25,26] Water conservation efforts require public cooperation and the efforts of the local governments in the form of plumbing codes.

Reuse of Water in Homes. Another form of in-house water savings might come from reuse of water. The recycled water may be the waste streams from sinks, bathtubs, showers, and laundry, which may be treated on-site and reused for toilet flushing and lawn sprinkling. Such an effort may provide a 30–40 percent reduction in water consumption and wastewater volume.

Reduction in Infiltration/Inflow. New sewers must utilize tight sewer connections and joints in order to reduce infiltration/inflow. Proper selection of appurtenances is also essential. For existing sewer systems, an effective sewer evaluation and rehabilitation program is necessary to reduce the undesired quantities of ground- and surface water. Large volumes of infiltration/inflow under wet weather conditions pose serious hydraulic overload difficulties at the treatment plants.

3-3-5 Flow Measurement

The knowledge of average and diurnal variations in wastewater flows, and sustained flows is essential for design and operation of wastewater treatment facilities. Many types of flow measurement devices are available that can be installed in the intercepting sewers, force mains, or at any location within a plant. Various flow measurement devices, their application and selection criteria, and design procedures are presented in detail in Chapter 10.

3-4 QUALITY OF WASTEWATER

Municipal wastewater contains over 99.9 percent water. The remaining materials include suspended and dissolved organic and inorganic matter, as well as microorganisms. These materials give physical, chemical, and biological qualities that are characteristic of residential and industrial wastewaters. In this section, physical, chemical, and biological qualities and variations in constituents and loadings in wastewater are discussed.

3-4-1 Physical Quality

The physical quality of municipal wastewater is generally reported in terms of temperature, color, odor, and turbidity. These physical parameters are summarized in Table 3-9.

3-4-2 Chemical Quality

The chemical quality of wastewater is expressed in terms of organic and inorganic constituents. Domestic wastewater generally contains 50 percent volatile and 50 percent

TABLE 3-7 Principal Flow-Reduction Systems and Devices, and Percentage Reduction over Conventional Devices

Flow Reduction System/Device	Description	Percentage Reduction in Water Use over Conventional System/Device
Pressure Reducing Valves (PRV)	Pressure reducing valves are installed to reduce the home pressure lower than the city water supply pressure. This eliminates unnecessary waste caused by spurts from faucet, leaks, and drips.	20–30
Sink Faucet		
Faucet aerator	Designed to provide smooth and even flow of water. By adding air, the aerated jet of water increases rinsing power of water, thus reducing amount of wash water used in kitchen and bathroom sink.	1–2
Limiting flow valve	Restrict flow to a constant rate independent of supply pressure.	1–2
Shower		
Flow limiting shower head	The device reduces water consumption by restricting and concentrating water jet.	10–12
Limiting flow valves	Restrict flow to a fixed rate, independent of supply pressure.	8–10
Washing Machine		
Level controller or water efficient washer	Level setting uses water in accordance to the wash load, or machine designed to use less water.	2–4
Dishwasher		
Water efficient dish washer	Machine designed to use less water.	4–10

Toilet		
Shallow trap water closet	Shallow trap toilets are similar to conventional toilets. Due to shallow trap they require less water per flush. These are cost-effective for new homes.	30–40
Dual-cycle toilet inserts	Toilet insert devices convert conventional toilets to dual-cycle operation (separate for urine or fecal waste transport).	15–18
Dual-cycle toilet	New types of toilets that have two flush cycles, one for urine and the other for fecal transport. Recommended for new constructions.	25–30
Reduced-flush device	Toilet-tank inserts that reduce the volume of flush by occupying a portion of the tank, or prevent the tank from completely emptying.	12–18
Flush valve: single-flush valve and dual-flush valve	The flush valves operate directly from the water supply lines. Controlled amount of water is used for flushing. The mechanism can also be used with shallow trap toilets. Recommended for new homes.	10–14 20–30
Immersed bottle brick or toilet dam in toilet tank	Plastic bottle filled with water and weighted with pebbles occupy tank space and reduce water used per flush (obstruction to float must be avoided). Bricks are also used, but they flake and may clog tubes and valves.	2–5
Toilet leak detector	Tablets that dissolve in tank and release dye to indicate leak.	—
Vacuum-flush toilet system	The toilets use air and small amount of water or foam to transport waste. Such toilets are commonly used in commercial aircraft.	30–40
Recirculating toilet system	The flush water is recirculated for certain period. The system operates on a closed-loop principle in which waste is accumulated in a holding tank. Chemicals are used to suppress the microbiological activity in recirculating water. Other innovations utilize mineral oil as flushing medium. The holding tank is emptied periodically by vacuum trucks. Such toilets are commonly used in commercial aircraft.	25–45
Urinals	Use of urinals in homes reduce water consumption. Wall-type urinals with flush valve for homes use 5–6 L per flush.	10–15

Source: Adapted in part from Refs. 4, 7, and 23–26.

TABLE 3-8 Water-Saving Ideas for Homes

Location	Water-Saving Ideas
Bathtub	A full bathtub holds 190 L. One can bathe adequately in one-quarter full bathtub.
Shower	Water consumption in quick shower is less than bathing (40–70 L). Turn off water while soaping up. Use light spray.
Toilet	Do not flush more than necessary. Use water-saving devices discussed in Table 3-7. Water leaks or waste can be detected by adding a few drops of food coloring in the tank. Coloring appears in the toilet if there is a leak.
Washing machine	Use load selector for large or small loads if there is one. Otherwise, wash only full loads. Using cold water saves energy. Buy a machine that uses less water and energy per unit weight of wash. Use suds saver attachment and use less detergent.
Dishwasher	Preclean dishes with used paper napkins. Wash full load. Use less detergent.
Utility sink	Preclean dishes with used paper napkins. Soak overnight with small quantities of low-sudsing detergent. Save rinse water for next soak. Plan ahead to thaw frozen foods in air, not in running water.
Drinking water	Do not run tap waiting for cold water. Use ice cube or keep water pitcher in the refrigerator. Use paper cups at water fountain to avoid waste.
Garbage disposal	Avoid using garbage grinders. Use leftover food for feeding pets. Start a compost pile.
Bathroom sink	Turn off water while shaving or brushing teeth.
Faucet	Check all faucets including outside hose connections for leaks. Replace worn washers, O rings, packing, and faulty fixtures. A pinhole leak can waste up to 1000 L water per day. In many areas leaks cause about 95 percent complaints about excessive water bills.
Lawn, garden	Water slowly, thoroughly, and as infrequently as possible. Water at night to minimize evaporation. Keep close watch on wind shifts. Select hardy species and native plants that do not need as much watering. Mulch heavily. Let grass grow higher in dry weather—saves from burning, and less water is needed. Recycle water from bathtub, kitchen sinks, etc. A 1.25-cm garden hose under normal water pressure pours out more than 2300 L/h. A 2-cm hose uses almost 7000 L/h.
Backyard pool	Cover when not in use to prevent evaporation, keep clean, and reduce algae growth. Recycle wading pool water for plants, shrubs, lawns.
Carwash	Try to wash car near hedges, shrubs, or lawn. Wash car in short spurts from hose. Use a commercial carwash that recycles water.

Source: Adapted in part from Refs. 6, 7, 25, and 26.

TABLE 3-9 Physical Quality of Wastewater

Parameter	Description
Temperature	The temperature of wastewater is slightly higher than that of water supply. Temperature has effect upon microbial activity, solubility of gases, and viscosity. The temperature of wastewater varies slightly with the seasons, but is normally higher than air temperature during most of the year and lower only during the hot summer months.
Density	The density of municipal wastewater is essentially the same as that of water (1000 kg/m^3)
Color	Fresh wastewater is light gray. Stale or septic wastewater is dark gray or black.
Odor	Fresh wastewater may have a soapy or oily odor, which is somewhat disagreeable. Stale wastewater has putrid odors due to hydrogen sulfide, indol and skatol, and other products of decomposition. Industrial wastes impart other typical odors. Because of odors associated with wastewater treatment facilities, area residents have often vigorously resisted and rejected wastewater treatment plant projects.
Turbidity	Turbidity in wastewater is caused by a wide variety of suspended solids. In general, stronger wastewaters have higher turbidity.

fixed material, although, the ratio of volatile to fixed solids in dissolved and suspended solids differ greatly. Different chemical analyses furnish useful and specific information with respect to the quality and strength of wastewater. The uniformity of procedures prescribed in the Standard Methods made a comparison of different results possible.[27] The typical composition of domestic wastewater and brief descriptions and significance of different tests are summarized in Table 3-10. A general discussion on organic components, total suspended solids, and inorganic salts of wastewater is given below.

Organic Components. The major components of organic matter in domestic wastewater are carbohydrates, proteins, and fats, oils, and grease. Carbohydrates and proteins are easily biodegradable. Fats, oils, and grease are more stable, and are decomposed slowly by microorganisms. Wastewater may also contain small fractions of synthetic detergents, phenolic compounds, and pesticides and herbicides. These compounds, depending on their concentrations, may create problems such as nonbiodegradability, foaming, or carcinogenicity.[28-30] The concentrations of these toxic organic compounds in municipal wastewaters are very small. Their sources may be industrial wastes and surface runoffs. These pollutants are subject to "categorical standards" in the industrial pretreatment standards. Discussion on pretreatment standards is provided in Chapter 2. Additional information is provided in subsequent sections.

TABLE 3-10 Typical Chemical Quality of Raw Domestic Wastewater

Chemical Quality Parameters	Description	Concentration Range	Concentration Typical
Total solids	Organic and inorganic, settleable, suspended and dissolved matter.	375–1800	730
Settleable, mL/L	Portion of organic and inorganic solids that settles in 1 h in an Imhoff cone. These solids are approximate measure of sludge that is removed in a sedimentation basin.	5–20	10
Suspended (TSS), mg/L	Portion of organic and inorganic solids that are removed by filtration through a glass-fiber or polycarbonate membrane filter. These solids may fall into colloidal range.	120–360	230
Fixed, mg/L	Noncombustible or mineral components of total suspended solids.	30–80	55
Volatile, mg/L	Combustible or organic components (at 550 ± 50°C) of total suspended solids.	90–280	175
Dissolved (total), mg/L	Portion of organic and inorganic solids that is not filterable. Solids smaller than one millimicron (mμ)[a] fall in this category.	250–800	500
Fixed, mg/L	Noncombustible or mineral components of total dissolved solids.	145–500	300
Volatile, mg/L	Combustible or organic components (at 550 ± 50°C) of total dissolved solids.[b]	105–300	200
BOD_5, mg/L	Biochemical oxygen demand (5-d, 20°C). It represents the biodegradable portion of organic component. It is a measure of dissolved oxygen required by microorganisms to stabilize the organic matter in 5 days.	110–400	210
COD, mg/L	Chemical oxygen demand. It is a measure of organic matter and represents the amount of oxygen required to oxidize the organic matter by strong oxidizing chemicals (potassium dichromate) under acidic condition.	200–780	400
TOC, mg/L	Total organic carbon is a measure of organic matter. TOC is determined by converting organic carbon to carbon dioxide. It is done in a high-temperature furnace in presence of a catalyst. Carbon dioxide is quantitatively measured	80–290	150

Total nitrogen (TN),[c] mg/L	Total nitrogen includes organic, ammonia, nitrite, and nitrate nitrogen. Nitrogen and phosphorus along with carbon and other trace element serve as nutrients thus accelerate the aquatic plant growth in natural water.	20–85	40
Organic (ON) (as N), mg/L	It is bound nitrogen into protein, amino acid, and urea.	8–35	20
Ammonia ($NH_3 - N$) (as N), mg/L	Ammonia nitrogen is produced as first stage of decomposition of organic nitrogen.	12–50	20
Nitrite and nitrate (as N), mg/L	Nitrite and nitrate nitrogens are the higher oxidized forms of nitrogen. Both of these forms of nitrogen are absent in raw domestic wastewater.	0–small	0
Total phosphorus (TP),[d] mg/L	Total phosphorus exists in organic and inorganic form. Phosphorus in natural water is a source of eutrophication.	4–8	6
Organic (as P), mg/L	Organic phosphorus is bound in proteins and amino acids.	1–3	2
Inorganic (as P), mg/L	Inorganic form of phosphorus exists as orthophosphate and polyphosphate.	3–6	4
pH	pH is indication of acidic or basic nature of wastewater. A solution is neutral at pH 7.	6.7–7.5	7.0
Alkalinity (as $CaCO_3$), mg/L	Alkalinity in wastewater is due to presence of bicarbonate, carbonate, and hydroxide ion.	50–200	100
Hardness (as $CaCO_3$), mg/L	Hardness in wastewater is primarily due to calcium and magnesium ions. Hardness of wastewater depends on the hardness of water supply.	180–350	240
Chloride, mg/L	Chloride in wastewater comes from water supply, human wastes, and domestic water softeners.	30–100	50
Oils and grease, mg/L	These are soluble portion of organic matter in hexane. Their sources are fats and oils used in foods.	50–150	100

[a]micrometer (μm) = 10^{-6} m [also referred as micron (μ)]; nanometer (nm) = 10^{-9}. This unit was also referred to as millimicron ($m\mu$); angstrom (Å) = 10^{-10}m.
[b]Volatile fraction is obtained after ignition in a muffle furnace at 550 ± 50°C.
[c]$TN = ON + (NH_3 - N) + (NO_2 - N) + (NO_3 - N)$. Total Kjeldhal nitrogen (TKN) = $ON + (NH_3 - N)$.
[d]The concentration of phosphorus in municipal wastewaters in the United States has, in general, been falling over the past decade. In late 1960s typical total P concentrations in raw wastewater were 10–12 mg/L. Currently, concentrations are usually in the range of 3–7 mg/L where phosphorus-based detergents are regulated. It is expected that in future the concentration of P in municipal wastewater will decrease.

Measurement of Organic Content. Most methods used for measurement of organic matter in wastewater are indirect measures. Some of these are presented below.

Biochemical Oxygen Demand. Biochemical oxygen demand (BOD_5) is the most commonly used parameter to express the strength of municipal and industrial wastewaters. The BOD_5 test is important for the design of biological treatment facilities, determining organic loadings to the treatment plants, and evaluating the efficiency of treatment systems. The test is also useful in stream pollution control activities. By its use, it is possible to determine the degree of organic pollution in streams at any time. The test is of prime importance in regulatory work and in studies designed to evaluate the purification capacity of receiving waters.

BOD is defined as the amount of oxygen utilized by a mixed population of microorganisms under aerobic condition to stabilize the organic matter. The BOD test is conducted by placing a measured amount of wastewater in a 300-mL standard BOD bottle and filling the bottle with dilution water that contains the essential nutrients and is saturated with dissolved oxygen. Well-acclimated microbial seed may be supplied if sufficient microorganisms are not already present in the wastewater sample. Seeding is generally used for industrial wastewaters. Municipal wastewater is mostly well seeded. The dissolved oxygen is determined in the diluted sample initially and after incubation at 20°C for 5 d.[27]

The BOD of wastewater is not a single point, but rather a time-dependent variable. A 5-d incubation has become a standard practice. It takes approximately 20 d to stabilize all biodegradable organic matter and nitrogen contained in a wastewater sample. The ultimate first-stage BOD is the total carbonaceous oxygen demand. The nitrogenous oxygen demand is called second-stage BOD. Discussions on carbonaceous and nitrogenous oxygen demands may be found in many references.[2,10,19,30] A typical BOD curve with time is shown in Figure 3-3. Different BOD relationships are summarized in Eqs. (3-4) through (3-7). The BOD_5 of municipal wastewater is given in Table 3-10. The effluent requirements of BOD_5 for secondary treatment are discussed in Sec. 2-6-2.

$$\mathrm{UOD} = L_o + L_n \tag{3-4}$$

$$y = L_o(1 - e^{-Kt}) \text{ or } L_o\,(1 - 10^{-kt}) \tag{3-5}$$

$$y = \frac{(D_1 - D_2) - (B_1 - B_2)f}{P} \tag{3-6}$$

$$K_T = K\theta^{T-20} \tag{3-7}$$

where

B_1 and B_2 = dissolved oxygen of seeded control before and after incubation, mg/L

D_1 and D_2 = dissolved oxygen of diluted samples immediately after preparation and after incubation at 20°C, mg/L

f	= ratio of seed in control sample to seed in seed control (% seed in diluted sample)/(% seed in seed control)
K	= reaction rate constant to the base e, at 20°C, d^{-1} (for domestic wastewater, $K = 0.2$–$0.3\ d^{-1}$)
k	= reaction rate constant to the base 10, at 20°C ($k = K/2.303$)
K_T	= reaction rate constant to the base e at any temperature T°C, d^{-1}
L_o	= ultimate carbonates BOD_5, or first-stage BOD_5, mg/L (for domestic wastewater BOD_5 is approximately equal to 2/3 L_o)
L_n	= ultimate nitrogenous oxygen demand, or second stage BOD, mg/L
P	= decimal volumatic fraction of sample used
θ	= a constant, the value may vary depending upon the temperature range. Most common value used is 1.047.
UOD	= ultimate oxygen demand, mg/L

Oxygen is also used for conversion of ammonia into nitrite and nitrate. This is called nitrogenous oxygen demand. Nitrification reactions are shown in Figure 3-3. A total of 2 moles of O_2 is consumed for nitrification of each mole of N. Therefore, nitrogenous oxygen demand is 4.57 g BOD_L per g N.

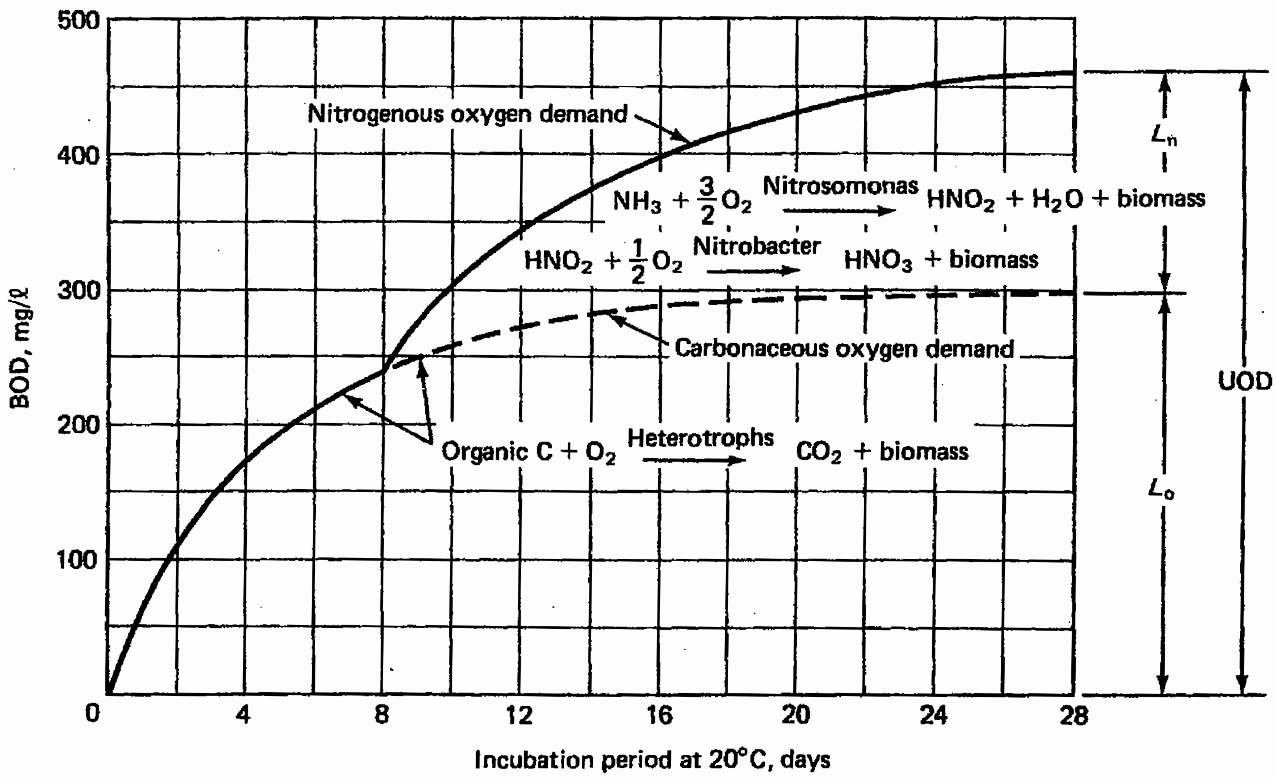

Figure 3-3 Typical BOD Curve for Domestic Wastewater Showing Carbonaceous and Nitrogenous Oxygen Demands.

Other Methods. Although the BOD_5 test is universally used to express the organic strength of wastewaters, this test has many serious limitations. The test requires a 5-d period for a result. Furthermore, preparation of acclimated seed for industrial and toxic wastes is required. In addition, only biodegradable soluble organics are measured, and nitrification may cause serious interference.

There are a number of other tests that are more rapid tools to quantify the organic matter in wastewater samples. *Chemical oxygen demand* (COD) and *total organic carbon* (TOC) tests have been presented in Table 3-10. Other tests that are also used to express the organic strength of wastewaters are *total oxygen demand* (TOD) and *theoretical oxygen demand* (ThOD). TOD measurement uses the conversion of organic compounds into stable oxides in a platinum-catalyzed combustion chamber. This test is quickly performed if instrumentation is available. ThOD measurement is based on stoichiometric relations. There are certain relations among BOD_5, COD, TOC, TOD and ThOD tests. For each wastewater such relations must be established by laboratory measurement program. The degree of correlation depends on the consistency of the constituents. The typical ratios between different tests for domestic and many industrial wastewaters are available in the literature.[2,31,32] Methods commonly used for measurement of organic matter in water and wastewater are summarized in Table 3-11.

Total Suspended Solids. The total suspended solids (TSS) in wastewater may be caused by sand, silt, clay, and organic matter. These are filterable solids removed with a filter having a particle retention of 1.5 μm. The typical total suspended solids concentration in the municipal wastewater is 230 mg/L (Table 3-10). The suspended solids when discharged into the natural waters may increase turbidity of the water and when they settle to the bottom may ruin the spawning and breeding grounds of aquatic animals. Organic solids at the bottom progressively decompose using up dissolved oxygen and produce noxious gases. Therefore, the secondary effluent standards are defined in terms of BOD_5 and TSS. These effluent standards are discussed in Sec. 2-6-2.

Total Dissolved Solids. Total filterable solids are considered dissolved although colloidal particles in the range of 10^{-3} to 1 μm may be included.[2] Total dissolved solids of wastewater fall into two general categories: nontoxic and toxic.

Nontoxic Dissolved Solids. Municipal wastewater contains many dissolved salts that may alter chemical qualities such as pH, buffering capacity, salinity, scaling, or corrosiveness of the wastewater. Many compounds include calcium, magnesium, sodium, potassium, iron, and manganese salts of carbonate, bicarbonate, chloride, sulfate, nitrate, and phosphate.[33] To some extent their concentrations depend on the chemical makeup of the water supply, and the use of domestic and industrial water softeners. The mineral makeup of wastewater is important in evaluating the reuse potential of wastewater. Concentrations and significance of nutrients (P and N), pH, alkalinity, total dissolved solids, hardness, and chloride in wastewater are given in Table 3-10. Many soluble organic compounds in wastewater are biodegradable and serve as food to the microorganisms. Others may be nonbiodegradable. A detailed discussion of the significance and measurement of minerals in wastewater may be found in some excellent references.[2,27,34-36]

TABLE 3-11 Methods Commonly Used for Measurement of Organic Matter in Water Samples

Measurement	Description
Nonspecific	
BOD_5	Biochemical oxygen demand (5-d, 20°C), represents the amount of oxygen required to stabilize the organic matter by microorganisms under aerobic condition. It represents the biodegradable portion, and is conducted over 5 days and at 20°C.
COD	Chemical oxygen demand represents the amount of oxygen required to oxidize the organic matter by strong oxidizing chemical (potassium dichromate) under acidic condition.
TOC	Total organic carbon represents organic carbon present in the organic matter. It is measured by converting organic carbon to carbon dioxide in a high-temperature furnace in the presence of a catalyst.
TOD	Total oxygen demand measurement uses the conversion of organic compounds into stable oxides in a platinum-catalyzed combustion chamber.
ThOD	Theoretical oxygen demand measurement is based on stoichiometric relation when the chemical formula of the organic matter is known.
Color	Most organic matters in water cause color. Therefore, color measurement is used to quantify organics. Colorimetric analysis is performed to measure the color.
UV absorbance	Ultraviolet absorbance is at a specific wave length (UV 254 nm) used to quantify groups of organic compounds such as aliphatic, aromatic, complex multiaromatic, and multiconjugated humic substances.
Fluorescence	Some organic compounds absorb UV energy and then release energy at some longer wavelength. This phenomenon provides a basis for measurement of organics.
Specific	
Gas chromatography	The sample is vaporized and swept by a carrier gas through a chromatographic column. The emergence of the compound is detected and measured.
Mass Spectroscopy	The sample is vaporized and the compounds are separated by gas chromatography. The bombardment of the organic molecules by the rapidly moving electrons breaks the organic molecule into charged fragments. The mass-to-charge ratio of each fragment provides quantitative analysis.
High-pressure liquid chromatography	The carrier stream is composed of a solvent or mixture of solvents maintained under high pressure. The compounds are separated in a solid or liquid stationary phase and measured.

Source: Adapted in part from Refs. 2, 9, 10, 19, 23, 27, and 30.

Toxic Dissolved Solids. Many organic and inorganic solids in wastewater may interfere with the treatment plant performance, pass through unchanged, transformed, generated, or accumulated in the sludge. Both organic and inorganic compounds are included in this category. There are approximately 129 priority pollutants in 65 classes to be regulated by categorical discharge standards. They are selected on the basis of their known or suspected carcinogenicity, mutagenicity, teratogenicity, or high acute toxicity.[2,23] Many of the organic priority pollutants are also classified as volatile organic compounds (VOCs). The EPA priority pollutant list is provided in Table 3-12.[29,30] These pollutants are sub-

TABLE 3-12 List of EPA Priority Pollutants

1. *acenaphthene
2. *acrolein
3. *acrylonitrile
4. *benzene
5. *benzidine
6. *carbon tetrachloride (tetrachloromethane)
***chlorinated benzenes** (other than dichlorobenzenes)
7. chlorobenzene
8. 1,2,4-trichlorobenzene
9. hexachlorobenzene
***chlorinated ethanes** (including 1,2-dichloroethane, 1,1,1-trichloroethane and hexachloroethane)
10. 1,2-dichloroethane
11. 1,1,1-trichloroethane
12. hexachloroethane
13. 1,1-dichloroethane
14. 1,1,2-trichloroethane
15. 1,1,2,2-tetrachloroethane
16. chloroethane ***chloroalkyl ethers** (chloromethyl, chloroethyl and mixed ethers)
17. bis (chloromethyl) ether
18. bis (2-chloroethyl) ether
19. 2-chloroethyl vinyl ether (mixed)
***chlorinated naphthalene**
20. 2-chloronaphthalene
***chlorinated phenols** (other than those listed elsewhere; includes trichlorophenols and chlorinated cresols)
21. 2,4,6-trichlorophenol
22. parachlorometa cresol
23. *chloroform (trichloromethane)
24. *2-chlorophenol ***dichlorobenzenes**
25. 1,2-dichlorobenzene
26. 1,3-dichlorobenzene
27. 1,4-dichlorobenzene
***dichlorobenzidine**
28. 3,3-dichlorobenzidine
***dichloroethylenes** (1,1-dichloroethylene and 1,2-dichloroethylene)
29. 1,1-dichloroethylene
30. 1,2-trans-dichloroethylene
31. *2,4-dichlorophenol
***dichloropropane and dichloropropylene**
32. 1,2-dichloropropane
33. 1,2-dichloropropylene (1,3-dichloropropene)
34. *2,4-dimethylphenol
***dinitrotoluene**
35. 2,4-dinitrotoluene
36. 2,6-dinitrotoluene
37. *1,2-diphenylhydrazine
38. *ethylbenzene
39. *fluoranthene ***haloethers** (other than those listed elsewhere)
40. 4-chlorophenyl phenyl ether
41. 4-bromophenyl phenyl ether
42. bis (2-chloroisopropyl) ether
43. bis (2-chloroethoxy) methane ***halomethanes** (other than those listed elsewhere)
44. methylene chloride (dichloromethane)
45. methyl chloride (chloromethane)
46. methyl bromide (bromomethane)
47. bromoform (tribromomethane)
48. dichlorobromomethane
49. trichlorofluoromethane
50. dichlorodifluoromethane
51. chlorodibromomethane
52. *hexachlorobutadiene
53. *hexachlorocyclopentadiene
54. *isophorone
55. *naphthalene

TABLE 3-12 List of EPA Priority Pollutants—cont'd

56. *nitrobenzene ***nitrophenols** (including 2,4-dinitrophenol and dinitrocresol)
57. 2-nitrophenol
58. 4-nitrophenol
59. 2,4-dinitrophenol
60. 4,6-dinitro-o-cresol ***nitrosamines**
61. *N*-nitrosodimethylamine
62. *N*-nitrosodiphenylamine
63. *N*-nitrosodi-n-propylamine
64. *pentachlorophenol
65. *phenol **phthalate esters**
66. bis (2-ethylhexyl) phthalate
67. butyl benzyl phthalate
68. di-n-butyl phthalate
69. di-n-octyl phthalate
70. diethyl phthalate
71. dimethyl phthalate ***polynuclear aromatic hydrocarbons**
72. benzo (a) anthracene (1,2-benzanthracene)
73. benzo (a) pyrene (3,4-benzopyrene)
74. 3,4-benzofluoranthene
75. benzo (k) fluoranthane (11,12-benzofluoranthene)
76. chrysene
77. acenaphthylene
78. anthracene
79. benzo (g,h,i) perylene (1,12-benzoperylene)
80. fluorene
81. phenanthrene
82. dibenzo (a,h) anthracene (1,2,5,6-dibenzanthracene)
83. indeno (1,2,3-cd) pyrene (2,3-ophenylenepyrene)
84. pyrene
85. *tetrachloroethylene
86. *toluene
87. *trichloroethylene
88. *vinyl chloride (chloroethylene) **pesticides and metabolites**
89. *aldrin
90. *dieldrin
91. *chlordane (technical mixture & metabolites) *DDT and metabolites
92. 4,4′-DDT
93. 4,4′-DDE (p,p′-DDX)
94. 4,4′-DDD (p,p′-TDE) ***endosulfan and metabolites**
95. a-endosulfan-Alpha
96. b-endosulfan-Beta
97. endosulfan sulfate ***endrin and metabolites**
98. endrin
99. endrin aldehyde ***heptachlor and metabolites**
100. heptachlor
101. heptachlor epoxide ***hexachlorocyclohexane (all isomers)**
102. a-BHC-Alpha
103. b-BHC-Beta
104. r-BHC (lindane) -Gamma
105. g-BHC-Delta ***polychlorinated biphenyls (PCB's)**
106. PCB-1242 (Arochlor 1242)
107. PCB-1254 (Arochlor 1254)
108. PCB-1221 (Arochlor 1221)
109. PCB-1232 (Arochlor 1232)
110. PCB-1248 (Arochlor 1248)
111. PCB-1260 (Arochlor 1260)
112. PCB-1016 (Arochlor 1016)
113. *toxaphene
114. *antimony (total)
115. *arsenic (total)
116. *asbestos (fibrous)
117. *beryllium (total)
118. *cadmium (total)
119. *chromium (total)
120. *copper (total)
121. *cyanide (total)
122. *lead (total)
123. *mercury (total)
124. *nickel (total)
125. *selenium (total)
126. *silver (total)
127. *thallium (total)
128. *zinc (total)
129. **2,3,7,8-tetrachlorodibenzo-p-dioxin (TCDD)

*Specific compounds and chemical classes listed in the Natural Resource Defense Council (NRDC) consent decree and referenced in the Clean Water Act.
**This compound was specifically listed in the consent decree; however, due to its extreme toxicity, EPA recommends that laboratories not acquire an analytical standard for this compound.
Source: Refs. 29 and 30.

ject to *categorical standards* with an intention to restrict their discharge into the POTWs. *Prohibited standards* restrict the discharge of pollutants that may cause fire, explosion, corrosion, obstruct flow, or increase the temperature.[2,23,29,30]

3-4-3 Toxicity

On March 9, 1984, the U.S. EPA published a new national policy based on the 1993 Clean Water Act, prohibiting the discharge of toxic pollutants in toxic amounts into natural waters. As a result of these guidelines, toxicity tests have been instituted to protect human health and aquatic life from the toxic impacts caused by the release of contaminants into surface waters. The effluent may contain numerous toxic chemicals that may cause synergistic effects. For this reason, instead of specific chemical tests, biomonitoring was introduced. Biomonitoring requires appropriate bioassay organisms in an aquarium to determine the level of toxicity in the effluent. The tests are run in effluent samples with varying degrees of dilution, and records are kept on observations of death, deformities, reproduction, and growth of the test organisms. The tests may be short-term, intermediate, and/or long-term and static, recirculation, renewal, or flow-through. Common freshwater species are *Ceriodaphnia dubia* (water flea, daphnid shrimp), and *Pimephales promelas* (fathead minnow). The marine organisms included in the biomonitoring are *Champia parvula* (the red algae), *Mysidopsis bahia* (the mysid shrimp), *Menidia beryllina* (the inland silversides), and *Cyrinidon variegalus* (the sheephead minnow). The biomonitoring protocols have been published and commented on extensively in the literature.

The results of acute toxicity tests are reported in 48 or 96 hours. The LC_{50} is the concentration of effluent in dilution water that causes 50 percent mortality of the test organisms. The chronic toxicity is measured over a long period or generations. The results are based on mortality, reduced growth, or reproduction.[2,37-44] If a toxicity level is found and verified, toxicity reduction evaluation (TRE) is required. These studies involve toxicity investigation evaluation (TIE) and effluent toxicity treatability (ETT). The objective is to determine the best method to reduce or eliminate the toxicity of the effluent. The plan established by TIE or ETT is carried out under *corrective action.* The plan may include process modification, chemical change, construction, or industrial pretreatment. After reduction or elimination of toxicity, the plant is considered in compliance by EPA.

3-4-4 Microbiological Quality

The municipal wastewater contains microorganisms that play an important role in biological waste treatment. The principal groups of microorganisms of significance in wastewater treatment include bacteria, fungi, protozoa, and algae.

Basic Concepts. Microorganisms are grouped into three kingdoms: protista, plants, and animals. The members of the kingdom protista are called protists and include *procaryotes* (bacteria, blue-green algae) and *eucaryotes* (algae, fungi, and protozoa). These organisms are distinguished from the plant and animal kingdoms on the basis of a lack of differentiation of cells and tissues. Each cell contains nucleic acid [deoxyribonucleic acid (DNA)], which contains genetic information that is considered vital for the cell reproduction. In algae, protozoa, and fungi the nucleus is clearly defined (eucaryotic cells).

In bacteria and blue-green algae, the nucleus is poorly defined (procaryotic cells). Viruses fall between living and nonliving. They are not complete organisms, being made up of protein-protective coating surrounding a strand of nucleic acid.[5,45-48]

Microorganisms have different nutritional and environmental requirements such as carbon source, energy source, oxygen, temperature, and so on. Basic classifications based on these requirements are *autotrophic* organisms that derive their cell carbon from carbon dioxide, *heterotrophic* organisms that use organic carbon for cell synthesis, *phototrophs* that derive energy for cell synthesis from light, and *chemotrophs* that derive their energy from chemical reaction. Based on oxygen requirements, the microorganisms may be grouped into *aerobic, anaerobic,* and *facultative.* Aerobic microorganisms require the presence of molecular oxygen (O_2) for their metabolism. Anaerobic microorganisms grow in the absence of molecular oxygen. In fact, molecular oxygen is toxic to them. Facultative organisms can grow in the presence or absence of oxygen. The temperature plays an important role for growth and survival of the microorganisms. In accordance with the temperature range, the microorganisms may be classified as *psychrophilic* or *cryophilic, mesophilic,* and *thermophilic.* The optimum temperature ranges for these groups of organisms are 12–18°C, 25–40°C, and 55–65°C, respectively. The pH of water also influences the growth of organisms. Most organisms have an optimum pH in the range of 6.5 and 7.5.[2,5,44]

Because of the importance in wastewater treatment and water quality, bacteria, fungi, algae, protozoa, virus, and plants and animals are briefly discussed below.

Bacteria. Bacteria are simple-celled organisms that play a very important role in wastewater treatment. Most bacteria are harmless, many are actually beneficial as they consume organic matter, and some produce by-products that inhibit the growth of many pathogens. The pathogenic bacteria that are excreted by humans cause many diseases. Common diseases associated with pathogenic bacteria are gastroenteritis, typhoid and paratyphoid fever, dysentery, diarrhea, and cholera.

Most bacteria are grouped by form as spherical (cocci), rod-shaped (bacilli), curved rod-shaped (vibrios), spiral (spirilla), and filamentous. The size of cocci range from 1 to 3 μm, and bacilli range from 0.3 to 1.5 μm in width (or diameter) and from 1.0 to 10.0 μm in length.

Fungi. Fungi are aerobic, single to multicellular, nonphotosynthetic heterotrophic microorganisms. Most fungi and molds are saprophytes (obtain food from dead organic matter). Fungi, along with bacteria, play a principal role in waste treatment. Fungi can compete and perform better than the bacteria at lower pH, lower nutrients, and low moisture wastes.

Algae. Algae are simple organisms that are autotrophic and photosynthetic and contain chlorophyll. Many algae also contain different pigments and, therefore, may have various colors. Algae produce their own food from sunlight and nutrients. They play a significant role in waste stabilization ponds. Effluents containing high nutrients cause serious water-quality problems in natural waters. During algae blooms in reservoirs, algae cause turbidity and color, interfere with coagulation and sedimentation processes, and cause serious filter clogging. The chemicals produced by algae are *precursors* and are also associated with different taste and odor problems and production of undesired reaction by-products with chlorine.[2,9,48]

Protozoa. Protozoa are a group of unicellular, nonphotosynthetic, aerobic organisms. Several protozoa cause disease. *Giardia lamblia, Cryptosporidium,* and *Entamoeba histolytica* are transmitted by drinking water, while *Naegleria fowleri* may enter by nasal inhalation exposure from swimming in polluted water and causes amoebic meningoencephalitis. Protozoa feed upon bacteria in a wastewater treatment plant. Their presence is an indication of healthy operation.

Virus. Viruses are the smallest of infectious agents, some being as small as a single protein molecule. They are obligate parasites and, as such, require a host. Common virus-caused diseases are infectious hepatitis, gastroenteritis, and respiratory diseases. Viruses are more resistant to disinfection than bacteria, but in natural waters they are present in far fewer numbers than bacterial pathogens.

Plants and Animals. Plants and animals ranging in size from microscopic to larger are important in waste treatment and effluent quality control. Larger aquatic plants are used in natural treatment systems. These may include natural and constructed wetlands. This topic is covered in Chapter 24. Many smaller animals also play important roles in natural purification process. *Rotifers* and *Crustacea* are lower-order animals that prey on bacteria, protozoa, and algae and help to maintain a balance in population of primary producers and become a part of the food chain. Sludge worms such as tubifex and bloodworm, as well as other helminths and insect larvae, feed on sludge deposits and help to break down and solubilize the particulate organics. Parasitic worms (helminths) cause many diseases; common waterborne diseases are intestinal roundworm, *Guinea* worm, lung fluke, and *Schistostomiasis.* The transmission of worm and eggs occur due to contaminated drinking water or vegetables, and through body contact with polluted waters. Effective coagulation and filtration for turbidity control is a viable method of removing helminths from water supply.[5,9,10,19]

Indicator Organisms. In general, pathogens grow rapidly and multiply inside the human body, and tend to die off rapidly in nature. There are some pathogens that are hardy and persist. Some organisms form spores and are resistant to chlorination.

In order to determine the presence of pathogenic organisms in natural and treated water, the microbiologist must have a reliable measurement technique. Unfortunately, the analytical procedures for detection of pathogenic organisms are not clear cut. Hence, rather than look for the specific pathogens, there is a need to find a group of indicator organisms to measure the potential of a water to transmit diseases. The ideal indicator organism should have the following characteristics:[2,5,44,47]

1. Detection should be quick, simple, and reproducible.
2. Results should be applicable to all waters, that is, the numbers should correlate with the degree of pollution (higher numbers in sewage, less in polluted waters, and none in unpolluted waters).

3. The organism should have greater or equal survival time, and be present in larger numbers than the pathogens.
4. It should not grow in nature.
5. It should be harmless to man.

Naturally, no organism or group of organisms possess all these characteristics. However, the coliform organisms currently used as indicator organisms have many of these qualities. These are nonpathogenic bacteria whose origin is in fecal matter. The presence of these bacteria in water is an indication of fecal contamination and probably unsafe water. The coliform bacteria are gram-negative, non-spore-forming bacilli capable of fermenting lactose with the production of acids and gases. The coliform bacteria are members of the Enterobacteriacae family and include genera *Escherichia, Klebsiella, Citrobacter,* and *Enterobacter.* The *Escherichia coli (E. coli)* species appears to be most representative of fecal contamination. Fecal *Streptococci* and *Enterococci* are also used as indicator organisms under specific conditions.

The coliform organisms were originally believed to be entirely of fecal origin, but it has been shown that certain genera can grow in soil. Therefore, the presence of coliform organisms in surface water may be due to fecal wastes from human and animal sources and from soil erosion. To separate fecal coliforms *(E. coli)* from possible soil type, special tests are generally run. Frequently, for this purpose, fecal *Streptococci* are also used as indicator organisms, because they also originate from the intestinal tract of warm-blooded animals.[27]

The standard techniques used to enumerate the coliform organisms are (1) multiple tube fermentation and (2) the membrane filter. The multiple tube fermentation technique uses lactose broth media, which is fermented by coliform group. Gas bubbles collected inside an inverted inner vial is an indication of gas formation. The test may be carried out to presumptive, confirmed, or completed test levels. Series of dilutions in multiple tubes are used for statistically enumeration. The result is reported as most probable number per 100 mL (MPN/100 mL).[2,5,27] Standard MPN index tables of 95% confidence limits for a various number of tubes at dilutions of 10, 1.0, and 0.1 mL are provided in the *Standard Methods.*[27]

The membrane filter technique is used to quantify the coliform organisms present in the wastewater. This technique is faster and gives the actual number rather than the most probable number. Using a standard filtration apparatus, a dilute sample is filtered through a membrane filter having a rated pore diameter of 0.45 μm. The filter is placed over an absorbent pad containing Endo-type selective media and incubated. The colonies with a golden-green metallic sheen are counted. The procedure is given in the *Standard Methods.*[27] The number of specific indicator organisms found in untreated municipal wastewater is total coliform 10^5–10^8, fecal coliform 10^4–10^5, fecal *Streptococci* 10^3–10^4, and *Enterococci* 10^2–10^3 per mL. Generally, the ratio of fecal coliforms (FC) to fecal *Streptococci* (FS) in a sample can be used to show whether the suspected contamination derives from human or from animal wastes. As a general rule, the FC/FS ratio for domestic animals is less than 1.0, whereas the ratio for human beings is more than 4.0.[2,5]

3-5 CHARACTERIZATION OF WASTEWATER

Wastewater treatment facilities are normally designed for average loadings of BOD, suspended solids, and other constituents. Designs based on average conditions with no considerations for peak concentrations or average sustained concentrations may result in effluent quality that may often not meet the short-term, consecutive-day average limitations. Wastewater quality data also exhibit seasonal, daily, and hourly variations. A sampling program is often necessary to develop peak mass loading and average sustained loading conditions. Some excellent references on sampling, analysis, and data presentation are available in the literature.[2,5,27,32,36] The data should be used to develop seasonal, daily, and hourly variations; average concentrations; flow-weighted average; mass loadings; and sustained peak mass loadings. Typical ratios of one-day average sustained peak to average mass loading of BOD_5, suspended solids, total Kjeldhal nitrogen, ammonia nitrogen, and total phosphorus are, respectively, 2.5, 2.7, 2.0, 1.5, and 1.6.[2,5,23]

3-6 UNIT WASTE LOADINGS AND POPULATION EQUIVALENTS

Loadings of suspended and dissolved solids in municipal wastewater on a per capita basis remain relatively uniform. The variation in constituent loadings per capita per day may be due to industries served, usage of garbage grinders and domestic water softeners, and discharge of septage. In small treatment facilities their effects may be significant. Unit waste loadings in municipal wastewater may be developed from flow rate (liters per capita per day) and concentrations of various constituents in mg/L. Concentrations of many constituents are given in Table 3-10. Important unit waste loadings developed from Table 3-10 are summarized in Table 3-13.

TABLE 3-13 Unit Waste Loadings Derived from Table 3-10

Constituent	Typical Unit Waste Loadings (g/capita · d)
BOD_5	95[a]
COD	180
Total suspended solids (TSS)	104
Total nitrogen (as N)	18
Organic nitrogen (as N)	9
Ammonia nitrogen (as N)	9
Total phosphorus (as P)	4

[a]Average flow 450 L/c · d (118 gpcd) × (210 mg/L) × (1 g/1000 mg) = 94.5 g per capita per day. See Table 3-10 for typical concentrations of BOD_5, COD, TSS, various forms of nitrogen, and total phosphorus.

The population equivalent (P.E.) of a waste may be determined by dividing the total mass per day by the per capita mass loadings. Population equivalent has been used as a technique for determining industrial waste treatment costs. Population equivalents may be determined on the basis of flow, BOD_5, COD, TSS, P, N, etc. As an example, an industrial waste of 1000 m^3/d and having a BOD_5 of 500 mg/L would have a population equivalent in terms of $BOD_5 = 0.5$ g/L $\times$ 1/(95 g/capita $\cdot$ d) $\times$ 10^3 $m^3/d \times 1000$ L/m^3 = 5263 (95 g per capita per day is taken from Table 3-13).

3-7 PROBLEMS AND DISCUSSION TOPICS

3-1 Visit the water treatment plant in your community. Obtain water supply data for 1 year. Estimate the following:

(a) Annual average water demand, Lpcd

(b) Ratio of average water demand for the month of July or August with respect to the annual average water demand

(c) Ratio of average water demand for the month of January or February with respect to the average annual water demand

3-2 Obtain the flow data from the wastewater treatment plant in your community. Develop the following information.

(a) Determine average wastewater flow under dry weather flow condition, Lpcd.

(b) Prepare the typical diurnal flow pattern for a typical dry day and for a maximum wet day.

(c) Calculate the ratios of peak dry weather flow to average day.

(d) Calculate the ratio of peak wet weather flow to average day.

(e) Compare the peak dry weather flow with those obtained from Eqs. (3-1) and (3-2).

3-3 Check the average daily water demand and average daily wastewater flow lines obtained from the diurnal water demand and wastewater flow curve given in Figure 3-1. What percentage of the water is returned to the wastewater treatment facility?

3-4 A 200-home subdivision is proposed. Calculate the annual water saving that can be achieved if all homes are installed with faucet aerators, water efficient washing machines and dishwashers, and shallow trap water closets, instead of conventional plumbing. Assume average nonconserving water consumption is 380 Lpcd, and there are, at an average, 3.5 residents per home. Make use of the information provided in Table 3-2 and average savings given in Table 3-7.

3-5 A home has installed a pressure reducing valve (PRV) at the line supplying water into the home. If water usage is 350 Lpcd and there are four residents, calculate percent water saving. Typical breakdown of residential water uses is given in Table 3-2. Average savings from PRV is found in Table 3-7.

3-6 In a home total average water loss caused by toilet leaks, faucet drips, and leakage from the lawn sprinkler is approximately 4 percent of total water consumed. The water supply pressure is 345 kPa. A PRV is installed, which drops the supply pressure in the home to 110 kPa. Assuming water consumption is 304 Lpcd and all leaks behave like orifices, calculate the water saving that will be achieved from these sources after PRV is installed.

3-7 A wastewater treatment plant receives, at an average, 87 percent of water supply during dry weather conditions. Average municipal water demand expected is 340 Lpcd. Calculate average and maximum dry weather flows. Population of the town is 45,000.

3-8 A small subdivision of a city is being developed. The ultimate zoning plan shows the following residential, institutional, and commercial establishments. Single family population in low-income, medium-income, and high-income units are 500, 800, and 1500.

Apartment residents	500
One hotel/motel	500 units
One hospital	300 beds
One rest home	150 beds
One boarding school	1500 students
5 restaurants serving	1900 customers
One bar having	150 customers
Shopping centers	250 employees
Office buildings	1500 employees
Three barber shops	30 chairs
Two beauty salons	20 stations
One commercial laundry	20 machines
One service station	10 bays
One movie theater	500 seats

Calculate the residential water demand and institutional and commercial water demand. What is the average water demand for the entire community?

3-9 A city of 60,000 residents has an average water demand of 350 Lpcd. The institutional and commercial, and industrial average areas in the city are 200 and 300 ha, and water demand expected is 20 and 23 m^3 per ha per day. The public water use and water unaccounted for are 10 and 6 percent of total municipal water demand, respectively. Calculate total municipal demand and each component quantity as a percent of total municipal demand.

3-10 A municipal service area is 1650 ha. Average estimated infiltration/inflow allowance is 3500 L per ha per day. Calculate the I/I quantity per capita per day. Assume the population density of the service area is 25 persons per ha.

3-11 The total length of a sewerage system of a city is 200 km. The weighted equivalent diameter of the sewers is 21 cm. Calculate the I/I per capita per day that may reach the wastewater treatment plant if the I/I allowance is 1400 L per day per cm per km. Average sewerage length per capita is 8 m.

3-12 A primary sedimentation tank is designed for an average flow. The design detention time and overflow rates at average flow are 2 h and 37 $m^3/m^2 \cdot d$. Calculate the detention time and overflow rates under the peak flow conditions when ratio of peak to average flow is 2.7.

3-13 Calculate the carbonaceous BOD after 7 d at 30°C. Use Eqs. (3-5) through (3-7). BOD_5 at 20°C is 120 mg/L, and K at 20°C is 0.25/d.

3-14 Calculate total theoretical oxygen demand, total nitrogenous oxygen demand, and total carbonaceous oxygen demand of an industrial waste. The waste is represented by the chemical formula $C_6N_2H_6O_2$. Assume that nitrogen is converted to ammonia and then to nitrate. The average concentration of waste is 210 mg/L.

3-15 The ultimate oxygen demand (UOD) of a waste is expressed by the following equation:

$$\text{UOD (mg/L)} = A \times BOD_5 + B \times (NH_3 - N)$$

Calculate the factors A and B if the reaction rate K in Eq. (3-5) is 0.21 per day, and the ammonia nitrogen is fully converted to $NO_3 - N$.

3-16 An industry is discharging 2500 m^3/d wastewater into a sanitary sewer. The concentrations of BOD_5, TSS, TN, and TP are 200, 280, 35, 10 mg/L, respectively. Calculate the population equivalent of the industry based on flow, BOD_5, TSS, TN, and TP.

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4

Wastewater Treatment Unit Operations and Processes, and Process Diagrams

4-1 INTRODUCTION

Wastewater treatment units generally fall into two broad divisions: *unit operations* and *unit processes*. In the unit operations, the treatment or removal of contaminants is brought about by the physical forces. In the unit processes, however, the treatment occurs predominantly due to chemical and biological reactions. Often the terms "unit operations" and "unit processes" are used interchangeably because many processes are integrated combinations of operations serving a single primary purpose. As an example, activated sludge combines mixing, gas transfer, flocculation, and biological phenomena to remove biodegradable organics.

Wastewater treatment plants utilize a number of unit operations and processes to achieve the desired degree of treatment. The collective treatment schematic is called a flow scheme, process diagram, flow sheet, process trains, or flow schematic. Many different process trains can be developed from various unit operations and processes for the desired degree of treatment. However, the most desirable process train is the one that is most cost-effective. Many guidelines have been developed that may aid in evaluation and selection of process diagrams.[1-6] An example of cost-effectiveness analysis and selection of a process train is provided in Chapter 6. Basic considerations for developing a process train include (1) characteristics of wastewater and degree of treatment required; (2) requirement of the regulatory agencies; (3) proximity to the buildup areas; (4) topography and site conditions, land area available, and hydraulic requirement; (5) existing facilities; (6) available equipment; (7) preference and experience of the designer; (8) plant economics; (9) quantity and quality of sludge from each process; (10) level of expertise of treatment plant operation personnel; and (11) minimal environmental consequences and maximum environmental improvements.

Wastewater treatment facilities are designed to process the liquid and solid portions of the wastewater. In this chapter the unit operations and processes applicable to the treatment of liquid and solid portions are discussed in separate sections. Many treatment processes, however, are used to treat the liquid and solid portions of the wastewater simultaneously. Examples of such processes are natural systems such as stabilization ponds, aerated lagoons, land treatment, and wetlands. In these processes many chemical and biological forces act collectively to stabilize the liquid and solid components in the wastewater. These processes are discussed separately in Chapters 13 and 24.

4-2 LIQUID TREATMENT SYSTEMS

4-2-1 Pumping, Flow Measurement, and Flow Equalization

Liquid treatment may utilize pumping, flow measurement, and sometimes flow equalization. Although these systems do not provide any treatment, they are considered as a part of the overall process diagrams. A brief discussion of pumping station flow measurement and flow equalization is provided below.

Pumping Station. Treatment plants are normally located at a low point (near a river or a lake, for example) in order to provide gravity flow into the collection systems. At the plant site, the wastewater is pumped to an adequate height when topography dictates, at which time it will flow by gravity through the various treatment units. Pumping stations are also equipped with a wet well that intercepts incoming flows and tends to equalize pump loading. Pumping stations are usually located after bar screens and many times are located after grit removal, primary sedimentation, or even complete treatment. The objective is to remove the coarse solids, grit, and organic solids prior to pumping, because these solids often present operational difficulties at the pumping station. However, the cost of construction and operation of these units deep in the ground must be weighed against the cost of pumps that are designed to handle solids, with the treatment units above the ground. Chapter 9 discusses pumping station, pump selection, design criteria, and design procedure.

Flow Measurement. Flow measurement at wastewater treatment facilities is essential for plant operation, process control, and record keeping. The flow measurement devices may be located in the interceptor sewer, after the pumping station, or at any other location within a plant. Details on various flow measurement devices, their application, design criteria, and design example are given in Chapter 10.

Flow Equalization. Flow equalization is not a treatment process. It is simply the damping of the flow rate and mass-loading variations. Due to the cyclic nature of wastewater flows and organic strengths, many wastewater treatment units are designed for maximum conditions. By the use of flow equalization, the plants can be designed and operated under a nearly constant ideal flow and mass-loading condition, thus minimizing shock and achieving maximum utilization of the facilities. The flow equalization process consists of providing storage capacity and adequate aeration and mixing time to prevent odors and solids deposition. The flow equalization basins are in-line or off-line, and generally follow preliminary screening and grit removal to minimize mixing requirements. In in-line equalization, the entire flow is discharged into a completely mixed basin. Controlled flow pumps or gates are used to maintain a constant daily average flow through the plant. In an off-line system an overflow structure is built to bypass the excess flow into an equalization basin. Under low-flow conditions, the stored flow is then routed through the plant. In both systems, nearly constant flow rates are maintained through the plant. However, considerable damping of constituent mass loadings (such as BOD, COD, TSS, etc.) to the downstream processes is achieved with in-line equalization, but only slight damping is achieved with off-line equalization.

The volume required for flow equalization is determined by using an inflow mass diagram. Detailed analysis for the volume requirements and damping of flow rate and mass loadings may be found in several excellent references.[7-11]

4-2-2 Reactor Types and Process Considerations

Wastewater treatment is generally achieved in tanks or reactors in which physical, chemical, and biological changes occur. The principal types of reactors used for wastewater treatment are (1) batch reactor, (2) plug-flow or tubular-flow reactor, (3) complete-mixed or continuous-flow stirred tank reactor (CFSTR), and (4) arbitrary-flow reactor. Each of these reactors is briefly discussed below.

Batch Reactor. The reactor operates on fill and draw principal. No flow enters or leaves during operation period. Examples are BOD test, respirometry, and aeration study.

Plug-Flow or Tubular-Flow Reactor. In these reactors the particles leave the reactor in the same sequence as they enter. There is minimal longitudinal dispersion and particles retain their identity and remain in the tank for a time equal to the theoretical detention time. Such condition is generally encountered in a pipe flow or in a long-narrow channel (chlorine contact basin, or several CFSTRs connected in series). If a continuous or slug feed of nonreactive (conservative) dye tracer is applied into a reactor, the tracer profile at the effluent is shown in Figure 4-1(a). The theoretical detention time t_o is basin volume/flow.

Continuous-Flow Stirred Tank Reactor (CFSTR). In these reactors the entering particles are dispersed immediately throughout the tank. Complete mixing is generally achieved in round or square basins that are uniformly mixed. The continuous and slug

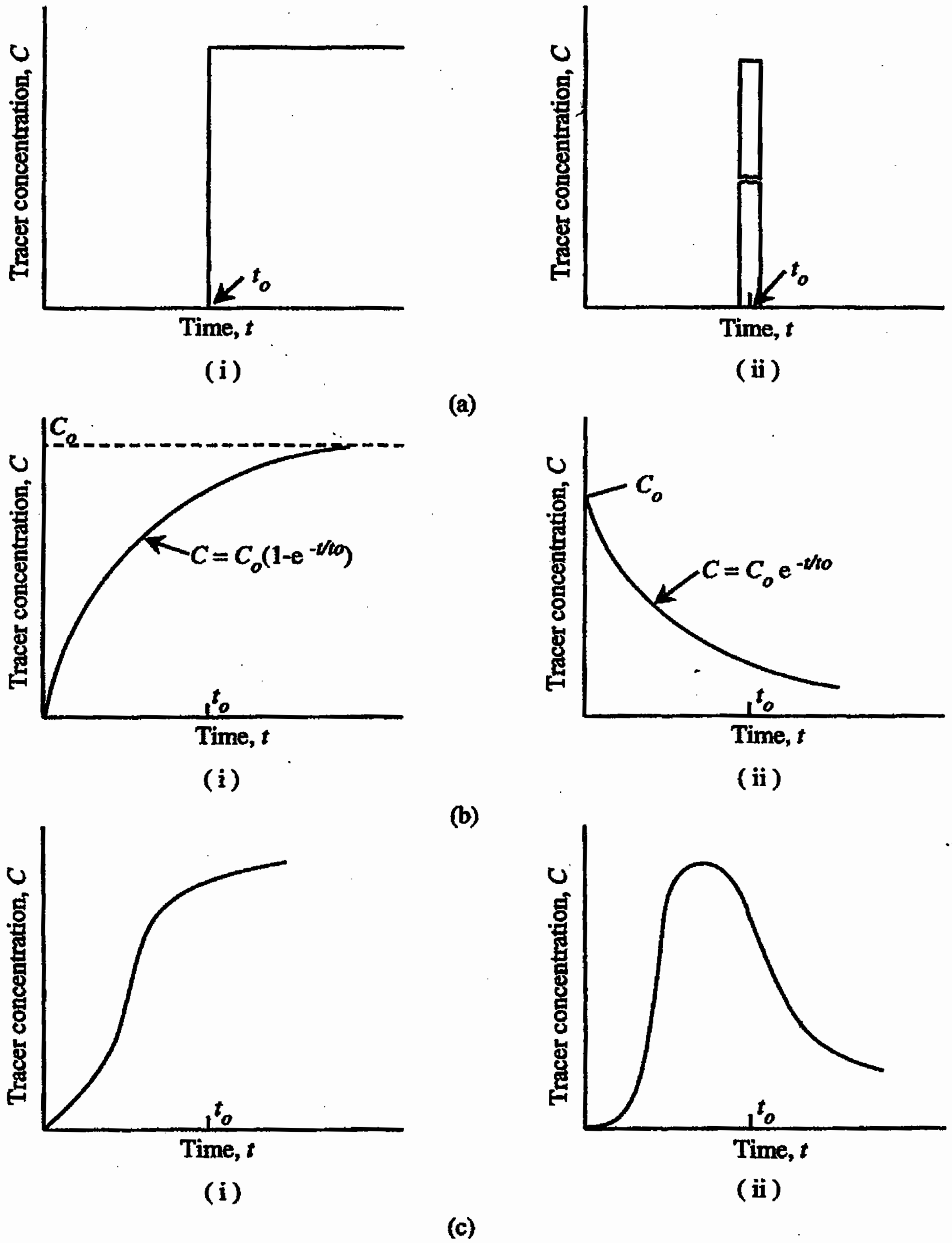


Figure 4-1 Dye Tracer Tests: (a) plug flow reactor, (i) continuous feed, (ii) slug feed; (b) completely mixed reactor, (i) continuous feed, (ii) slug feed (purged reactor); (c) arbitrary flow reactor, (i) continuous feed, (ii) slug feed.

feed tracer is applied into a reactor that produces a concentration of C_o; the tracer profile at the effluent end is shown in Figure 4-1(b). The tracer concentration at time t is C.

Arbitrary-Flow Reactor. In an arbitrary flow reactor, partial mixing between plug-flow and complete-mixed reactor is encountered. This type of mixing is generally occurs in aeration tanks and settling basin. The dye tracer profile is shown in Figure 16-1(c). Detailed discussion on this subject may be found in Refs. 7 and 8.

4-2-3 Unit Operations and Processes

Municipal wastewaters contain approximately 99.9 percent water. The small fraction of solids include organic and inorganic, suspended and dissolved matters. The removal of various contaminants from the liquid depends on the nature of the impurities and their concentrations. The coarse and settleable inorganic and organic solids are generally removed in primary treatment units that include unit operations such as bar screens, grit removal, and sedimentation facilities. The removal of dissolved organics is readily achieved in biological or chemical treatment processes that may be added to the primary treatment. The combined system is called a secondary treatment plant.[a] Many unit operations and processes may be added to the existing primary or secondary treatment systems to achieve the removal of nutrients and other contaminants. This is called tertiary treatment or advanced wastewater treatment.

A summary of different unit operations and processes considered in the design of primary, secondary, and advanced wastewater treatment facilities is provided in Table 4-1.[12-18] Figure 4-2 illustrates the components of these treatment units. Treatment levels achieved across these units and unit combinations are summarized in Table 4-2. As wastewater is processed at different treatment units, a concentrated waste stream containing various constituents is removed. Table 4-3 provides the characteristics of waste streams indicated in Figure 4-2.

4-2-4 Combination of Unit Operations and Processes into Process Diagrams

Many unit operations and processes described in Table 4-1 and Figure 4-1 can be combined to develop a process diagram to achieve a desired level of treatment. The level of treatment may range for removal of BOD_5 and TSS, nitrogen, and phosphorus, to complete demineralization. A number of wastewater treatment process combinations and process diagrams and resultant effluent quality are summarized in Table 4-4. Figure 4-3 illustrates several process flow diagrams.

To develop the best possible process diagram, designers must evaluate many factors that are related to operation and maintenance, process efficiency under variable flow conditions, and environmental constraints.[12,19] In Table 4-5 various factors that are considered important in the selection of process diagrams are evaluated.

Text continued on page 86.

[a]The secondary treatment as defined by the U.S. Environmental Protection Agency is directed principally toward the removal of BOD_5 and total suspended solids (see Sec. 2-6-2).

TABLE 4-1 Major Physical, Chemical, and Biological Treatment Unit Operations and Processes Used for Liquid Treatment

Unit No.	Unit Operations and Processes[a]	Principal Application	Ref.
A	Screening or comminution (UO)	Racks or bar screens are the first step in wastewater treatment. They are used to remove large objects. Comminution may also be provided for grinding of screens and other coarse solids.	Chap. 8
B	Grit removal (UO)	Grit removal facility is used to remove heavy material such as sand, gravel, cinder, eggshell, and the like.	Chap. 11
C	Primary sedimentation (UO)	The purpose of primary sedimentation facility is to remove settleable organic solids.	Chap. 12
D	Suspended growth biological reactor (UP)	The process is used to remove dissolved organics. Air is supplied for oxygen gas transfer into the liquid phase. Principal variation is an activated sludge process.	Chap. 13
E	Attached growth aerobic biological reactor (UP)	The process is used to remove dissolved organics. Principal variation is a trickling filter.	Chap. 13
F	Suspended and attached growth anaerobic biological reactor (UP).	These processes are not used for treatment of municipal wastewaters because of low organic content. Industrial wastes with high organic strength are best suited. Common variations of suspended growth reactors are anaerobic contact process and upflow anaerobic sludge blanket process (UASB). Common variations of attached growth anaerobic process are anaerobic filters, and expanded bed anaerobic filters. Because these processes are not used for POTWs, the stations across the unit, and treatment efficiency are not included in Figure 4-2 and Table 4-2.	Chap. 13
G	Sequencing batch reactor (UP)	This is a fill and draw activated sludge treatment system. The filling, decanting and refilling operations are arranged so that anaerobic, anoxic, and aerobic conditions develop for biological phosphorus, and nitrogen removal along with BOD removal.	Chap. 13

H	Combined anaerobic, anoxic and aerobic process with suspended, or attached growth reactors (UP)	The process uses multiple reactors and return and recirculation lines. The process has capabilities to enhance biological phosphorus and nitrogen removal.	Chap. 13
I	Secondary clarifier (UO)	Secondary or final clarifier is the sedimentation facility used in conjunction with a biological or chemical treatment process.	Chap. 13
J	Disinfection (UP)	Disinfection facilities are used to reduce the number of water borne pathogens in the effluent. Chlorination is the most common method. Removals across the unit are insignificant.	Chap. 14
K	Dechlorination (UP)	Chlorine being toxic to aquatic life, the residuals must be removed before the effluent is discharged into the aquatic environment. Many reducing agents are added to achieve dechlorination. Removals across the unit are insignificant.	Chap. 14
L	Coagulation and chemical precipitation (UP)	Coagulation involves chemical addition, mixing, precipitation and flocculation. The process is used for precipitation of suspended solids, BOD, and phosphorus. Commonly used chemicals are alum, iron salts, and polymers. The process uses flash mix and flocculation basins.	Chaps. 12, 24
M	Single stage lime precipitation (UP)	Lime is used for precipitation of suspended solids, BOD, and phosphorus. The process is similar to coagulation.	Chaps. 12, 24
N	Two stage lime precipitation (UP)	Excess lime is used in two stages. Suspended solids, BOD, and phosphorus are precipitated. Two clarifiers I1 and I2 are generally needed (Fig. 4-2).	Chap. 24
O1 O2	Nitrification (UP)	Process is used to convert ammonia to nitrate. It is achieved in suspended or attached growth biological reactor. Nitrification can be achieved in a single stage aerobic reactor with carbonaceous BOD removal (O1) or it may be achieved in a separate stage aerobic reactor after BOD removal (O2).	Chaps. 13, 24
P	Denitrification (UP)	Nitrite and nitrate are reduced to nitrogen gas by microorganisms. Denitrification is achieved under anaerobic condition in suspended or attached growth reactors. An organic source such as methanol is needed.	Chaps. 13, 24

Continued

TABLE 4-1 Major Physical, Chemical, and Biological Treatment Unit Operations and Processes Used for Liquid Treatment—cont'd

Unit No.	Unit Operations and Processes[a]	Principal Application	Ref.
Q	Volatalization and ammonia stripping (UO)	Ammonia gas is air stripped from the wastewater in a stripping tower. Other volatile organic compounds are also removed by volatilization and gas stripping.	Chap. 24
R	Breakpoint chlorination (UP)	Ammonia nitrogen is oxidized to nitrogen gas by breakpoint chlorination in a mixing basin.	Chap. 24
S	Natural systems (UP)	Many natural systems such as pond processes, land treatment systems, and natural and constructed wetlands are considered for complete treatment, or polishing of final effluent from secondary treatment plants.	Chap. 24
T	Filtration, microstraining (UO)	Filtration is used to polish the effluent. Total suspended solids and turbidity are removed. Microstrainers are also used for the same purpose, in particular to remove algae from stabilization pond effluents.	Chap. 24
U	Carbon adsorption (UP)	Carbon adsorption is used to remove soluble refractory organics from wastewater effluent.	Chap. 24
V	Ion exchange (UP) (ammonia nitrogen)	Ion exchange is used to demineralized the wastewater effluent. It is also used to selectively remove ammonia in a bed of clinoptilolite (a zeolite resin).	Chap. 24
W	Reverse osmosis or ultrafiltration (UO)	It is a demineralization process applicable to production of high quality water from the effluent. The water is permeated through semipermeable membrane at high pressure.	Chap. 24
X	Electrodialysis (UO)	It is a demineralization process. Electrical potential is used to transfer the ions through ion selective membranes.	Chap. 24

[a]UO = unit operation; UP = unit process.
Note: Pumping stations, flow metering, and flow equalization may be a part of the overall process diagram. These systems are not included in Table 4-1 as they do not provide any treatment.
Source: Adapted in part from Refs. 7 and 12–18.

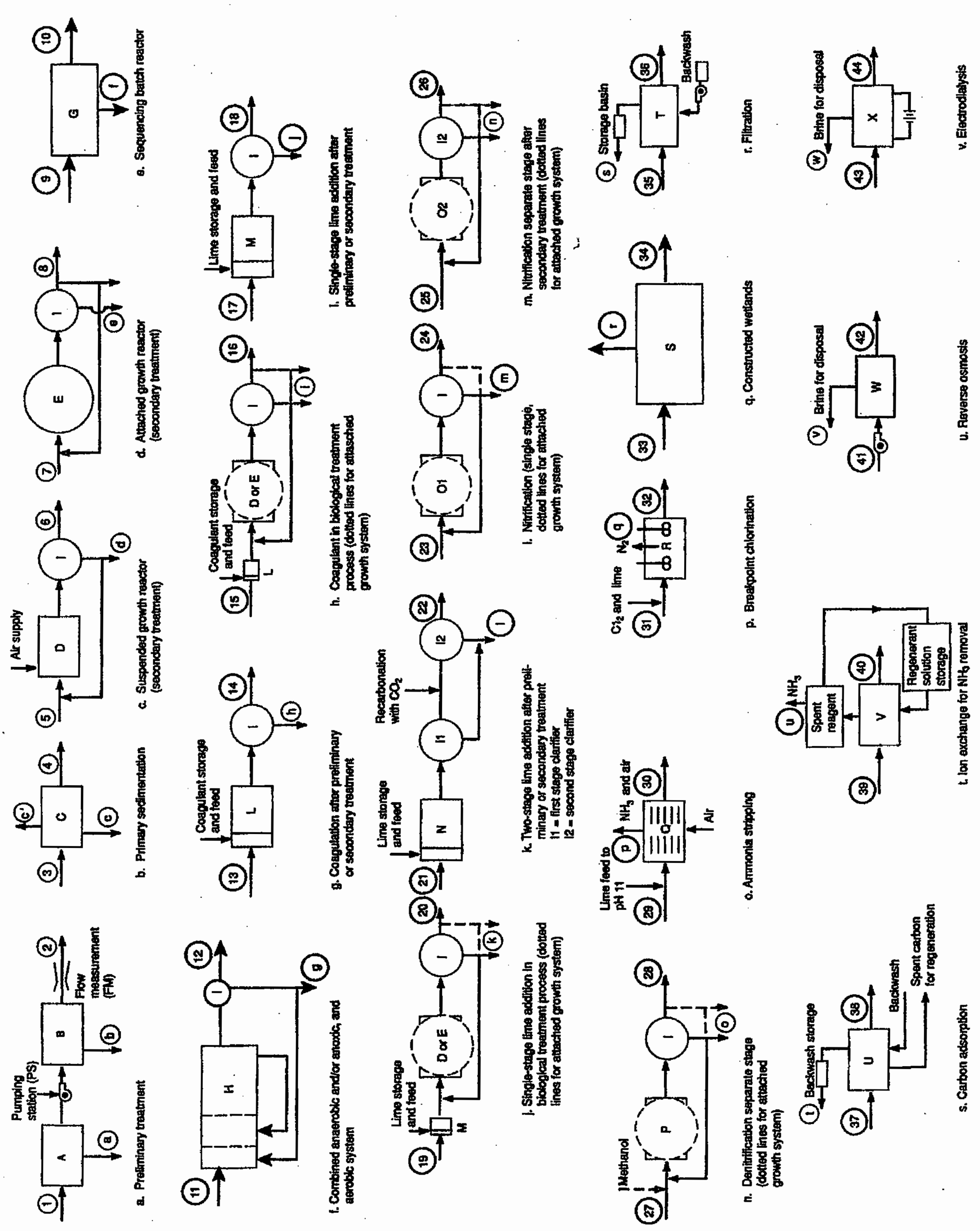


Figure 4-2 Unit Operations and Unit Processes Used for Liquid Treatment at Primary, Secondary, and Advanced Wastewater Treatment Facilities (see Table 4-1 for definition of unit operations and processes, Table 4-2 for removal efficiencies across the units, and Table 4-3 for characteristics of waste streams).

TABLE 4-2 Degree of Treatment Achieved by Various Unit Operations and Processes

Treatment Units or Combinations	Treatment Units Involved[a]	Stations across the Units[b]	Removal Efficiency (Percent) across the Units[c]					
			BOD_5	COD	TSS	TP	ON	NH_3-N
a. Preliminary treatment (PT)	A, PS, B, FM	1–2	small[d]	small[d]	small[d]	small[d]	small[d]	small[d]
b. Primary sedimentation	C	3–4	30–40	30–40	50–65	10–20	20–40	0
c. Activated sludge (conventional)	D, I	5–6	80–85	80–85	80–90	10–25	60–85	8–15
d. Trickling filter (high rate)	E, I	7–8	60–80	60–80	60–85	8–12	60–80	8–15
e. Sequencing batch reactor (SBR)	G	9–10	85–95	85–90	80–95	60–90	70–95	90–95[e]
f. Combined anaerobic, anoxic, and aerobic system	H, I	11–12	90–95	85–90	80–95	70–90	70–95	90–95[e]
g. Coagulation and sedimentation after preliminary or secondary treatment	L, I	13–14	40–70	40–70	50–80	70–90	50–90	0
h. Coagulation in biological treatment process	L, D or E, I	15–16	80–90	80–90	70–90	75–85	60–90	0
i. Single-stage lime addition after preliminary treatment or after secondary treatment	M, I	17–18	50–70	50–70	60–80	70–90	60–90	0
j. Single-stage lime addition in biological treatment	M, D or E, I	19–20	80–90	80–90	70–80	75–85	60–90	0
k. Two-stage lime addition after preliminary treatment or after biological treatment	N, I1, I2	21–22	50–85	50–85	50–90	85–95	70–90	0

l. Nitrification single stage with carbonaceous BOD removal	O1, I	23–24	80–95	80–90	70–90	10–15	75–85	85–95[e]
m. Nitrification separate stages, suspended or attached growth	O2, I2	25–26	50–70	50–60	small[f]	0	40–50	90–96[e]
n. Denitrification separate stages, suspended or attached growth	P, I	27–28	small[f]	small[f]	small[f]	5–10	5–10	small[g]
o. Ammonia stripping	Q	29–30	0	0	0	0	0	60–95
p. Breakpoint chlorination	R	31–32	—	—	0	0	—	80–90
q. Constructed wetlands for effluent polishing	S	33–34	50–70	40–60	50–80	60–90	60–90	85–95
r. Filtration	T	35–36	20–60	0–50	60–80	20–50	50–70	0
s. Carbon adsorption	U	37–38	50–85	50–85	50–80	10–30	30–50	0
t. Ion exchange for ammonia nitrogen removal	V	39–40	small	small	small	small	small	90–95[h]
u. Reverse osmosis	W	41–42	90–100	90–100	90–100	90–100	90–100	60–90
v. Electrodialysis	X	43–44	20–60	20–60	90–100	90–100	80–95	30–50

[a]For explanation of symbols see Table 4-1 and Figure 4-2; PS = pump station, FM = flow measurement
[b]For station numbering see Figure 4-2. Station numbers across units F, I, J, and K and treatment efficiencies are not given (see Table 4-1 for definition)
[c]For definition of terms see Table 3-10.
[d]BOD_5 or COD removal may vary if communitor, and/or grit washing is used. With no communitor and no grit washing, BOD_5 removal may be 0–5 percent and TSS removal 5–10 percent.
[e]Nitrate nitrogen (NO_3-N) concentration may reach 4–6 mg/L (as N) in the effluent from processes G and H, 8–10 mg/L (as N) from processes D and E, and 15–28 mg/L (as N) with processes O1 and O2.
[f]Removal is normally small and is considered zero. BOD_5 and COD may increase if a carbon source is added.
[g]Ammonia-N removal is small but NO_3-N removal may reach 90–95 percent.
[h]For selective ammonia removal. BOD_5, COD, TSS, TP, and ON removals are small. NH_3 removal is 90–95 percent.
Note: BOD_5, COD, TSS, TP, ON, and (NH_3-N) removal in disinfection by chlorination and dechlorination or UV system are considered insignificant.
Source: Adapted in part from Refs. 7 and 12–18.

TABLE 4-3 Characteristics of Waste Streams from Different Unit Operations and Processes Given in Table 4-1 and Figure 4-1

Waste Stream Identification[a]	Treatment Units Involved[b]	Description	Constituents and Processing	Refs.
a	A	Screening	Coarse solids are often communited and returned to wastewater stream. Normally screenings are disposed of by landfilling. Quantity 20.0×10^{-3} $m^3/10^3$ m^3.	Chap. 8
b	B	Grit	Heavy inorganic solids. Quantity 30.0×10^{-3} $m^3/10^3$ m^3. Often washed prior to disposal by landfilling.	Chap. 11
c	C	Primary sludge	Gray and slimy material has offensive odors. Quantity 105–165 g/m^3. Solids content 3.0–8.0 percent.	Chap. 12
c′	C	Scum	Consists of floatable materials such as oil, grease, fats, waxes, etc. These are quite odorous. Quantity 8 g/m^3.	Chap. 12, 16
d	D, I	Biological sludge (secondary or waste-activated sludge)	Biological solids. Quantity 60–100 g/m^3. Solids content 0.3–2.0 percent. Often returned to primary sedimentation basin for concentration.	Chap. 13
e	E, I	Biological sludge (trickling filter sludge)	Biological solids. Quantity 50–90 g/m^3. Solids content 3 percent.	Chap. 13
f	G	Biological sludge (secondary or waste activated sludge)	Biological solids quantity 60–100 g/m^3. Solids content 0.3–1.0 percent.	Chap. 13
g	H, I	Biological sludge	Biological solids. Quantity 60–100 g/m^3. Solids content 0.3–2.0 percent.	Chap. 13
h	L, I	Chemical precipitation sludge (metal hydroxide sludge)	Sludge is slimy or gelatinous. Quantity 80–300 g/m^3. Solids content 3–4 percent.	Chap. 24
i	L, (D or E), I	Chemical-biological sludge	Solids production 100–150 g/m^3 at 0.8 percent solids from activated sludge and 50–80 g/m^3 at 2 percent solids from [illegible]	Chap. 24

j	M, I	Single-stage lime sludge	Sludge is somewhat slimy or gelatinous. Quantity 500–600 g/m^3. Solids content 2–5 percent.	Chap. 24
k	M, (D or E), I	Lime-biological sludge	Solids production 200–300 g/m^3 at 1 percent solids from activated sludge, and 150–200 g/m^3 at 3 percent solids from trickling filter.	Chap. 24
l	N, I1, I2	Two-stage lime sludge	Sludge is slimy and gelatinous. Quantity 900 g/m^3. Solids content 4–5 percent.	Chap. 13
m	O1, I	Single-stage nitrification	Solids production 70–100 g/m^3 at 0.8 percent solids from activated sludge and 40–70 g/m^3 at 3 percent solids from trickling filter.	Chap. 24
n	O2, I2	Separate-stage nitrification	Solids production 10–12 g/m^3. Solids content 0.8–3 percent.	Chap. 24
o	P, I	Separate-stage denitrification	Solids production 10–20 g/m^3. Solids content 0.8–2 percent.	Chap. 24
p	Q	Gaseous waste stream containing NH_3 and VOCs	NH_3 and VOCs. Direct discharge may not be allowed. Scrubbing may be needed to remove contaminants and air recycle.	Chap. 24
q	R	Gaseous waste streams containing nitrogen and other VOCs	The treated effluent will contain undesirable by-products of chlorinated organic compounds.	Chap. 24
r	S	Harvested algae and other aquatic plants	The quantity will vary depending upon the process and harvesting need.	Chap. 24
s	T	Filter backwash	Liquid waste containing suspended solids is produced. This waste stream is normally returned to the head of the plant.	Chap. 24
t	U	Activated carbon backwash	Liquid waste containing suspended solids. This waste stream is normally returned to the head of the plant.	Chap. 24
u	V	Brine from ion exchange system	Contains NH_3 and other salts. Ammonia removal from spent reagent will be needed for reuse of chemicals.	Chap. 24
v	W	Brine from reverse osmosis system	Dissolved salts require special treatment and disposal.	Chap. 24
w	X	Brine from electrodialysis system	Dissolved salts require special treatment and disposal.	Chap. 24

[a]For explanation of waste stream symbols see Figure 4-2.
[b]For explanation of process symbols see Table 4-1 and Figure 4-2.
Source: Adapted in part from Refs. 7 and 12–18.

TABLE 4-4 Treatment Process Combinations and Process Trains

Process Train No.	Type of Treatment	Treatment Units and Combinations	Effluent Quality (mg/L)								Comments
			BOD_5	COD	UOD[a]	TSS	TP	ON	NH_3-N	NO_3-N	
(1)	Raw wastewater or preliminary treatment[b]	A + PS + B + FM	210	400	405	230	11	25	20	0	No treatment or insignificant treatment. The effect of side streams on influent quality is not considered.
(2)	Primary treatment	A + B + C + (J + K)	130	265	285	100	9	21	20	0	Not acceptable effluent quality.
(3)	Activated sludge	A + B + C + (D + I) + (J + K)	20	45	111	20	8	2	16	4	Meets secondary effluent quality.
(4)	Trickling filter	A + B + C + (E + I) + (J + K)	30	60	126	30	8	3	16	4	Marginal secondary effluent quality.
(5)	Activated sludge with filtration and disinfection	A + B + C + (D + I) + T + (J + K)	10	20	96	10	6	2	14	6	Effluent quality better than secondary effluent. Small nutrient removal.
(6)	Activated sludge/ nitrification in single stage	A + B + C + (O1 + I) + (J + K)	10	35	20	20	8	2	1	18	Effluent quality better than secondary effluent. No nutrient removal. Highly nitrified effluent.

(7)	Activated sludge with nitrification/denitrification separate stages	A + B + C + (O1 + I) + O2 + I + (P + I) + (J + K)	10	25	20	20	8	1	1	1	Effluent quality better than secondary effluent. Good nitrogen removal and well-stabilized effluent.
(8)	Sequencing batch reactor	A + B + C + G + (J + K)	10	25	20	10	2	2	1	8	Effluent quality is better than secondary effluent. Phosphorus and nitrogen removal, and complete nitrification of ammonia nitrogen.
(9)	Combined anaerobic, anoxic, and aerobic process with suspended growth reactor.	A + B + C + (H + I) + (J + K)	10	25	20	10	2	2	1	8	Effluent quality better than secondary effluent. Phosphorus and nitrogen removal, and complete nitrification of ammonia nitrogen.
(10)	Coagulation or lime precipitation, filtration, and carbon adsorption	A + B + (L or M + I) + T + U + (J + K)	10	30	105	10	1	2	20	0	Effluent quality better than secondary effluent. No ammonia removal. Excellent phosphorus removal. pH may be high if lime is used. Disinfection may not be required with lime.

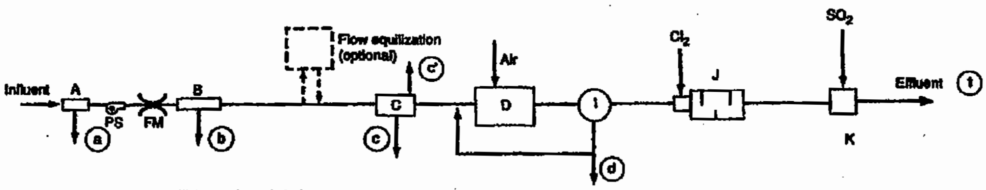
Continued

TABLE 4-4 Treatment Process Combinations and Process Trains—cont'd

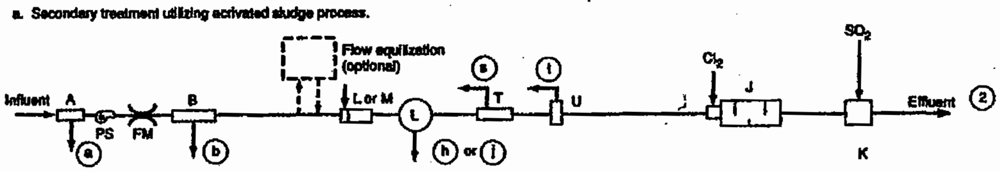
Process Train No.	Type of Treatment	Treatment Units and Combinations	Effluent Quality (mg/L)								Comments
			BOD_5	COD	UOD[a]	TSS	TP	ON	NH_3-N	NO_3-N	
(11)	Two-stage lime precipitation, filtration, and carbon adsorption	A + B + (N + I1 + I2) + T + U	5	20	98	5	1	1	20	0	Effluent quality better than secondary effluent. No ammonia removal. Excellent phosphorus removal. Effluent may require neutralization. Disinfection may not be required with high lime doses.
(12)	Lime or coagulant addition in activated sludge	A + B + C + (L or M + D + I) + (J + K)	15	35	99	15	2	1	17	0	Effluent quality better than secondary effluent. Good phosphorus removal.
(13)	Lime or coagulant addition in primary followed by activated sludge/nitrification (single stage)/filtration/carbon adsorption	A + B + (L or M + I) + (O1 + I) + T + U + (J + K)	5	10	17	5	1	1	2	18	Effluent quality better than secondary effluent. Excellent phosphorus removal and well-nitrified effluent.

(14)	Lime or coagulant addition in activated sludge followed by separate stage nitrification/ denitrification/ filtration	A + B + C + (L or M + D + I) + (O2 + I2) + (P + I) + T + (J + K)	5	20	12	5	2	1	1	1	Advanced waste treatment. Effluent low in BOD_5, TSS, and nutrients. Effluent reused.
(15)	Two-stage lime precipitation/ ammonia stripping/ filtration/carbon adsorption	A + B + (N + I1 + Q + I2) + T + U	5	25	12	5	1	1	1	0	Advanced waste treatment. Effluent low in BOD_5, TSS, and nutrients. Effluent may require pH adjustment. Effluent reused.
(16)	Activated sludge/ filtration/reverse osmosis	A + B + C + (D + I) + T + W	small	small	small	small	small	small	small	small	Demineralization of wastewater. Effluent reused.

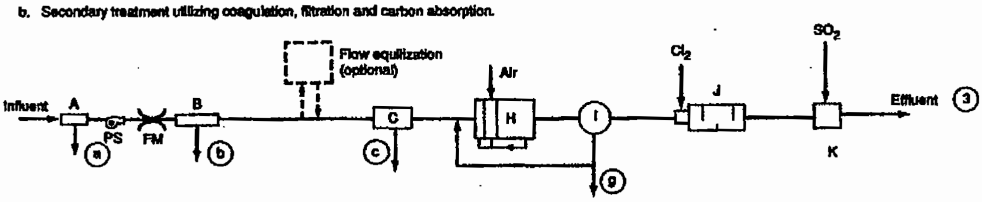
[a]Ultimate oxygen demand, UOD = 1.5 BOD_5 + 4.5 (NH_3-N) [see also Eq. (3-4)].
[b]The effluent quality from the preliminary treatment facility is assumed same as the influent quality at the head of the plant. The effect of mixing the side streams from the sludge-processing areas is not considered in these estimates (see also Figure 4-3).



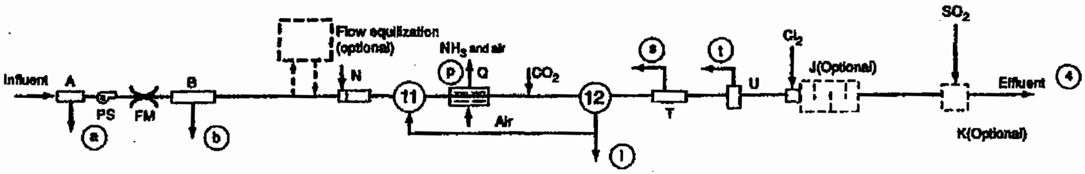
a. Secondary treatment utilizing activated sludge process.



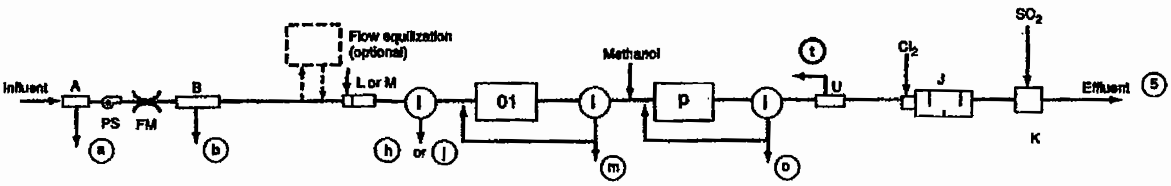
b. Secondary treatment utilizing coagulation, filtration and carbon absorption.



c. Biological nutrient removal, anaerobic, anoxic and aerobic treatment.



d. Advanced waste treatment utilizing single stage lime precipitation, ammonia stripping, filtration and carbon adsorption.



e. Advanced waste treatment utilizing coagulation, single stage activated-sludge-nitrification, denitrification, and filtration.

Parameter mg/L	Influent*	Effluent points				
		1	2	3	4	5
BOD_5	210	20	10	10	5	5
COD	400	45	30	25	25	20
UOD	405	111	105	20	12	12
TSS	230	20	10	10	5	5
TP	11	8	1	2	1	1
ON	25	2	2	2	1	1
NH_3 -N	20	16	20	1	1	1
NO_3	0	4	0	8	0	0

*The influent quality is assumed after mixing with the side streams from the sludge processing areas.

Figure 4-3 Process Flow Diagrams of Wastewater Treatment Plants (see Figure 4-1 and Tables 4-1, 4-2, 4-3, and 4-4 for the description of treatment units, unit combination, and efficiencies).

TABLE 4-5 Important Factors Other Than Costs That Are Also Considered in Selection of Wastewater Treatment Unit Operations and Processes

Treatment Units and Combinations	Land Requirements[a]	Adverse Climatic Conditions	Ability to Handle Flow Variations	Ability to Handle Influent Quality Variation	Industrial Pollutants Affecting Process	Reliability of the Process	Ease of Operation & Maintenance	Occupational Hazards[b]	Air Pollution	Waste Products
Preliminary treatment (A + B)	Min.	—	Good	Good	Min.	Good	Fair		Odors	Screenings and grit
Pumping (PS)	Min.	Freezing of valves	Good	Good	Min.	Good	Fair	Equipment	Odors	None
Primary sedimentation (C)	Mod.	—	Fair	Good	Min.	Good	Good		Odors	Sludge
Coagulation and sedimentation (L + I) or (M + I)	Min.	Freezing of chemical lines	Good	Very good	Mod.	Good	Good	Chemicals	Odors	Sludge
Trickling filter (E + I)	Max.	Freezing of piping and valves	Good	Fair	Mod.	Good	Good	Equipment	Odors	Sludge
Activated sludge, conventional (D + I)	Mod.	—	Fair	Good	Mod.	Good	Fair	Equipment	VOCs	Sludge

Continued

TABLE 4-5 Important Factors Other Than Costs That Are Also Considered in Selection of Wastewater Treatment Unit Operations and Processes—cont'd

Treatment Units and Combinations	Land Requirements[a]	Adverse Climatic Conditions	Ability to Handle Flow Variations	Ability to Handle Influent Quality Variation	Industrial Pollutants Affecting Process	Reliability of the Process	Ease of Operation & Maintenance	Occupational Hazards[b]	Air Pollution	Waste Products
Activated sludge with chemicals (L or M) + (D + I)	Min.	Freezing of chemical lines	Good	Good	Max.	Good	Good	Chemicals	VOCs	Sludge
Sequencing batch reactor (G)	Min.	—	Poor	Good	Mod.	Fair	Poor	Equipment	VOCs	Sludge
Combined anaerobic, anoxic, and aerobic system (H + I)	Min.	—	Fair	Good	Mod.	Good	Good	Equipment	VOCs	Sludge
Filtration, Microstraining (T)	Mod.	—	Good	Poor[c]	Min.	Fair	Fair	—	None	Backwash waste
Activated carbon (U)	Mod.	—	Good	Poor[c]	Max.	Good	Fair	Fires, explosion	Regenerant gas	Spent carbon
Two-stage lime treatment (N + I1 + I2)	Max.	—	Good	Good	Min.	Good	Fair	Chemicals	None	Excess sludge

Biological nitrification (O1 + I) or (O2 + I2)	Max.	Efficiency decreases in cold climate	Fair	Poor	Mod.	Fair	Fair	—	VOCs	Sludge
Biological denitrification (P + I)	Max.	Efficiency decreases in cold climate	Fair	Fair	Mod.	Fair	Fair	Chemical	Odors	Sludge
Ion exchange (V)	Min.	—	Poor	Fair	Max.	Good	Good	Chemicals	Ammonia	Waste regenerant
Ammonia stripping (Q)	Mod.	Freezing	Fair	Fair	Min.	Good	Fair	—	Ammonia, VOCs	Ammonia VOCs
Disinfection (J) (chlorine)	Min.	—	Good	Fair	Mod.	Good	Good	Chemicals, Explosion	Chlorine odor	Adds solids in effluent
Dechlorination (K)	Min.	—	Good	Fair	Mod.	Good	Good	Chemicals, Explosion	Pungent odor of SO_2	Add solids in effluent

[a]Min. = minimum, Mod. = moderate, Max. = maximum.
[b]Occupational hazards due to mechanical equipment, chemicals, compressed gases, fumes, and like.
[c]Influent quality in particular TSS and foam affects the performance seriously.
Source: Ref. 5.

4-3 SLUDGE PROCESSING AND DISPOSAL

Safe handling and disposal of various residues produced in different treatment units are of equal importance. The solids portion includes screenings, grit, scum, and primary and secondary sludge. The screenings and grit are generally disposed of by landfilling. The sludge (including scum), which may contain solids in concentrations of 0.5–5 percent, offers complex processing and disposal problems. It is odorous and contains large volumes of water. Because the treatment and disposal of sludge is expensive, sludge-handling costs are often the overriding consideration in the design of wastewater treatment plants.[20,21]

4-3-1 Unit Operations and Processes

In general, the sludge-processing and disposal methods include thickening, stabilization, dewatering, and disposal. Many unit operations and processes are utilized at various stages of sludge processing and disposal. To develop a cost-effective system of sludge treatment, the best combination of treatment processes must be chosen. Many of these unit operations and unit processes are illustrated in Figure 4-4.[7,13] A summary of different sludge treatment systems commonly used in the overall sludge processing train is provided in Table 4-6.[7,12-18,20,21]

4-3-2 Performance Data

Most of the sludge-processing facilities produce two streams: (1) processed solids and (2) liquid streams. The liquid streams (also called side streams) must be treated again, and these liquids from various sludge-processing units are returned to the head of the plant. The returned flows often contain high concentrations of suspended solids and BOD. Often, equalization facilities are provided to distribute the hydraulic and material loading over 24 hours of operation.[20] To predict the incremental loadings due to returned flow, material mass balance at average design flow must be performed to determine the final loadings to the plant. The procedure for material mass balance analyses is shown in Chapter 13. Table 4-7 provides side stream quality and operational performance data of various sludge-processing facilities.[20,21] This information is used to develop the material mass-balance relationships in Chapter 13.

4-3-3 Combination of Unit Operations and Processes into Process Diagrams

Proper selection of the sludge-processing equipment is important for trouble-free operation of a wastewater treatment facility. Sludge is quite odorous and may cause serious environmental problems. Therefore, such factors as solids captured, chemical quality of returned flows, ability to handle variable quality of sludge, ease of operation, and odors are often given serious consideration. Some of these factors that must be considered in final selection of appropriate process diagrams are assessed qualitatively in Table 4-8.[5] Several process diagrams for sludge processing are developed in Figure 4-5.

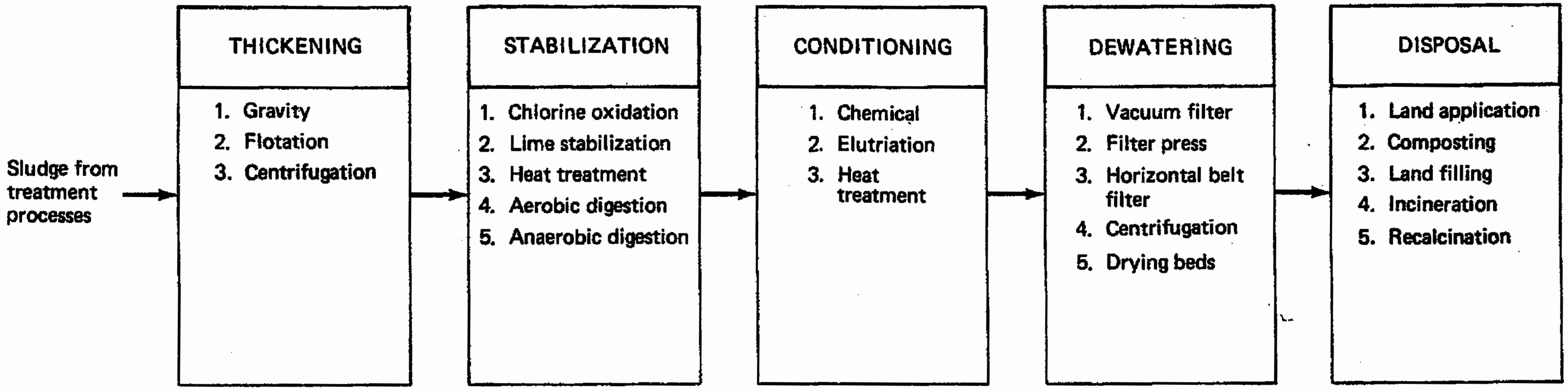


Figure 4-4 Alternative Unit Operations and Processes for Sludge Processing and Disposal.

TABLE 4-6 *Unit Operations and Processes Used for Processing and Disposal of Sludge*

Unit No.	Unit Operations and Processes[a]	Principal Applications	Ref.
SA1	Gravity thickening (UO)	Thickening of sludge is done to concentrate solids and reduce the volume. Gravity thickening is used for thickening primary, secondary, or combined sludges.	Chap. 16
SA2	Dissolved air flotation (UO)	Solids are concentrated by flotation. Air is dissolved at high pressure. As the pressure is released, air bubbles rise and float the suspended solids. Dissolved air flotation is generally used for waste-activated sludge or chemically precipitated sludge.	Chap. 16
SA3	Centrifugation (UO)	Solids are thickened or dewatered under the influence of centrifugal force (100–600 times the force of gravity).	Chap. 16
SB1	Chemical Oxidation (UP)	Sludge is stabilized to reduce pathogens, eliminate offensive odors, and control putrefaction. Chemicals such as chlorine, hydrogen peroxide, or ozone are commonly used.	Chap. 17
SB2	Lime stabilization (UP)	Excess lime is used to raise the pH to 12 or higher. A high pH sludge will not putrefy, create odor, or pose health hazard.	Chap. 17
SB3	Aerobic digestion (UP)	Sludge is aerated for extended period (10–15 days). Sludge is stabilized. Used at small plants.	Chap. 17
SB4	Anaerobic digestion (UP)	Sludge is digested under anaerobic condition. Methane is recovered as energy source.	Chap. 17
SC1	Chemical conditioning (UP)	Sludge is conditioned to improve its dewatering characteristics. Chemicals such as alum, iron salts, lime and polymers are used.	Chap. 18
SC2	Elutriation (UO)	Elutriation is washing of sludge to remove alkalinity. Elutriated sludge needs less chemicals to condition.	Chap. 18
SC3	Heat treatment or thermal conditioning (UP)	Heating of sludge at 140–200°C conditions and stabilizes the sludge.	Chap. 18

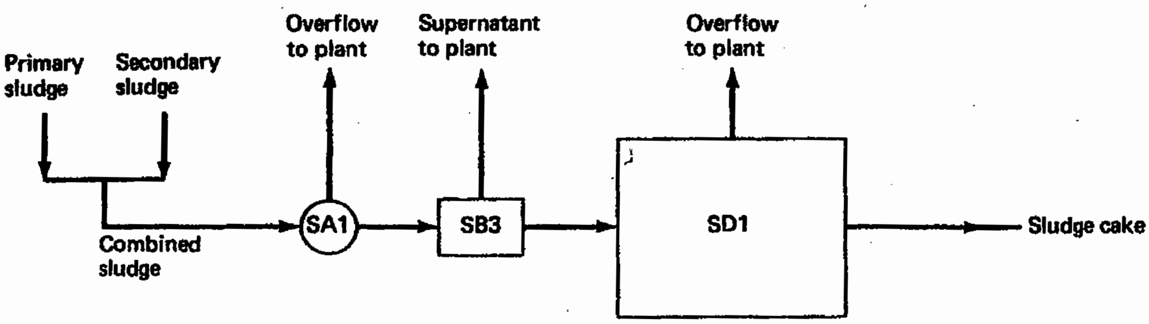
SD1	Drying beds (UO)	Sludge is dewatered in shallow beds of sand. The liquid is removed by underdrain system. Different variations are (1) perforated pipe under drain, (2) paved under drain, (3) artificial media, (4) combination with decanting system, (5) vacuum assisted, (6) slotted co-polymer plastic filter tiles.	Chap. 18
SD2	Centrifugation (UO)	Sticky chemical sludges are dewatered using centrifuge.	Chap. 18
SD3	Vacuum filter (UO)	Rotary vacuum filters are used for dewatering the sludge.	Chap. 18
SD4	Filter press (UO)	Sludge is pressed between filter cloth held vertically in a frame.	Chap. 18
SD5	Horizontal belt filter (UP)	Sludge is pressed between horizontally mounted continuous belts.	Chap. 18
SE1	Land application or biosolids utilization (UP)	Digested liquid sludge or sludge cake is applied over farmland. The nutrients nitrogen and phosphorus are taken up by growing plants.	Chap. 19
SE2	Soil conditioner by composting (UP)	Sludge cake is composted and then used as soil conditioner.	Chap. 19
SE3	Soil conditioner by heat drying (UP)	The sludge cake is dried at a temperature of approximately 370°C. At this temperature part of the volatile matter is lost. The sludge is used as soil conditioner.	Chap. 19
SE4	Sanitary landfilling (UP)	Raw or digested sludge is buried if a suitable site is available within an economical hauling distance. Daily cover of 15–30 cm, and final cover of over 60 cm of compacted soil is required.	Chap. 19
SE5	Incineration (UP)	Incineration involves drying of sludge cake followed by complete combustion of organic matter. Wet-air oxidation is also used.	Chap. 19
SE6	Lime recalcining (UP)	Lime used in single- or two-stage lime precipitation is recovered by recalcining the sludge. Recalcining involves heating of dewatered lime sludge to about 1000°C in multiple-hearth furnace.	Chap. 19
SE7	Pyrolysis (UP)	Pyrolysis or destructive distillation is heating of sludge in oxygen free atmosphere to about 300–700°C. Gas, liquid, solid (charcoal), fractions are produced.	Chap. 19
SE8	Wet oxidation (UP)	Sludge solids are incinerated in liquid phase at high temperature (200–300°C) and high pressure (5–20 MN/m^2).[b]	Chap. 19

[a]UO = unit operation. UP = unit process.
[b]MN = megaN.
Source: Adapted in part from Refs. 7, 12–18, 20, and 21.

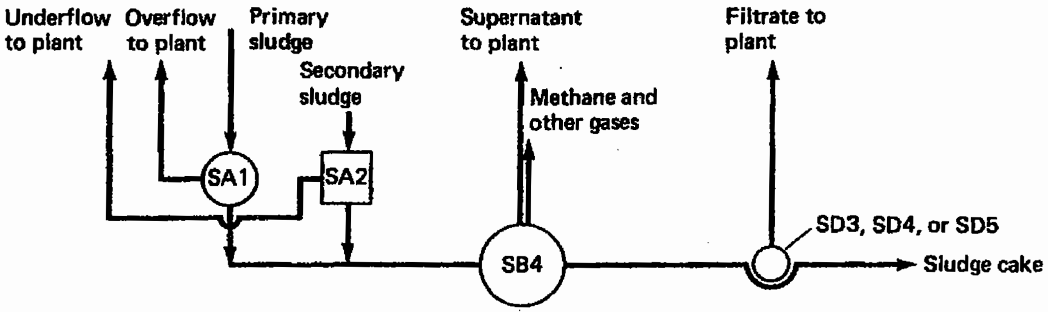
TABLE 4-7 Performance Data on Various Sludge-Processing Unit Operations and Processes

Sludge-Processing Systems	Sludge Stream Processed	TSS in Incoming Sludge (%)	TSS Captured in Processed Sludge (%)	TSS in Processed Sludge (%)	Returned Liquid	
					BOD$_5$ (mg/L)	TSS (mg/L)
Thickener						
Gravity (SA1)	Primary	3–6	85–92	8–10	160–600	300–1000
	Primary + waste-activated sludge	2–5	80–90	4.5–8.5	166–600	300–800
	Waste-activated sludge	0.5–1.2	75–85	2.5–3.5	100–400	200–600
Dissolved air flotation (SA2)	Waste-activated sludge	0.5–1.2	75–85	2–4	160–600	400–1000
Centrifugation (SA3)	Waste-activated sludge	0.5–1.2	75–90	2–6	50–500	100–1000
Stabilization						
Aerobic (SB3)	Thickened combined sludge	3–8.5	—	—	100–1000	1000–10000
Anaerobic (SB4)	Thickened combined sludge	3–8.5	—	—	1000–10000	3000–15,000
Dewatering						
Vacuum filter (SD3)	Digested sludge with chemical conditioning	3–8	90–98	15–25	500–5000	1000–20,000
Pressure filter (SD4, SD5)	Digested sludge with chemical conditioning	3–8	90–98	20–50	500–4000	1000–15,000
Centrifugation (SD2)	Digested sludge with chemical conditioning	3–8	80–95	10–35	1000–10,000	2000–15,000

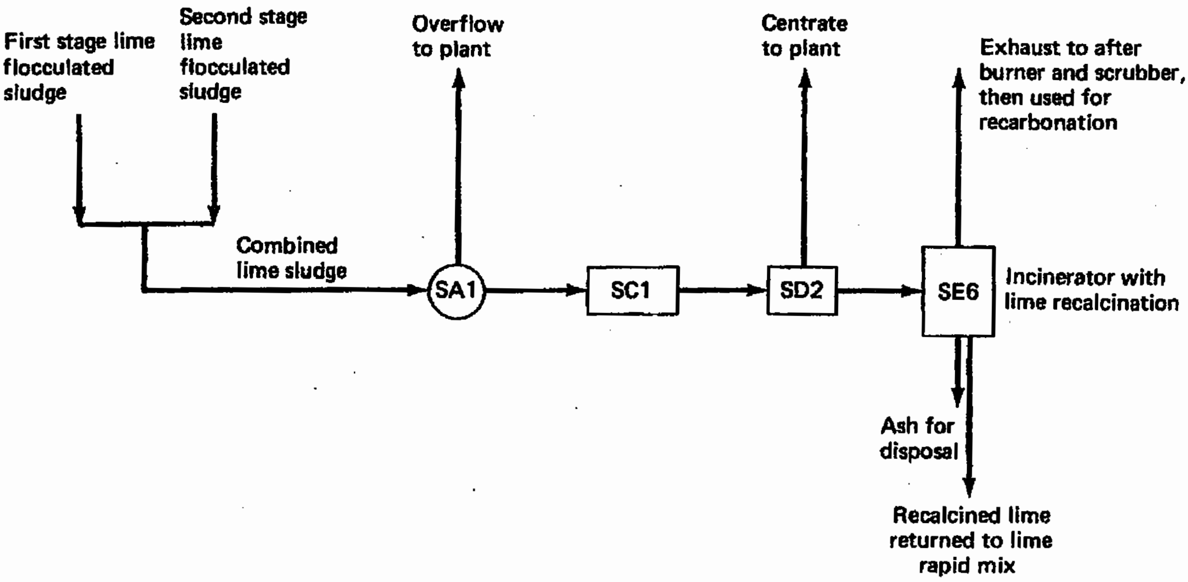
Source: Refs. 20 and 21.



a. Gravity thickening, aerobic digestion, and drying beds used for processing of primary and biological sludges from small wastewater treatment plants



b. Gravity thickening, air flotation, anaerobic digestion, and vacuum filtration used for processing of primary and biological sludges from large wastewater treatment plants



c. Gravity thickening, chemical conditioning, centrifugation and recalcination of chemical sludge from a two-stage lime treatment facility used at large wastewater treatment plants

Figure 4-5 Typical Sludge Treatment Flow Schemes (see Table 4-6 for process definition).

TABLE 4-8 *Factors Other Than Costs Normally Considered in Selection of Sludge-Processing Systems*

Sludge-Processing Systems	Land Requirements[a]	Adverse Climatic Conditions	Ability to Handle Flow Variations	Ability to Handle Influent Quality Variations	Industrial Pollutants Affecting Process	Reliability of the Process	Ease of Operation and Maintenance	Occupational Hazards	Air Pollution	Waste Products
Thickener										
Gravity (SA1)	Mod.	Freezing	Fair	Good	Min.	Good	Good	—	Odors	Thickened sludge. Liquid returned
Dissolved air flotation (SA2)	Mod.	Valve freezing	Fair	Good	Min.	Good	Fair	High pressure, explosion	VOCs	Thickened sludge. Liquid returned
Centrifuge (SA3)	Min.	—	Good	Good	Min.	Good	Fair	—	—	Thickened sludge. Liquid returned
Stabilization										
Aerobic (SB3)	Max.	Freezing	Good	Good	Max.	Good	Good	—	VOCs	Digested sludge. Liquid returned
Anaerobic (SB4)	Max.	—	Good	Good	Max.	Good	Good	Explosion, toxic gas	Gases	Digested sludge. CO_2 and methane gas. Liquid returned
Conditioning										
Elutriation (SC2)	Max.	Freezing	Good	Fair	Min.	Fair	Fair	—	Odors	Conditioned sludge. Liquid returned

Heat treatment (SC3)	Mod.	—	Fair	Fair	Min.	Good	Fair	Explosion	Odors	Conditioned sludge
Dewatering										
Drying beds (SD1)	Max.	High rainfall	Good	Good	Min.	Fair	Very good	—	Odors	Sludge cake. Liquid returned
Filtration (SD3, SD4 or SD5)	Mod.	—	Fair	Fair	Min.	Good	Good	Chemicals	Odors	Sludge cake. Liquid returned
Centrifuge (SD2)	Min.	—	Fair	Fair	Min.	Good	Good	Chemicals	—	Sludge cake. Liquid returned
Disposal										
Landfilling (SE4)	Max.	—	Good	Good	Max.	Very good	Good	Gas movement	Odors	Land reclaimed groundwater contamination
Land application (SE1)	Max.	Freezing	Good	Good	Max.	Fair	Good	—	Odors	Nutrients reused
Incineration (SE5)	Min.	—	Fair	Good	Min.	Very good	Fair	Explosion	—	Ash

[a]Min. = minimum, Mod. = moderate, Max. = maximum.
Source: Ref. 5.

4-4 PROBLEMS AND DISCUSSION TOPICS

4-1 A wastewater treatment plant has the following process train: bar screen, grit chamber, primary sedimentation, trickling filter (high rate), final clarifier, gravity filtration, and chlorine contact basins. Using the average percent removal efficiencies given in Table 4-2 for various units, estimate the effluent quality in terms of BOD_5, COD, UOD, TSS, TP, ON, NH_3-N, and NO_3-N. The influent quality after mixing with the return flows from the sludge-processing areas is as follows: $BOD_5 = 220$, COD = 450, UOD = 425, TSS = 255, TP = 9, ON = 8, NH_3-N = 21, and NO_3-N = 0. All units are in mg/L.

4-2 The sludge-processing train of a wastewater treatment facility includes gravity thickener, chemical conditioning, and centrifuge. Calculate the volume of sludge cake (m^3/d) and the concentration of TSS in the overflows from the thickener and centrifuge. Use the following data: Combined sludge is 380 m^3/d at 1.2 percent solids (sp. gr = 1.01), solids capture efficiencies of the gravity thickener and the centrifuge are 85 percent each. Gravity-thickened sludge has 4.5 percent solids (sp. gr = 1.03). Dry chemicals are added in the thickened sludge at a ratio of 3 percent of the weight of dry solids. The sludge cake has 25 percent solids (sp. gr = 1.06). Assume 75 percent chemicals added for sludge conditioning are incorporated into the sludge solids. See Chapter 13 for procedure.

4-3 A gravity thickener receives combined primary and secondary sludge. The combined sludge is 500 m^3/d and contains 1 percent solids. Dilution water at a rate of 350 m^3/d is blended with the sludge for improved thickener operation. Assume that the solids capture efficiency of the thickener is 90 percent and the thickened sludge has 6 percent solids. Calculate the average volume of the supernatant and average concentration of TSS. Specific gravities of combined and thickened sludges are 1.00, and 1.03, respectively.

4-4 A primary wastewater treatment facility uses sludge-drying beds for dewatering of raw primary sludge. The raw wastewater contains 250 mg/L TSS and 210 mg/L BOD_5. The average daily flow to the plant is 2784 m^3/d. In primary sedimentation basin TSS removal is 65 percent, and BOD_5 removal is 35 percent of incoming flow. The primary sludge has 3 percent solids. The solids and BOD_5 capture efficiency of the drying beds is 85 percent each, and the moisture content of the sludge cake is 72 percent. Conduct material mass balance analysis and determine the TSS and BOD_5 in the effluent from the primary treatment facility. Assume specific gravities of the primary sludge and sludge cake as 1.01 and 1.06, respectively.

4-5 Visit your local wastewater treatment plant. Obtain information on various unit operations and processes used to process liquid and residuals. Prepare the process train of the entire facility. Using the information provided in Tables 4-2 and 4-7, calculate the average effluent quality.

4-6 The effluent discharge permit of a state is 5/5/10/2 (BOD_5/TSS/TN/TP). The average influent BOD_5, TSS, TN and TP are, respectively, 200, 240, 44, and 15 mg/L (assume TN = 50%ON + 50% AN). Draw a process diagram using proper treatment processes to meet the state's effluent criteria. There could be numerous process combinations to achieve the same objective. You should select a process train that has the least number of treatment processes and is similar to a conventional wastewater treatment plant. Draw the process diagram and indicate effluent quality from each process.

4-7 A conventional suspended growth secondary wastewater treatment plant was designed. The average design capacity is 110,000 m^3/d, and the treatment plant is expected to receive wastewater of medium strength. Calculate the quantity of screenings, grit, scum, and primary and secondary sludge. Use the average values of residuals given in Table 4-3.

What will be the volume of combined sludge produced per day if solids content is 3 percent and specific gravity is 1.01?

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5

Predesign Studies

5-1 INTRODUCTION

For most wastewater projects, the engineering services are performed in three steps: (a) engineering reports or facility plan; (b) preparation of design plans, specifications, cost estimates, and contractual documents; and (c) construction compliance, inspection, administration, and acceptance. The U.S. Environmental Protection Agency (EPA) and state agencies have done much of the groundwork to develop and update the guidelines for preparation and completion of these engineering services. It is believed that the procedures for preparation of the facility plan and design plans and specifications that have evolved under many concepts and policies promulgated by the environmental laws and subsequent guidelines will continue to be followed in the future. Therefore, in this chapter the general discussions on engineering report or facility planning and design plans and specifications are provided. A model facility plan for a medium-sized wastewater treatment facility is given in Chapter 6. Design procedure and plans and specifications for the same facility are covered in Chapters 7–23.

5-2 PROJECT PLANNING

The predesign phase of any project involves a planning process. The planning process is a systematic method of (1) recognizing that a negative situation or problem exists; (2) collecting and analyzing the data about the situation or problem; (3) redefining the situation or problem in the light of analyzed data; (4) establishing objectives that, when achieved, will remedy the situation or problem; (5) developing alternatives and costs; (6) evaluating the effectiveness of various alternatives, timings, and priorities for achieving the objectives and modifying the plans to meet the changing conditions; and finally (7) selecting the alterative that utilizes the *best practicable waste treatment technology* that is most cost-effective and least controversial. In requiring this systematic approach, planning for water pollution control projects is similar to planning for any public works project. The end result of the planning process is an engineering report generally called engineering report, facility plan, engineering feasibility report, planning phase report, predesign phase report, or phase 1 report.

Historically, all planning reports for wastewater projects contain discussions on existing problems and the need for expansion, design period, population estimates, wastewater flow and characteristics, flow reduction, degree of treatment and selected process train, preliminary design, and cost estimates.

A preliminary project submittal may be required for review prior to preparation of an engineering report or facility plan. This submittal needs to include

- a description of problems that may have resulted in a need for a wastewater facilities project
- identification of government and consultant representatives authorized to provide information and seek regulatory agency approvals and decisions regarding the project
- identification of potential effluent discharge locations for the purpose of effluent quality requirements by the regulatory agency

Many states supply standard forms for preliminary project submittal.

5-3 FACILITY PLANNING

A facility plan is prepared to identify the water pollution problems in a specific area, evaluate alternatives, and recommend a solution.[1,2] Through the facility plan, the consultants make many decisions that are subsequently used in preparation of the detailed plans and specifications for the wastewater treatment facilities. Specifically, the recommendations are made concerning

- wastewater characteristics (initial and design) to include minimum average and peak flows; BOD, COD, suspended solids, nutrients, and any other constituents of concern
- infiltration/inflow problems and sewer rehabilitation
- existing wastewater treatment facilities and environmental concerns
- degree of treatment needed in order to meet the effluent and receiving water quality criteria
- intercepting sewer routing and other steps necessary for sewage collection
- renovation of existing facilities, flow modifications, new treatment units and processes, and site selection
- effluent disposal or reuse
- sludge handling, disposal, or reuse
- architectural, structural, mechanical, and electrical details usually excluded

The implementing agency (municipality or other) prepares a facility plan to develop a specific project that is cost-effective and environmentally sound.[1,3,4] Public participation in the decision-making process is required through public hearings and citizen advisory committees.

5-3-1 Preliminary Information for Facility Planning

The level of details required in a facility plan will vary according to the nature, size, and location of the undertaking. To make certain that applicants clearly understand the requirements of the State Revolving Fund (SRF), a preliminary meeting between the applicant, the applicant's consultant, and the state representative should be arranged. At this meeting, the major requirements, role of municipality, consultant, the state, EPA, and the extent of planning needed are discussed. Application forms and kits for preparing the facility plan are available from the state agency and are usually furnished at this meeting.

5-3-2 Contents of a Facility Plan

Several guides for preparing facility plans are available that are very helpful.[5-9] Many states and EPA regions have also developed instructional guides for preparing facility plans for specific regions of the country. These guidelines are very valuable and must be consulted to avoid unnecessary efforts and time delays. The contents of a facility plan are discussed below.

Project Description, Need, and Service Areas. Describe the project and its needs in terms of location, service areas, and problems associated with the existing treatment facilities. Discuss in detail the needs for improving or replacing the existing treatment facilities, including such factors as violations of effluent limitations, inability of existing facilities to meet the discharge compliance schedule, or potential public health hazards associated with the existing conditions.

Effluent Limitations. Include the effluent limitations for all discharges and the number of all National Pollutant Discharge Elimination System (NPDES) permits issued to facilities in the planning area. The effluent limitations may be secondary treatment or higher levels of treatment.[10-13]

Existing and Future Conditions. Describe the existing and future environmental conditions of the planning area. The description should be sufficient to provide a basis for analysis of alternatives and determination of impacts of the proposed action. The description should include the following items:[9,14-17]

- natural environment to include surface and groundwater hydrology, quantity, quality and uses of water, temperature, precipitation, and evaporation
- geology and aquatic plant and animal communities
- air quality and noise, energy production, and control
- economic, demographic and land use, and population projections
- statutory controls for sewer system
- industrial wastewater characteristics and composition
- combined municipal and industrial wastewater characteristics, infiltration/inflow analysis, and sewer system evaluation survey and rehabilitation

Engineering Criteria. The engineering criteria to be used in design of the project should be included with the facility plan.

Forecast of Wastewater Characteristics from the Service Area. The future size and capacity of the facility must be estimated over the design period. Based on population growth, industrial growth and flow, infiltration/inflow (I/I), and waste reduction measures, establish the future flow and wasteloads. The impact of the proposed project on all existing wastewater facilities, including gravity sewers, lift stations, and treatment facilities must be fully evaluated. The average, maximum, and sustained organic loads, TSS nitrogen, and phosphorus levels in the influent should be established.

Impact of industrial sources shall be documented. For projects with significant industrial contributions, evidence of adequate pretreatment strategies must be included along with documentation that industries are aware of the proposed new pretreatment limitations and user costs associated with the project. Documentation of the individual industrial participation in the project plan, including user charges, should be provided.

Septage and leachate may contribute significant organic load and other materials, which can cause operational problems and noncompliance with NPDES permit limitations. If septage or leachate are to be discharged to the wastewater treatment facility, proper allowance in process selection and plant design should be made.

Wastewater Treatment Process Alternatives and Alternative Selection. Once the size and scope of the water pollution problem is defined, develop various alternatives for wastewater treatment, effluent disposal, and sludge processing and disposal.[18,19] Screen systematically each alternative to determine those that can meet the federal, state, and local criteria. Then review the principal alternatives to identify those that have cost-effective potential. Develop cost data for the principal alternatives and then evaluate them critically for environmental consequences along with the cost-benefit analysis.[20] Finally, selecting one alternative, fully discuss the reasons for its selection and for the elimination of other alternatives.[21,22]

Description of Selected Wastewater Treatment Facility. Describe the selected treatment works in detail. Cover all elements, including service areas, collection sewers, interceptors, treatment processes, and alternatives considered, and selected methods for ultimate disposal or use of effluent and sludge. Develop preliminary engineering data, including design criteria, detention times, overflow rates, process loadings, removal efficiencies, initial and design flows and reserve capacity, and number of units and dimensions. Innovative and alternative (I/A) technologies (which are not included in the standards), if utilized in the facility will require special considerations. The state procedure for introducing and obtaining approval to use the technology must be addressed in the facility plan. Finally, a process diagram of the selected facility, including the sludge processing systems and recycled side streams, should be developed and included.

Operation and Maintenance of the Facility. A description of operation and maintenance of the facility is generally required by the approval agency. Portions of the project that involve complex operation or maintenance requirements shall be identified. This may include laboratory requirements for operation, industrial sampling, and self-monitoring. A plan for the method and level of treatment to be achieved during construction shall be developed and included in the facility plan that must be submitted to the regulatory agency for review and approval. The approved treatment plan must be implemented by inclusion in the plans and specifications, and through special contract documents.

Emergency Operation. All plants shall be provided with an alternate source of electric power or pumping capability to allow continuity of operation during power failure. This may involve (a) two independent power sources or (b) internal combustion engine equipment. The reviewing authority may issue a waiver based on the documentation of a stable long-term power outage record and holding capacity available for flow diversion.

Financial Status. Discuss the legal, institutional, managerial, and financial capabilities of the city. Indicate that the city has adequate funds for its share of construction, operation, and maintenance of the proposed facility. Where more than one municipality and industry are served by the project, it is necessary for the lead municipality to negotiate a service agreement.[23-25]

Environmental Impact Assessment Report. Consideration must be given to minimizing any potential adverse environmental effects of the proposed project. Compliance with planning requirements of national, state, and local regulatory agencies must be documented.

Prepare an environmental impact report as part of the facility plan. Include only the summary information in the main text. Attach the environmental impact assessment report as an appendix. EPA and most of the states have developed guidelines and procedures for preparation of the environmental impact assessment report.[21,22,26-29] The following topics should be addressed:

- project needs and description
- environmental setting, to include geological, hydrological, climatological and biological elements; flood plains and wetlands; historical or archeological resources; social and economic conditions of the population; land use and land use planning controls; and other programs and projects such as highways, water resources, etc.
- a description of alternatives considered during development of the proposed project
- a description and evaluation of potential direct, indirect, and secondary impacts, which may result from the proposed project, upon the social, economic, and environmental resources of the affected areas of the project, including an explanation of how each potentially adverse impact can be avoided, reduced to an acceptable level, or mitigated by structural and nonstructural measures
- an identification of beneficiaries and nonbeneficiaries of the proposed project and an assessment of the public acceptability of the project, its costs, and its potential environmental impacts

- a description of the potential adverse impacts that cannot be avoided should the project be implemented
- a description of the future of the environment without the proposed project
- a description of the extent to which the project may involve tradeoffs between short-term environmental losses and long-term gains or vice versa
- a summary of (a) comments obtained from appropriate agencies and the affected public, (b) an explanation of the methods used to obtain the comments, (c) documentation of coordination with agencies and public, and (d) a discussion of how specific concerns were considered in the evaluation of alternatives and the planning of the proposed project

Public Participation. The public should participate from the beginning in the facility-planning process so that interests and potential conflicts may be identified early and considered as the planning proceeds. The planners should define issues and analyze information so that the public will clearly understand the costs and benefits of alternatives considered during the planning process. Public hearings must be held. A report summarizing public participation should be prepared and submitted as part of the facility plan.[30] Many states require a slightly different format for the environmental impact assessment report and public participation schedule for applications submitted under the SRF Program.

Cost Breakdown. Prepare cost of performing the facility plan, design plans and specifications, and the construction costs of the project and provide them in the facility plan.

Time Schedule for Project Milestones. Provide in the facility plan the time schedule and project milestones for completion of the design plans and specifications and for construction of the project.

Appendixes. In order to keep the main text of the facility plan brief, the following information may be arranged in various appendixes and cross-referenced in the main text of the plan:

1. Preliminary designs, technical data, and cost estimates of alternatives
2. Infiltration/inflow analyses and sewer evaluation surveys
3. Environmental impact assessment report
4. Agreements, resolutions, and comments
5. Copies of the permits for the facility
6. Public participation report

5-3-3 Submission and Approval of the Facility Plan Submit the completed facility plan in the prescribed format to the regional or state clearinghouse, EPA, and regulatory agency for review and comments. The state and EPA criteria for review are presented in several publications.[5,9] The plan should then be revised or amended as necessary.

5-4 PREPARATION OF DESIGN PLANS, SPECIFICATIONS, COST ESTIMATES, AND SUPPORT DOCUMENTS

Once a community's facility plan application is approved by the state water quality agency, the design phase of the project is undertaken. This phase deals with preparation of detailed engineering design plans, specifications, and cost estimates. In this section, the basic requirements of design plans, specifications, and cost estimates are presented.

5-4-1 Preliminary Information for Design Plans and Specifications and Cost Estimates

The application to prepare the design plans and specifications may be necessary on specified forms in which each item is completed in accordance with the instructions. Of particular importance are such items as source of local share of project costs (general taxes, sewer revenue funds, etc.), a copy of the resolution authorizing the official representative (mayor, council member, etc.) to act on behalf of the applicant, and a statement regarding the availability of the proposed site. Many other documents needed may include

- proposed contracts or explanation for selection of consulting engineers
- user's charge or resolution indicating that a user charge system will be developed in accordance with the regulations
- letter of agreement from industries served, indicating compliance with the industrial cost recovery system developed for the project
- copy of existing sewer use ordinance or a letter of intent that such an ordinance will be enacted
- assurance of compliance with Civil Rights Act
- statement or resolution indicating compliance with the Uniform Relocation of Land Acquisition Policies Act
- copy of service agreements

Details on each of the above documents are available in Refs. 5 and 31–33. Predesign conferences may be necessary by the state and/or EPA to discuss the responsibilities and to finalize the administrative and technical requirements of the project. Many technical and administrative items that are considered during the design phase of the project are discussed in the following sections.

5-4-2 Project Design Plans and Specifications

The project, in addition to being designed in accordance with sound engineering practice, must take into account those engineering and environmental measures recommended in the approved facility plan. The end product of the design phase is a set of plans (drawings), specifications, and detailed construction cost estimates and contract documents that are suitable for bidding and construction purposes. The project specifications must comply with the state and federal requirements. Key elements of these requirements to be included in the contract documents are highlighted below.[5]

Technical Provisions of Specifications. The following items must be addressed in the plans and specifications for the project:

- General: Project title, design criteria, design details, and specifications should be indicated clearly.
- Reliability and flexibility: The proposed facilities must be reliable and provide for flexibility in operation. These include multiple units, ample pumping capacity and standby units, standby power, or provision for wastewater storage.
- Bypassing: Preventive measures for bypassing are needed during construction and operation.
- Mitigative measures: Mitigative measures required by environmental impact assessment report must be complied with. Examples are soil erosion control, hours of operation, backfilling and seeding, structural design for buildings in a flood plain, etc.
- Public water supply: Public water supplies must be protected by adequate backflow prevention devices (double check valves, air gap, etc.).
- Safety precautions: Occupational Safety and Health Administration (OSHA) and applicable state and local requirements must be complied with.[34,35]
- Equipment: Except where based on performance specifications, trade names may be specified for major items of equipment. In selecting equipment and components, the consultant must give careful considerations to equipment that can be operated and maintained with the least effort. Use of mercury in equipment must require special review and approval.
- Emergency alarms: Adequate alarms must be provided to warn of failures or dangers.
- Sewers: Sewers must be tested for infiltration. They must maintain minimum scouring velocity and have adequate capacity during peak flow periods.
- Pretreatment: Incompatible industrial wastes must be pretreated where applicable.
- Chemical storage: Chemicals must be properly stored in a curbed area large enough to hold the entire volume in the event of an accidental spill. Also, adequate safety protection gear must be provided for plant personnel.
- Ventilation: Adequate ventilation must be provided in all areas where necessary (such as wet well, dry well, chlorine room, chemical storage area, etc.).
- Laboratory facilities: Laboratory facilities must be sufficient to give the plant operator control over the operational efficiency of the treatment plant. Additionally, facilities must be adequate to conduct sampling and testing as required by the NPDES permit or the state agency.
- Component identification: Equipment, piping, switches, instruments, and the like must be clearly marked for ease of identification.
- Project sign: Proper project sign must be displayed during construction and operation phases.

Plan Contents. Detail plans shall consist of plan views, elevations, sections, and supplementary views that, together with the specifications and general layouts, provide the working information for the contract and construction of the facilities. They shall also include dimensions and relative elevations of structures, the location and type of equipment, location and size of piping, water levels, and ground elevations. The design plans should specifically contain the following components:[36]

- plans of sewers: A comprehensive plan of existing and new proposed sewers, topography and elevations, streams and boundaries, and streets should be provided. The plan and profile of sewers showing size, material, profile of direction of flow, length, invert and ground surface elevations, manholes, etc., should be clearly specified.
- pumping stations: A plan for pumping stations should include location, pump layout, suction and discharge piping and structures, topography and elevations, high and low water elevations, hydraulic profile, and other relevant information.
- wastewater treatment plant: The design plans and specifications should include plant location; general layout; schematic flow diagrams; location, dimensions, and elevations of all units and structures; pipings and appurtenances; bypassing; hydraulic profiles at minimum; design average; peak hourly flows, and elevations of high and low water levels in the receiving waters.

Specifications. Complete sealed technical specifications shall be submitted for the construction of sewers, wastewater pumping stations, wastewater treatment plants, and all other appurtenances and shall accompany the plans.[36]

The specifications accompanying construction drawings shall include, but not be limited to, specifications for the approval procedures for operation during construction. All construction information not shown on the drawings, which is necessary to inform the contractor in detail of the design requirements for the quality of materials, workmanship, and fabrication of the project, should be contained in the specifications.

The specifications shall also include the type, size, strength, operating characteristics, and rating of equipment; allowable infiltration; the complete requirements for all mechanical and electrical equipment, including machinery, valves, piping, and jointing of pipe; electrical apparatus, wiring, instrumentation, and meters; laboratory fixtures and equipment; operating tools and construction materials; special filter materials, such as stone, sand, gravel, or slag; miscellaneous appurtenances; chemicals when used; instructions for testing materials and equipment, as necessary to meet design standards; and performance tests for the completed works and component units. It is suggested that these performance tests be conducted at design load conditions wherever practical.

Supplemental General Provisions of Specifications. The requirement for supplemental general provisions in the specifications are covered in Ref. 33. These provisions include conditions relating to the following subjects:

- audit and access to records
- price reduction for defective cost or pricing data
- contract work hours and safety standards
- equal employment opportunity
- the specified minimum wage rates
- a covenant against contingency fees; antikickback regulations, gratuities, etc.

- copyrights
- a clean air and water clause

Bonding/Insurance. For the construction contracts the bonding and insurance requirements must be a part of the specifications. These include:[6]

- bid bond
- performance bond and payment bond
- fire and extended coverage, workmen's compensation, public liability and property damage, and "all risk" insurance as required by local or state law
- flood insurance as required during and after construction

5-4-3 Project Cost Estimates

The consultant is to prepare detailed construction cost estimates based on the scope of work as reflected in the project plans and specifications. This estimate is used to judge the reasonableness of the bids received.[5]

5-4-4 Continuing Work

While the following items need not be completed until the construction phase of the project is under way, work on them during the design phase should be maintained to ensure their timely completion.[6]

Plan of Operation. A plan of operation is required for all treatment facilities. The plan should be, in large measure, a sequential listing of actions needed to ready the plant and its personnel for operation when construction is complete. Matters such as staffing and training requirements, operation and maintenance procedures, reports, laboratory testing, and the like must be considered in the plan. In short, the plan must detail the "who, when, and where" of the facility operation and maintenance.

The operation and maintenance manual (prepared in conjunction with the plan of operation) is especially important because it provides plant personnel with detailed instructions for assuring efficient operation and proper maintenance of all plant components (including off-site pump stations, etc.). This manual should discuss how the facility is to be operated to meet the effluent standards contained in the NPDES permit and other state and federal requirements.

User Charge and Industrial Cost Recovery Systems. User charges are fees paid by users of the facilities to cover the operation and maintenance costs of the system. Industrial, commercial, and residential users are charged a proportionate fee based on the wastewater treatment service provided. Grantees should consult with the state agency and EPA before completing a user charge system.

At the time of the application for design plans and specifications, the applicants must submit a statement or resolution acknowledging their awareness of the need for preparing a user charge system and a procedural schedule for completing such a system. By the time the project is completed and ready to operate, an approved user charge system must be ready to be implemented.

Industrial Cost Recovery (ICR) is a system that recovers from industrial users of the wastewater treatment facilities industry's proportionate share of the capital and operating cost of the project. The recovered funds are generally used by the grantee primarily for loan payments, expansion, and reconstruction.[37-39]

Revision to Approved Plans. Any deviations from approved plans or specifications affecting capacity, flow, operation of units, or point of discharge shall be approved, in writing, before such change are made.[36] Plans or specifications so revised should, therefore, be submitted well in advance of any construction work that will be affected by such changes to permit sufficient time for review and approval. Structural revisions or other minor changes not affecting capacities, flows, or operation will be permitted during construction without approval. "As-built" plans clearly showing such alterations shall be submitted to the reviewing agency at the completion of the work.

5-5 DISCUSSION TOPICS

5-1 Obtain the most recently updated copy of Ref. 9. Review the chapter on preparation of facility plan. Prepare a detailed table of contents of a facility plan using the information contained in this source.

5-2 Review Refs. 26–28. Prepare a detailed table of contents for an environmental impact document for a wastewater treatment facility.

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6

Model Facility Plan

6-1 INTRODUCTION

A facility plan is prepared to identify the water pollution problems in a specific area, develop design data, evaluate alternatives, and recommend a solution. Most of the data developed in a facility plan are used in preparation of design plans, specifications, and cost estimates of the wastewater treatment facilities.

The purpose of this chapter is to develop a *model facility plan* for a medium-sized community. Efforts have been made to simplify the problem and keep the contents of the facility plan realistic and brief. In actual practice it is not uncommon to find facility plans for a medium-sized community that are several hundred pages in length, supported by many separate appendixes that are also voluminous. Such an effort is not warranted here. However, a generalized format has been developed in this chapter for preparation of a facility plan and for accruing the needed design information. This information is used for preparing the design plans and specifications for a medium-sized wastewater treatment facility in Chapters 7–23.

The names of the town and state and all other information used to prepare this facility plan are not real. The information is meant only to be illustrative. It is neither complete nor definitive.

6-2 PROJECT IDENTIFICATION

Owner/applicant:	Modeltown, Anystate
Project name:	Construction of Wastewater Treatment Facilities
Project number:	EPA Federal Grant Number EPA-XXX-000 State Environmental Protection Agency Project Identification Number XXX-000-XXX
Discharge permit number:	EPA NPDES NO. XXX State Pollution Control Permit No. XXX
Final submission date for the facility plan	January 31, 1998

6-3 OBJECTIVES AND SCOPE

The City of Modeltown currently owns and operates three small wastewater treatment plants that utilize trickling filters, stabilization ponds, and an aerated lagoon. These treatment facilities are inadequate and often are inoperable, causing serious water pollution problems, public nuisances, and health hazards. The existing facilities cannot provide adequate wastewater treatment without renovation or replacement. A Facility Plan Supplement for the construction of wastewater treatment facilities was approved by the state Environmental Protection Agency. This facility plan is compiled to support the city's application for funding from the state under the State Revolving Fund (SRF) Program. The objectives for this facility plan are to (1) develop and evaluate alternative wastewater treatment facilities, (2) select a cost-effective plan that will provide compliance with the state and federal water quality standards, and (3) develop the preliminary design and cost data for the proposed wastewater treatment facility.

6-4 EFFLUENT LIMITATIONS AND INITIAL DESIGN PERIOD

Pursuant to the state's continuing planning process, all of the water segments in the area have been classified as effluent limited segments. Thus, any project for the construction of publicly owned wastewater treatment works must be based on consideration of alternative best practicable waste treatment technology (BPWTT), including physical, chemical, and biological treatment, natural systems, on-site treatment and disposal, discharge

TABLE 6-1 Present and Projected Effluent Limitations of the State

	Current Effluent Limits		Projected Effluent Limits	
Parameter	**Average 30 Consecutive Day Sample**	**Average 7 Consecutive Day Sample**	**Average 30 Consecutive Day Sample**	**Average 7 Consecutive Day Sample**
BOD_5(mg/L)	20	30	10	20
Total suspended solids (mg/L)	20	30	10	20
Ammonia-N (mg/L)	—	—	1	2
Total-N (mg/L)[a]	—	—	10	14
Total-P (mg/L)[b]	—	—	1	2
Fecal coliform number/100 mL	200	400	100	200
pH	Shall remain between 6.0 and 9.0	Shall remain between 6.0 and 9.0	Shall remain between 6.5 and 8.5	Shall remain between 6.5 and 8.5

[a]Total-N = total nitrogen expressed as N. It includes organic, ammonia, nitrite, and nitrate nitrogen.
[b]Total-P = total phosphorus expressed as P. It includes organic and inorganic phosphorus.

into natural waters, and reuse of effluent.[1,2] However, a minimum of secondary treatment with nitrogen and phosphorus removal is required.

The EPA has issued a municipal discharge permit (NPDES) to the city, which expires on July 1, 1999. The state pollution control permit issued to the city also expires on November 30, 1999. The current and projected effluent limitations of the state are summarized in Table 6-1. Immediately upon issuance of the new permit from the state, the permittee must comply to the projected effluent limits within a period of one year. This implies that the upgraded facility must be completed and set into operation by January 2000. Therefore, year 2000 is established as the *initial year* for the project.

6-5 EXISTING AND FUTURE CONDITIONS

6-5-1 Economic, Demographic, and Land Use

Modeltown is a thriving community located at the confluence of the East and West Forks of the Big River. The economy of the town is based on agriculture, allied enterprise, and retail business. The major industries include one slaughterhouse, one meat-processing and packaging plant, one dairy, and two canning industries. The town also has large business and commercial establishments. There are 12 modern public grade and high schools and one college that also serves the surrounding farming area.

The planning department of the city, based on census data, has estimated the current and future populations of the town. The census and estimated population data for the town are summarized in Table 6-2 and plotted in Figure 6-1. These population forecasts have been coordinated with the designated management agency for Modeltown and are in conformance with the areawide wastewater management population forecasts established under the Clean Water Act.

TABLE 6-2 Census and Estimated Population of Modeltown

Year	Population
1960 (census)	7500
1970 (census)	11,700
1980 (census)	23,000
1990 (census)	40,000
1995 (estimated)	48,000
2000 (estimated, initial year)	58,800
2005 (estimated)	68,000
2010 (estimated)	76,000
2015 (estimated, design year)	80,000

Note: See Chapter 2 for methods of population forecasting.

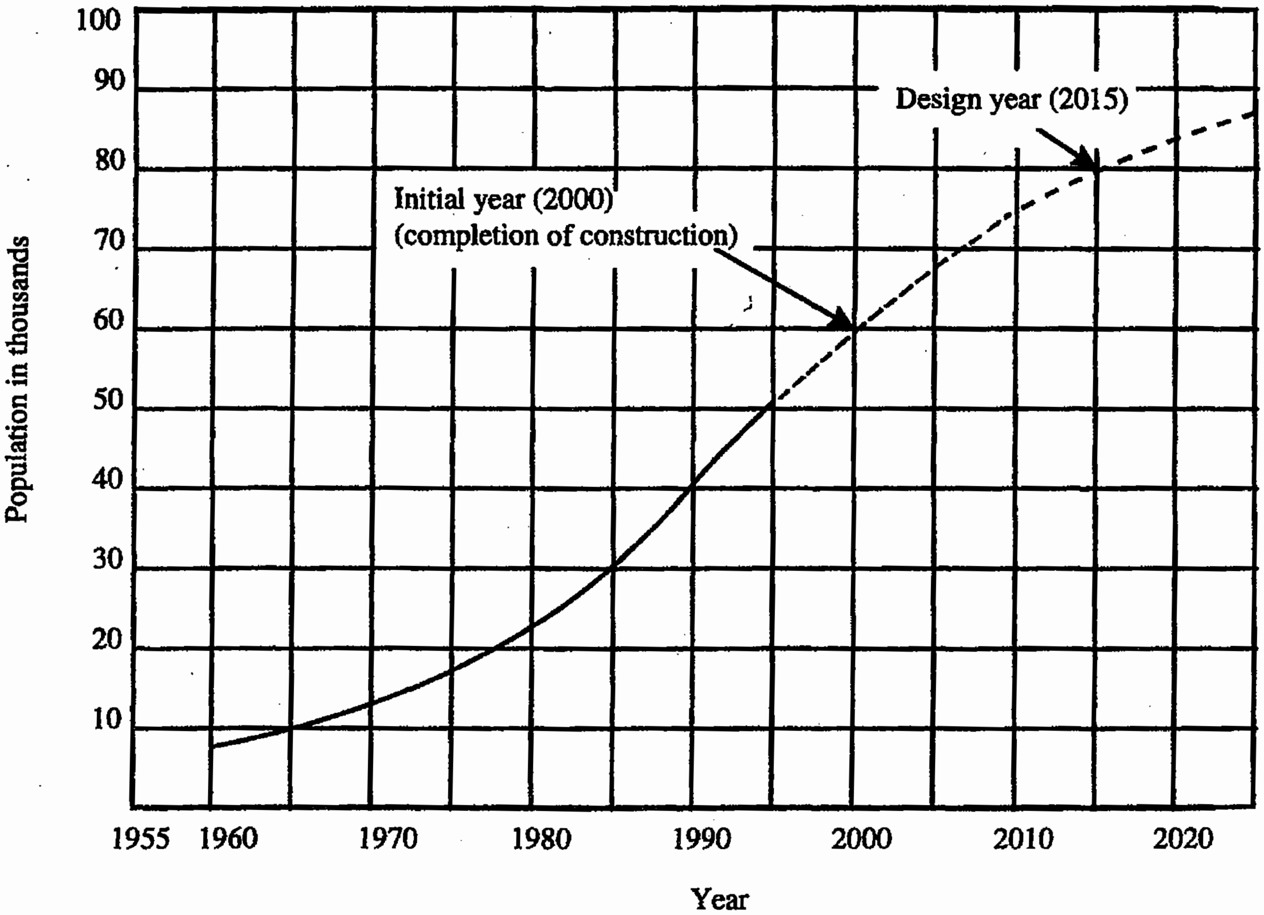

Figure 6-1 Census and Estimated Population Data for the City.

The existing land use within the city has not changed substantially from that presented in the 1995 Comprehensive Plan developed by the City Planning Commission. There has been some change in transition of vacant land to single-family dwellings since the plan was developed. The land use plan for 1995 and that for the assumed design year (2015) are illustrated in Figure 6-2.[a]

6-5-2 Natural Environment

The region is subjected to frequent precipitation with the average annual rainfall being approximately 89 cm (35 in.), with high-intensity showers during the late summer and early fall. The climate is mild with minimum and maximum temperatures ranging from −2°C in the winter to 40°C in the summer. Gentle, rolling plains characterize the topography of the area. Soils range from clays to a mixture of clay and silt.

There are no surface water impoundments in the planning area. Groundwater is of high quality and serves as the only source of water supply for the town. The East and West Forks of the Big River are grossly polluted with sewage effluent discharged from

[a]The design or staging period is assumed to be 15 years. This is based on the expected flow growth factor. The flow growth factor for this problem is discussed in Sec. 6-6-4. Additional discussion on design and staging periods may be found in Sec. 2-2.

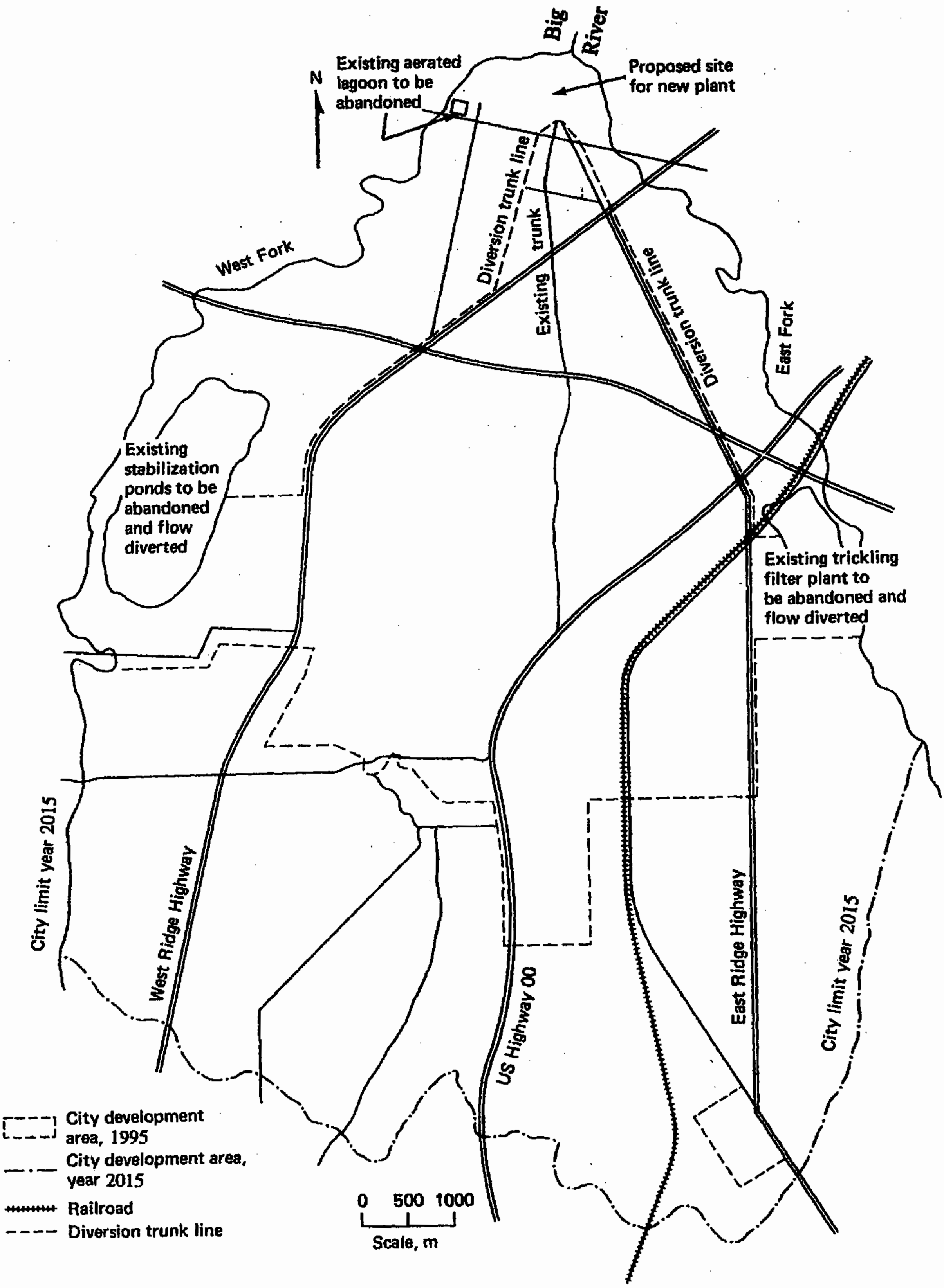

Figure 6-2 Modeltown Existing and Proposed Wastewater Treatment Facilities, Major Roads and City Limits: Years 1995 and 2015 Populations 48,000 and 80,000, Respectively.

TABLE 6-3 30-Day Mean Low Flow and Water Quality Data for the River at the Confluence of the East and West Forks

Condition	Value
Mean dry weather flow conditions	
Mean flow (cfs)	120
Mean flow (L/s)	3400
Average water depth (m)	1.2
Average width (m)	5.5
Average velocity (m/s)	0.5
Water quality at mean flow condition	
BOD_5 (mg/L)	6.8
Total dissolved solids (mg/L)	290
DO (mg/L)	4.0
Ammonia nitrogen (mg/L)	2.0
pH	7.3
Total phosphorus (mg/L)	0.4

the wastewater treatment plants. The 30-d mean low flow and water quality data for the river at the confluence of the East and West Forks are summarized in Table 6-3.

6-5-3 Wastewater Collection and Treatment Systems

The sanitary wastewater collection system for the city was first constructed in 1945 to serve a small portion of the town. Since then, the sanitary sewer system has been periodically extended as need occurred to serve the newly developed areas.

The city sewerage system has a total length of 569 km (354 miles). The sewer lines are 15 cm (6 in.) minimum to 76 cm (30 in.) maximum diameter concrete and vitrified clay pipes with predominantly cement mortar and compression joints. The equivalent diameter of the sanitary sewers is 27 cm.[b] Some parts of the city have storm sewers that were first constructed in 1965 and then expanded as the need occurred. A small portion of the town still does not have storm sewers. There are no combined sewers in the entire service area.

Approximately 1250 residents in the southern section of the town are not served by the sanitary sewer system. These homes have individual septic tanks, absorption

[b]The equivalent diameter of sanitary sewer system is the weighted average obtained from Eq. (6-1):

$$\text{Equivalent diameter} = \frac{d_1L_1 + d_2L_2 + d_3L_3 + \ldots}{L_1 + L_2 + L_3 + \ldots} \tag{6-1}$$

where L_1, L_2, and L_3 are lengths of sewers of diameter d_1, d_2, and d_3. Typical length distribution of average sewer sizes are 15 cm, 4 percent; 20 cm, 71 percent; 38 cm, 13 percent; 50 cm, 4 percent; 60 cm, 3 percent; 68 cm, 4 percent; and 76 cm, 1 percent. See Table 7-1 for additional details.

trenches, and open ditches. There is some concern about groundwater contamination and public health hazard if this practice is allowed to continue. As mentioned earlier, the groundwater is the only source of water for this community.

There are three wastewater treatment facilities operated by the city. These include the stabilization ponds, the trickling filters, and an aerated lagoon. The locations of these treatment facilities are shown in Figure 6-2. A brief description of each of these treatment facilities is given in Table 6-4.

6-5-4 Statutory Controls for Sewer Systems

City ordinance prohibits the discharge of storm water, groundwater, roof runoff, foundation drainage, and other subsurface drainage into the sanitary sewers. Tight house connections are also required under the city ordinance.

All industrial discharges into the sewer system are regulated by the city ordinance. The ordinance requires grease traps for all food-handling establishments and pretreatment of all industrial wastes (where necessary) to a level that is compatible with the publicly owned treatment works (POTWs).

6-5-5 Industrial Wastewater Characteristics and Composition

There are five industries in town: one slaughterhouse, one meat-processing and packaging plant, one dairy, and two canning industries. Table 6-5 provides a summary of pretreatment and wasteload information from these industrial operations.

6-5-6 Combined Municipal and Industrial Wastewater Characteristics

Dry Weather Conditions. Flow data at wastewater treatment plants are essential for planning and design of treatment facilities. Detailed discussion on flow-measuring devices and design procedures are provided in Chapter 10. Three 90° V-notch weirs were installed in the intercepting sewers of the three treatment facilities. Hourly flow measurements were recorded over a period of three months from June 10 through September 9, 1995. Average dry weather flows were plotted at hourly intervals to obtain the diurnal flow patterns.

The typical dry weather diurnal flow curve for the area served by the stabilization pond is shown in Figure 6-3. Similar curves (not shown here) for the other two areas were also developed from the recorded flow data at the trickling filter plant and the aerated lagoon. The average, peak, and minimum daily flows were established from these curves. Based on the estimated connections and resident population, the average per capita flows were also obtained. These results and calculation procedures are summarized in Table 6-6.

Six 24-h composite wastewater samples of influent were collected at each of the three V-notches. These samples were analyzed for pH, BOD_5, total suspended solids, total phosphorus, and ammonia and total nitrogen. The average values of these analyses are summarized in Table 6-6.

TABLE 6-4 Description of Existing Wastewater Treatment Facilities Operated by the City

Treatment Facility	Description
Stabilization ponds	Three stabilization ponds are located on west side of the town which were first constructed in 1955. These ponds serve the residential, commercial, and industrial areas of the southwestern and central parts of the town. Each pond is 7.3 hectares (18 acres) earthen basin. The average water depth is about 1.2 m. Inner side slope is 2 horizontal to 1 vertical, and length to width ratio is 2. A manually cleaned bar screen precedes the ponds, followed by a flow division box that divides the flow equally into three ponds. The effluent is discharged into the West Fork by gravity. A lift station pumps the effluent into the West Fork under high flow conditions. The pumps have been repaired frequently; major renovation was done in 1980. The ponds, division box, and bar screen are surrounded by a 1.2-m (4-ft) high levee to protect against flooding. The effluent is of poor quality and the area residents have frequently complained about odor problems and mosquito infestation caused by these stabilization ponds.
Trickling filter plant	A treatment plant is located on the east side of the town and utilizes a manually cleaned bar screen and two high-rate trickling filters. This plant was constructed in 1975. The service area for this plant includes residential, commercial, and industrial establishments located on the southeastern and central part of the town. The trickling filters are each 27.5 m diameter (90 ft), 2 m deep (6 ft) with recirculation system. Two final clarifiers, each 18.3 m diameter (60 ft), provide clarification. A lift station pumps the raw sewage from the wet well into the trickling filters. The undigested waste sludge is pumped into the drying beds for dewatering. The sludge cake is landfilled on the plant property. The pumps at the lift station have been repaired frequently. When the pumps are inoperable, raw sewage bypasses the treatment plant and discharges into the East Fork. Area residents have frequently complained about the odors from this plant.
Aerated lagoon	An aerated lagoon is located on the north side of the town. The facility serves the residential, commercial, and industrial areas on the north-central part of the town. The aerated lagoon was constructed in 1985. It has nine floating aerators rated at 9 kW (12 HP) each. The average water depth and surface area are approximately 4.6 m (15 ft) and 1.2 hectare (3.0 acres), respectively. The length to width ratio is 2, and side slope is 4 horizontal to 1 vertical. Effluent is discharged by gravity into the river near the confluence of the East and West Forks. The total suspended solids and total BOD_5 in the effluent are considerably higher than the effluent limitations.

TABLE 6-5 Characteristics of Industrial Wastes Discharged into the Municipal Sewers

Industry	Location and Flow to	Flow (L/s)	Pretreatment Provided	Effluent into Sewers	
				BOD_5(mg/L)	TSS(mg/L)
Slaughterhouse	North industrial park (aerated lagoon)	8.5	Lined aerated lagoon	250	350
Meat-processing and packaging plants	North industrial park (aerated lagoon)	3.1	Grease trap and lined aerated lagoon	240	300
Dairy	East industrial park (trickling filter)	9.6	Lined aerated lagoon	200	240
Canning industry 1	West industrial park (stabilization pond)	6.1	Grease trap and lined aerated lagoon	230	300
Canning industry 2	West industrial park (stabilization pond)	7.7	Grease trap and lined aerated lagoon	240	320
	Total	35.0			

Note: 1 mgd = 1.54 cfs = 43.6 L/s = 3768 m^3/d.

Infiltration/Inflow (I/I) Analysis. An infiltration/inflow analysis[c] of the entire sewage collection system was completed as one of the requirements for obtaining a loan from the concerned State Water Pollution Control Agency under SRF program.[3] Visual inspections of a large number of key manholes showed that they were in excellent condition. The inspected sewers were free of solids depositions, root intrusions, or disrepair.

During the month of August, high-intensity storms and intermittent showers continued to occur over a 10-d period. The first high-intensity storm occurred on August 4, after a long dry spell. This storm caused a 10.2-cm (4-in.) rainfall in six hours. Hourly flow measurements recorded at the three V-notches on this day were used to determine the inflow into the sewer system. Since the groundwater table was low, the infiltration was negligible, and the excess flow over the dry weather diurnal flow into the sewer system was the inflow.[4] The inflow data of the August 4 showers for the area served by the stabilization ponds are illustrated in Figure 6-3. Similar plots for the other two areas (not shown here) were also developed. The maximum inflow resulting from the high-intensity showers of August 4 at the three wastewater treatment facilities is given in Table 6-7.

[c]An infiltration/inflow report is normally submitted as an appendix to a facility plan. This report contains the results of dry and wet weather flows and a cost-effectiveness analysis to treat the wet weather flows or to perform a sewer evaluation survey and rehabilitation program.

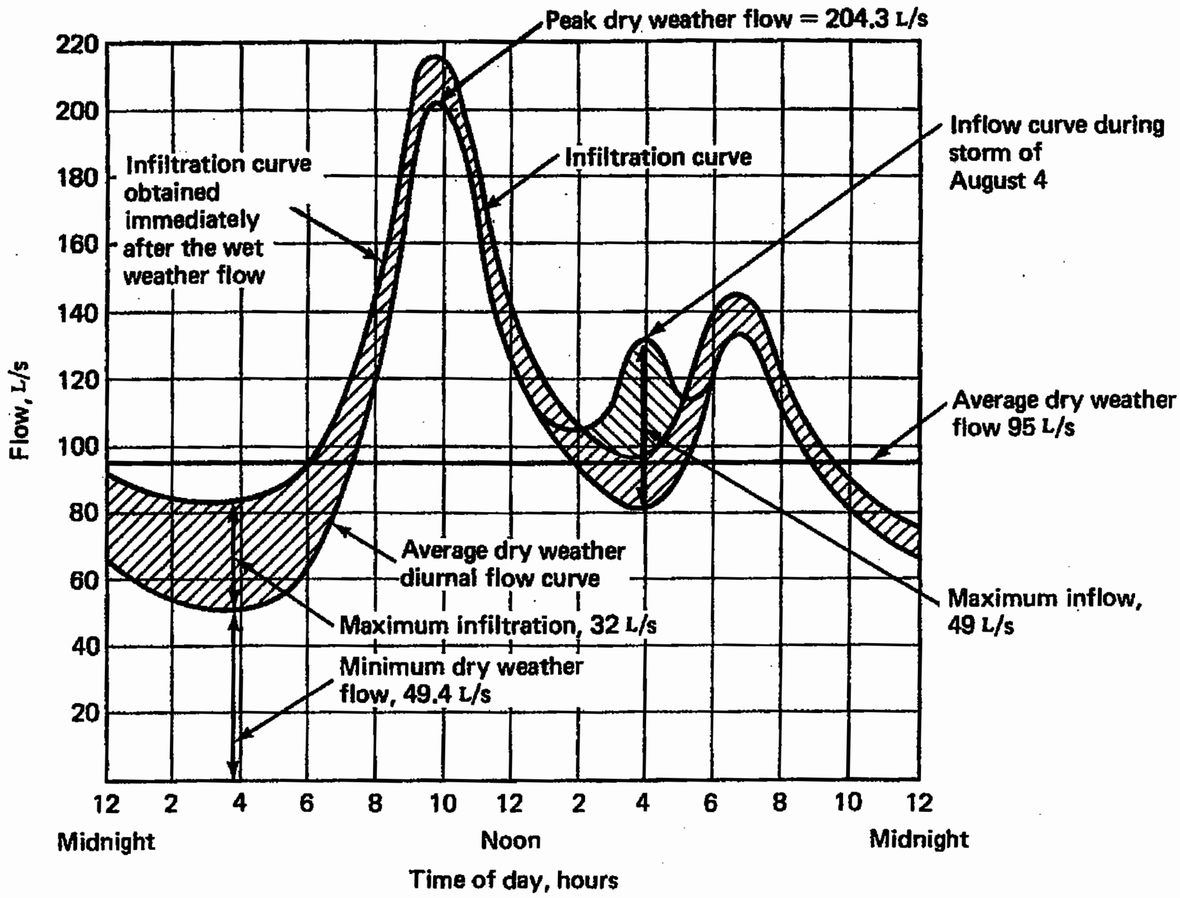

Figure 6-3 Average Diurnal Dry Weather Flow, and Infiltration/Inflow Measured at the 90° V-Notch Installed at the Stabilization Ponds. Similar Plots Were Also Developed at Other Plants.

Immediately after the continuous 10-d wet weather condition, the recorded flow data at the three V-notch weirs provided a measure of the wet weather infiltration into the sewer system. The maximum excess flow over the dry weather diurnal flow gave the infiltration values. The hourly variation of recorded infiltration in the interceptor sewer at the stabilization pond is shown in Figure 6-3. Similar plots for the other two areas were also prepared. The maximum infiltration values of the three service areas are summarized in Table 6-7.

The results in Table 6-7 indicate that the total average infiltration and inflow values from three service areas are 152 and 285 Lpcd, respectively [total I/I = 437 Lpcd]. The actual I/I value of 437 Lpcd is significantly less than the permissible allowance of 780 Lpcd (Sec. 3-3-2) often reached in a collection system. This gives a total infiltration/inflow allowance of 1330 L per d per cm per km (1436 gal per d per in. per mile) of collection system.[d] It would be more economical to collect and treat this flow than to rehabilitate the collection system. Therefore, a sewer evaluation survey was not cost-effective and was not considered.[5]

[d] $$\frac{437\ \text{Lpcd} \times 46{,}750\ \text{population served by sewerage system}}{569\ \text{km (length of sewerage system)} \times 27\ \text{cm (equivalent diam.)}} = 1330\ \text{L/cm} \cdot \text{km} \cdot \text{d}$$

See Table 6-6 and Sec. 6-5-3 for population and for sewer lengths and equivalent diameters.

TABLE 6-6 Dry Weather Influent Characteristics at Three Wastewater Treatment Facilities. The Information Is Based on 1995 Flow Data

Facility	Estimated Number of Connections	Estimated Population Served	Industries Served	Dry Weather Flow: Average L/s	Lpcd	Ratio Maximum to Avg.	Ratio Minimum to Avg.	Average Concentrations (mg/L): BOD_5	Total Suspended Solids	Total Solids	Total P	Ammonia-N	Total N	pH
Stabilization ponds	5520	18,100	Two canning industries	95	453	2.15[a]	0.52[b]	250	265	910	8.0	19	32	7.2
Trickling filter	4270	13,100	Dairy industry	67	442	2.00[c]	0.48[c]	255	265	890	7.8	20	37	7.3
Aerated lagoon	4890	15,550	Slaughterhouse and meat-processing and packaging industry	80	445	1.94[c]	0.50[c]	245	251	930	8.2	18	39	7.1
Septic tanks	320	1250	None	—	—	—	—	—	—	—	—	—	—	—
Total	15,000	48,000												
Average	—	—		242[d]	447	2.03	0.50	250	260	910	8.0[e]	19.0	36	7.2

[a]204.3/95 = 2.15 (max. and average dry weather flows are obtained from Figure 6-3).
[b]49.4/95 = 0.52 (min. and average dry weather flows are obtained from Figure 6-3).
[c]Obtained from the diurnal dry weather flow data for both service areas. The procedure is similar to that for stabilization ponds given in Figure 6-3. The diurnal dry weather flow and I/I plots for areas served by trickling filter plant, and aerated lagoon are not given in this chapter.
[d]Includes residential, commercial, and industrial flows. The industrial contribution is 35.0 L/s (see Table 6-5); therefore, the residential plus commercial contribution is (242 − 35) L/s = 207 L/s; the per capita residential and commercial contribution = (207 L/s × 60 s/min × 60 min/h × 24 h/d)/(48,000 − 1250) population served = 383 Lpcd.
[e]There is continued decrease in total P concentration due to increased usage of phosphate free detergents.
Note: Lpcd = liters per capita per day.

TABLE 6-7 Results of Infiltration and Inflow Study at Three Wastewater Treatment Facilities

Treatment Facility and Area Served	Maximum Infiltration[a]		Maximum Inflow[a]	
	L/s	Lpcd	L/s	Lpcd
Area served by stabilization ponds	32.0[b]	153[c]	49.0[b]	234
Area served by trickling filter plant	22.8[d]	150	50.2[d]	331
Area served by aerated lagoon	27.7[d]	154	52.2[d]	290
Average	28	152[e]	50	285[e]

[a]These values are in excess over the diurnal dry weather flow.
[b]See Figure 6-3.
[c](32 L/s × 24 h/d × 60 min/h × 60 s/min)/18,100 people = 153 Lpcd.
[d]These values are obtained from plots similar to Figure 6-3, not shown in this chapter.
[e]Total infiltration/inflow allowance = (152 + 285) Lpcd = 437 Lpcd.

6-6 FORECAST OF WASTEWATER CHARACTERISTICS FROM SERVICE AREAS

6-6-1 Projections under Existing Conditions

A recent residential and industrial survey conducted by the City Planning Board indicated that the per capita municipal wastewater flow will gradually increase approximately 6 percent by the year 2015 in spite of the anticipated flow reduction measures. This survey also indicated that the area industries anticipate significant industrial growth. Although most of the industries are attempting to reduce the water demand and the resulting wastewater discharges, the portion of the flow contributed by the industries will continue to rise because of increased industrial production.

6-6-2 Flow Reduction Measures

Conservation of water is gaining popularity among the general public and the regulatory agencies. Water conservation has the beneficial effect of reducing the volume of wastewater that must be collected and treated. There are a wide variety of water-saving devices that are effective in flow reduction. Many devices are commonly manufactured as part of most faucet combinations and are readily available from most appliance suppliers. Some of these devices are faucet aerators, flow-limiting shower heads, and water-saving toilet devices. Many of these devices and the water savings achieved by their use are discussed in Chapter 3.

Another form of in-house water saving might come from the reuse of water by means of a closed-loop system. The recycled water could be the waste streams from washbasins, bathtubs, and the laundry. These waters would be collectively treated and reused for toilet flushing and lawn sprinkling.

Most local governments have regulatory powers that can be exercised to achieve a reduction in water demands. These regulatory alternatives (also called nonstructural alternatives) may come in the form of plumbing code and zoning ordinances to require water-saving devices in new homes and in commercial and industrial establishments. Another method of achieving flow reduction is to encourage the use of plumbing fixtures and appliances that reduce water consumption. In all cases, a comprehensive community awareness program is necessary for the general public to understand the importance of water conservation and the costs and benefits of the water-saving devices.

6-6-3 Projection of Wastewater Characteristics for Initial and Design Periods

The social acceptability and the ease with which many of the water-saving devices can be installed in existing and new systems and the public attitude and cooperation toward use of water saving devices will all affect the flow reduction. However, it is expected that, in spite of all available watersaving techniques and all public awareness programs, there will not be any significant reduction in wastewater volumes. It is safe to assume, however, that implementation of all water conservation efforts will prevent a net (Lpcd) increase in the domestic and commercial flow rates.

The wastewater characteristics for the initial and design years were developed using the present flow records and data obtained from the infiltration and inflow studies. The basic procedure used to develop the initial and design flow characteristics are given below:

1. The average initial dry weather flow (year 2000) is calculated using a wastewater flow of 447 Lpcd (Table 6-6). This is equal to the current average flow. No increase in residential, commercial, and industrial flow rates is assumed.
2. The average design dry weather flow (year 2015) is calculated using no increase in residential and commercial flow rates and approximately a 244 percent increase in the industrial flow. The projected combined average flow rate for all three service areas is 475 Lpcd.[e] This gives an overall Lpcd increase of 6.3 percent in municipal dry weather wastewater flow rate.
3. The peak dry weather flows for initial and design years for the individual service areas were obtained using the measured ratio of maximum to average flow.
4. The peak dry weather flow for initial and design years for the combined facility for the three service areas were estimated using Eq. (3-1).[6,7]

[e] $$\text{Residential + commercial flow} = \frac{383 \text{ Lpcd (see footnote d of Table 6-6)} \times 80{,}000 \text{ population}}{24 \text{ h/d} \times 3600 \text{ s/h}} = 354.6 \text{ L/s}$$

Industrial flow = 35.0 L/s (Table 6-5) × 2.44 = 85.4 L/s

$$\text{Total flow rate} = \frac{(85.4 + 354.6) \text{ L/s} \times (24 \times 60 \times 60)\text{s/d}}{80{,}000 \text{ population}} = 475 \text{ Lpcd}$$

Percentage increase = (475 − 447) Lpcd / 447 Lpcd × 100 = 6.3

5. The wet weather peak flows for initial and design years were obtained by adding the average infiltration/inflow allowance of 437 Lpcd to the peak dry weather flows (see Table 6-7). This assumption is reasonable because newer sewers will have less infiltration/inflow, while older lines will have an increase in infiltration/inflow.
6. The minimum initial and design flows were obtained from the average measured ratio of minimum to average flows.
7. The quality of raw wastewater produced in the service area is not expected to change significantly over the design period. This assumption is reasonable since the per capita wastewater volume from residential and commercial areas will not increase over the design period, and industrial wastewater discharges into the municipal sewers will be strictly controlled under the city ordinance to meet the industrial pretreatment standards.

Tables 6-8 and 6-9 have been prepared from the above assumptions to summarize the wastewater characteristics at the initial and design periods.

6-6-4 Check the Design Period

The planning period of a facility may be divided into staging periods (10, 15, or 20 years) depending on the ratio of wastewater flow expected at the design and initial years (see Table 2-1). For the facility under consideration, the growth factor = (440 L/s)/(304 L/s) = 1.5 (see Table 6-9). The design or staging period at a growth factor of 1.5 is 15 years (Table 2-1).

6-7 WASTEWATER TREATMENT PROCESS ALTERNATIVES

A wastewater treatment process train is a combination of many unit operations and processes that are arranged to achieve the specific treatment objectives. To achieve the secondary level of wastewater treatment, various treatment and disposal alternatives were evaluated as part of the facility-planning process. Many treatment and disposal alternatives that have been evaluated in this facility plan are

- wastewater treatment process alternatives
- effluent disposal alternatives
- sludge processing and disposal alternatives

A brief description of each of the treatment and disposal alternatives follows.

6-7-1 Wastewater Treatment Process Alternatives

Comparative evaluations were conducted for physical-chemical and biological treatment processes and natural systems to meet the effluent limitations given in Table 6-1. Discussion of various physical-chemical and biological treatment processes, including natural systems, are presented in Chapters 4, 8–19, and 24. Readers are referred to these chapters for process description, effluent quality, and principal advantages and disadvantages

TABLE 6-8 Population and Wastewater Flows from Three Service Areas at Initial and Design Periods

	Year 2000 Initial Conditions					Year 2015 Design Conditions				
Treatment Plant	**Population Served[a]**	**Average Flow (L/s)**	**Peak Dry Weather (L/s)**	**Minimum Dry Weather Flow (L/s)**	**Peak Wet Weather Flow (L/s)**	**Population Served[a]**	**Average Flow (L/s)**	**Peak Dry Weather Flow (L/s)**	**Minimum Dry Weather Flow (L/s)**	**Peak Wet Weather Flow (L/s)**
Stabilization ponds	24,000	124[b]	267[c]	65[d]	388[e]	35,000	192[f]	413[g]	100[h]	590[i]
Trickling filter plant	16,700	86	172	41	257	24,000	132	264	63	385
Aerated lagoon	18,100	94	182	47	274	21,000	116	225	58	331
Total for the city	58,800	304	—	—	—	80,000	440	—	—	—

[a]Obtained from population projections of each service area.

[b] $\dfrac{24{,}000 \text{ population} \times 447 \text{ Lpcd}}{(24 \times 3600) \text{ s/d}} = 124 \text{ L/s}.$

[c]124 L/s × 2.15 (ratio of peak and avg. flows, Table 6-6) = 267 L/s.

[d]124 L/s × 0.52 (ratio of min. and avg. flows, Table 6-6) = 65 L/s.

[e] $267 \text{ L/s} + \dfrac{24{,}000 \text{ population} \times 437 \text{ Lpcd I/I allowance (Table 6-7)}}{(24 \times 3600) \text{ s/d}} = 388 \text{ L/s}.$

[f] $\dfrac{35{,}000 \text{ population} \times 475 \text{ Lpcd}}{(24 \times 3600) \text{ s/d}} = 192 \text{ L/s}.$

[g]192 L/s × 2.15 (ratio of peak and avg. flow, Table 6-6) = 413 L/s.

[h]192 L/s × 0.52 (ratio of min. and avg. flow, Table 6-6) = 100 L/s.

[i] $413 \text{ L/s} + \dfrac{437 \text{ Lpcd I/I allowance (Table 6-7)} \times 35{,}000 \text{ population}}{(24 \times 3600) \text{ s/d}} = 590 \text{ L/s}.$

TABLE 6-9 Characteristics of Combined Wastewater from Three Service Areas at Initial and Design Periods

Design Conditions	Initial Year 2000	Design Year 2015
Total population served	58,800	80,000
Average wastewater flow (Lpcd)	447	475
Average wastewater flow (L/s)	304	440
Minimum wastewater flow (L/s)	152[a]	220
Maximum dry weather flow (L/s)	669[b]	916
Peak flow	966[c]	1321
Average sewage characteristics		
BOD_5 (mg/L)	250	250
Total suspended solids (mg/L)	260	260
Total solids	910	910
pH	7.2	7.2
Total phosphorus (mg/L)	7.6[d]	6.0[d]
Ammonia N	19.0	19.0
Organic N	17.0	17.0
Total nitrogen (mg/L)	36.0	36.0

[a]304 L/s × 0.5 (ratio of min. and avg. flows for three areas, Table 6-6) = 152 L/s.

[b]$304 \text{ L/s} \left(1 + \dfrac{14}{4 + \sqrt{58.8}}\right) = 669 \text{ L/s}$ [see Eq. (3-1)].

[c]$669 \text{ L/s} + (58{,}800 \text{ population} \times 437 \text{ Lpcd I/I allowance, Table 6-7}) \dfrac{1}{(24 \times 3600) \text{ s/d}} = 966 \text{ L/s}$.

[d]Reduction in total phosphorus concentration continues to occur due to band on phosphorus based detergents. Average total phosphorus concentration in 1995 sampling data was 8.0 mg/L (see Table 6-6).

of each process. A physical-chemical treatment system utilizing coagulation, clarification, ammonia stripping, filtration, and carbon adsorption will produce an effluent quality that is superior to the desired effluent quality. However, there are many inherent difficulties that include (1) large quantities of sludge produced for processing at thickening, digestion, and dewatering facilities; (2) pumping to higher elevation due to increased head loss in filters; (3) carbon regeneration; (4) escape of VOCs from the stripping process; (5) higher level of trained personnel needed; (6) concern over transport, storage, and use of chemicals; and (7) high energy consumption. Furthermore, the physical-chemical processes have better applications for tertiary or advanced wastewater treatment. A discussion on advanced wastewater treatment facilities may be found in Chapter 24.

An evaluation of principal natural treatment systems for primary treated or secondary effluent indicates treatment superior to secondary level. However, there are many uncertainties and limitations of the natural systems, including (1) large land area is needed, and (2) disruptions are caused by severe weather conditions. General discussion on natural systems may be found in Chapter 24.

Biological treatment processes of the present day appear to be economical for secondary treatment with enhanced nutrient removal. Because of many uncertainties and inherent problems with physical-chemical and natural systems, the biological treatment system with enhanced nutrient removal has been selected. Primary treatment facility followed by anaerobic-anoxic-aerobic biological reactor and a final clarifier will produce the effluent quality given in Table 6-1.

6-7-2 Effluent Disposal Alternatives

A total of nine methods of effluent disposal were investigated in this facility plan:

1. Natural evaporation
2. Urban reuse
3. Industrial reuse
4. Agricultural reuse
5. Recreational and aesthetic reuse
6. Habitat restoration/enhancement
7. Groundwater recharge
8. Augmentation of portable water supplies
9. Discharge into the natural waters

General discussions on each of the above disposal methods, their suitability and limitations, and comparative evaluation are provided in Chapter 15. In summary, natural evaporation and agricultural reuse will require large land areas and are affected by adverse climatic conditions and may cause groundwater contamination. Exceptionally high effluent quality is needed for urban reuse, groundwater recharge, recreational and aesthetic reuse, and augmentation of potable water supplies. There are no natural wetlands around, and potential for constructed wetlands does not exist for habitat restoration and enhancement. No industrial market exists for process and cooling water in the immediate vicinity of the study area. At the same time, the treated effluent from the proposed facility will have a quality significantly higher than that of the secondary effluent. Its quality is sufficient to permit its safe disposal into receiving waters while meeting the proposed state and federal permit requirements. Therefore, effluent disposal into natural waters has been selected.

6-7-3 Sludge Processing and Disposal Alternatives

The objectives of sludge treatment and disposal are (1) to reduce the organic matter in sludge to a relatively stable and inoffensive material; (2) to reduce the volume for ultimate disposal; (3) to destroy or control pathogens; (4) to prevent air, water, and land pollution; and (5) to utilize biosolids. Sludge processing and disposal involves thickening, stabilization, dewatering, and disposal. Various alternatives of sludge processing and disposal have been evaluated in Chapter 4. Design details and equipment selection for sludge-processing systems are also given in Chapters 16–19. Based on an evaluation of various competing sludge-processing alternatives, the following systems have been selected:

1. Gravity thickening of combined primary and phosphorus stripped biological sludges
2. Sludge stabilization by anaerobic digestion
3. Belt filter press for dewatering

4. Use of biosolids on city property and park areas, organic gardening and agricultural uses, and excess or unused sludge disposed of by sanitary landfilling at the plant property[9]

Disposal of all residues shall be in accordance with the guidelines set under the Resource Conservation and Recovery Act.[8] Systems evaluation procedure and merits and demerits of different systems are provided in detail in Chapters 4 and 19.

6-8 ENGINEER'S DEVELOPMENT OF THE PROPOSED PROJECT

A total of five alternative wastewater collection and treatment systems (including no action alternative) were evaluated to select the most cost-effective wastewater treatment plant. These alternatives are presented in Table 6-10. Preliminary evaluation of these alternatives is given below.

TABLE 6-10 Wastewater Treatment Plant Alternatives

Alternative	Description
Alternative A—no action	Continue using stabilization ponds, trickling filters, aerated lagoon, and septic tanks.
Alternative B—three plants	Upgrade the trickling filter facility, and construct two new treatment plants to replace the stabilization ponds and aerated lagoon. All facilities must meet new permit requirements.
Alternative C—one treatment plant and two force mains	Abandon stabilization ponds and aerated lagoon. Construct two pumphouses and force mains to divert flows from stabilization ponds and aerated lagoon to the trickling filter plant. Upgrade and expand the existing trickling filter plant at this location to treat the combined diverted flows. The renovated trickling filter facility must comply with the new permit requirements.
Alternative D—two treatment plants and one diversion sewer	Abandon stabilization ponds and aerated lagoon. Construct a gravity intercepting sewer to divert wastewater from the stabilization pond to the aerated lagoon site. Construct a new plant at this site to treat the combined flows. Renovate the trickling filter facility. Both constructed and renovated facilities must comply with the new permit requirements.
Alternative E—one treatment plant and two diversion sewers	Abandon all three existing treatment facilities. Construct two gravity-intercepting sewers to divert flows from trickling filter plant and stabilization ponds to the aerated lagoon site. Construct a new treatment plant at this location to treat the combined flow. The effluent quality must comply with the new permit requirements.

6-8-1 Preliminary Evaluation of Treatment Plant Alternatives

A preliminary evaluation of various alternatives indicate that alternatives A, B, and C do not meet the basic requirements of water quality compliance, facility centralization, and energy conservation. For these reasons these alternatives were not given further consideration. A brief discussion is provided below. Alternatives D and E meet all above basic requirements, and a cost-effectiveness analysis of these two alternatives is given in Sec. 6-8-2.

1. The implementation of the no-action alternative (alternative A) would mean discharge of inadequately treated effluent into the receiving waters and violation of state permit conditions. Continued use of septic tanks and absorption trenchs might cause groundwater pollution. Both these conditions would result in a serious health hazard to the area residents. Odors emanating from the stabilization ponds, drying beds, and receiving waters will continue to be environmental issues. Thus, the inadequate wastewater treatment facilities would cause a loss of opportunity for orderly development and economic growth of the area.
2. The implementation of alternative B would provide a decentralized approach to wastewater collection and treatment. The decentralized approach reduces the processing of large volumes of wastewater at one location. However, the centralized wastewater treatment works in general permit improved planning and coordination of collection and treatment works, facilitate application of new technology, allow efficient monitoring of effluent by regulatory agencies, reduce the inventory system by providing multiple and compatible equipment, and economize the construction and operating costs. For these reasons, the city and the area regulatory agency have encouraged centralization of the facilities where possible.
3. The implementation of alternative C would require sewage pumping at an average of 192 L/s and 116 L/s across town and from north to south, respectively. Pumping of large volumes of sewage long distances is not economical. Also, standby pumping units and dual power sources are necessary to avoid pumping disruptions. Present trends in planning of wastewater treatment facilities are to select alternatives that minimize energy requirements and avoid uncertainties.

6-8-2 Cost-Effectiveness Evaluation of Alternatives D and E

A cost-effectiveness evaluation of alternatives D and E is necessary to finally select a process train for implementation. Detailed discussion on cost-effectiveness analysis may be found in Ref. 10. Alternatives D and E minimize pumping of sewage over long distances, utilize a partial or complete centralization approach, and provide acceptable effluent quality. These alternatives are described in great detail to compare and select the most cost-effective system.[f]

[f]Alternatives that meet effluent limitations, such as B and C, could not be dismissed in a facility plan without a more detailed consideration of the economies and environmental effects. In this example, however, only alternatives D and E have been evaluated to illustrate the procedure for selection of the most cost-effective and environmentally acceptable alternative.

Description of Alternative D. This alternative includes (1) diversion of flow from the stabilization pond to the aerated lagoon facility, (2) construction of a new plant at this site, and (3) renovation of the trickling filter facility. Each component is discussed below.

Diversion of Flow from Stabilization Pond to Aerated Lagoon. A 91-cm (36-in.) gravity diversion sewer will be constructed along the West Ridge Highway to divert an average design flow of 192 L/s. This sewer will intercept several existing collectors in its proposed length of 3.5 km (2.2 miles).

New Treatment Plant at Aerated Lagoon Site. A new activated sludge facility with biological nutrient removal capability will be constructed at the aerated lagoon site to treat the combined flows from the stabilization pond and aerated lagoon. The average design flow is 308 L/s [7.0 million gallons per day (mgd)]. Table 6-8 provides a summary of flows. The diagram includes preliminary treatment;[g] primary sedimentation; suspended growth biological system with anaerobic, anoxic, and aerobic treatment capability; final clarification; chlorination; and dechlorination. The sludge is thickened, anaerobically digested, and dewatered by belt filter presses. The sludge cake is applied over city land, and the excess is disposed of by sanitary landfilling.

Trickling Filter Plant. The existing trickling filter facility will be upgraded to treat an average design flow of 132 L/s (3.0 mgd). Table 6-8 gives a summary of the flow data. The grit removal facility and primary sedimentation basin with chemical precipitation for phosphorus removal will be added. The existing lift station, trickling filter units, final clarifiers, and chlorination facility will be renovated to provide full nitrification. A separate stage denitrification facility with clarification will be constructed after the trickling filter and final clarifier. Also, a dechlorination facility will be added. The sludge will be digested aerobically and dewatered over the existing drying beds. The sludge cake will be applied over city land and the remaining quantity landfilled.

Description of Alternative E. This alternative includes diversion of flows from the trickling filter plant and stabilization ponds to the aerated lagoon site. A new plant will be constructed to treat the combined flows. Various components of this alternative are discussed below.

Diversion of Flow from Stabilization Pond to Aerated Lagoon. The diversion trunk sewer is the same as described in alternative D.

Diversion of Flow from Trickling Filter Plant to Aerated Lagoon. A 76-cm (30-in.) gravity diversion sewer will be constructed to convey the flow from the trickling filter plant to the aerated lagoon site. This sewer will be 2.9 km (1.8 miles) long and will intercept many trunk sewers along its length on East Ridge Highway.

[g]Preliminary treatment facility includes bar screen, lift station, grit removal, and flow measurement facilities.

Treatment Plant at Aerated Lagoon Site. A new treatment plant will be designed for an average flow of 440 L/s (10 mgd) to treat the combined flow from three service areas. Initial and design flow data and wastewater characteristics are given in Table 6-9. The process diagram will include preliminary treatment; primary sedimentation; a combination of anaerobic, anoxic, and aerobic processes for biological nutrient removal; final clarification; and UV disinfection.

Cost Comparison of Alternatives. The construction, operation, and maintenance costs of wastewater treatment alternatives can be developed using many published cost curves.[11-14] Several computer programs are also available for preliminary design and cost estimates of wastewater treatment facilities.[14-16] However, these cost estimates are useful only for evaluation of wastewater treatment alternatives.

The construction, operation, and maintenance cost estimates for alternatives D and E are calculated from cost equations. These cost equations are developed and updated to May 1996 dollars, and are presented in Appendix C.

The capital and O&M costs are further updated to 1998 dollars. To estimate the ENR cost index for 1998, the ENR indices for the years 1994 and 1995 and four months average value for 1996 are plotted.[17] Linear projection for the desired period are utilized. The design engineers should use the actual value of the index but not estimated value. The method utilized here is used to estimate the projected costs beyond the period of available ENR index.[h] The procedure is presented for illustrative purposes only. Tables 6-11 and 6-12 summarize the construction, operation, and maintenance costs of alternatives D and E, respectively. Readers may refer to Appendix C for more information on this subject.

The costs of alternatives D and E for cost-effectiveness analysis include capital construction costs and annual costs for operation and maintenance. The wastewater treatment systems are evaluated for cost-effectiveness over the planning period as defined in the Municipal Wastewater Treatment Works Construction Grants Program (see Sec. 2-6-4).[3]

The procedure for cost comparison of alternatives includes determination of the present worth and equivalent annual costs for each project alternative.[18] The present worth may be thought of as the sum that, if invested now at a given rate, would provide exactly the funds required to make all necessary expenditures during the planning period. Equivalent annual cost is the expression of a nonuniform series of expenditures used as a uniform annual amount to simplify calculations of present worth. Detailed procedures for making these calculations are well known and explained in many textbooks.[19,20] General procedure is provided in Table 6-13. The interest rates for formulation and evaluation of construction grants projects are published by EPA. These are the *Discount Rates* and are periodically revised and published in different sources. The interest rate for FY 1996, which applies to all facility plans, is 7.652 percent.[21] In addition, the concerned state agencies also publish the new developments in the SRF pro-

Text continued on page 138.

[h]These cost estimates were made in May 1996, and at that time available ENR cost Index was until April 1996. The final submission date for the facility plan was January 31, 1998 (see Sec. 6-2).

TABLE 6-11 Estimated Construction, and Operation and Maintenance Costs of Alternative D

System Components and Description	Service (years)	Construction Cost (millions of dollars)		Annual O&M Cost (thousand of dollars)	
		1996[a]	1998[b]	1996[c]	1998[b]
A. 91-cm (36-in.) gravity diversion sewer to divert from stabilization ponds to aerated lagoon. Total trunk length 3.5 km (2.2 miles), average design flow 192 L/s (4.4 mgd) [Eqs (C-1) and (C-2)]	50	2.447	2.477	3.9	4.0
Engineering and construction-supervision 15 percent, and contingencies 15 percent (total = 30 percent)	—	0.734	0.743	—	—
Total costs of gravity diversion sewer	—	3.181	3.220	3.9	4.0
B. Renovation of Trickling filter plant					
Design average and peak flows are 132 L/s (3 mgd) and 385 L/s (8.75 mgd), respectively.					
Lift station (renovation)	15	0.664[d]	0.672	49.5[d]	50.1
Chemical addition with ferric chloride [Eqs (C-11) and (C-12)]	20	0.086	0.087	132.8	134.4
Preliminary treatment (new) without bar screen [Eqs (C-7) and (C-8)]	30	0.168	0.170	36.0	36.5
Primary sedimentation (new) [Eqs (C-9) and (C-10)], $Q_E = Q_{DESIGN}$	50	0.438	0.443	30.7	31.0
Trickling filter to achieve complete nitrification (renovation)	50	0.828	0.839	31.5	31.9
Denitrification with separate clarifier (new) [Eqs (C-25) and (C-26)]	40	0.907	0.918	227.2	230.0
Final clarifier (renovation)	40	0.698	0.706	65.3	66.1
Chlorination (renovation)	15	0.158	0.160	51.8	52.4
Dechlorination (new) [Eqs (C-33) and (C-34)], $Q_E = Q_{DESIGN}$	15	0.064	0.065	26.0	26.3
Gravity thickener, combined sludge (primary + trickling filter)	50	0.151[e]	0.153	7.65[e]	7.75
Aerobic digester (new), [Eqs (C-41) and (C-42)], $Q_E = Q_{DESIGN}$	50	0.476	0.482	44.4	45.0
Drying beds (renovation)	20	0.180	0.182	39.4	39.88
Landfilling of sludge (renovation)	20	0.079	0.080	38.3	38.74

Miscellaneous structure, administrative offices, laboratories, shops, and garage (renovation)	50	0.293	0.296	83.3	84.32
Support personnel	—	—	—	6.75	6.84
Subtotal, Cost of trickling filter plant	—	5.19	5.253	870.6	881.2
Piping 10 percent, electrical 8 percent, instrumentation 5 percent, and site preparation 5 percent (total = 28 percent)	—	1.453	1.471	—	—
Subtotal	—	6.643	6.724	870.6	881.2
Engineering and construction-supervision 15 percent, and contingencies 15 percent (total = 30 percent)	—	1.993	2.017	—	—
Total cost of trickling filter plant	—	8.636	8.741	870.6	881.2
C. Construction of new activated sludge treatment facility at aerated lagoon site: design average and peak flows are 308 L/s (7 mgd) and 921 L/s (21 mgd), respectively					
Lift station, [Eqs (C-3) and (C-4)], $Q_E = 5\ Q_{DESIGN}$	15	4.132[f]	4.182	125.0[f]	126.5
Preliminary treatment, [Eqs (C-7) and (C-8)]	30	0.282	0.286	50.6	51.2
Primary treatment, [Eqs (C-9) and (C-10)], $Q_E = Q_{DESIGN}$	50	0.720	0.728	56.3	57.0
Anaerobic, anoxic and aerobic system[g], [Eqs (C-13) and (C-14)]	30	2.969	3.01	205.6	208.1
Final Clarifier, [Eqs (C-17) and (C-18)], $Q_E = Q_{DESIGN}$	40	1.457	1.474	94.2	95.3
Chlorination, [Eqs (C-31) and (C-32)], $Q_E = Q_{DESIGN}$	15	0.352	0.356	86.9	87.97
Dechlorination, [Eqs (C-33) and (C-34)], $Q_E = Q_{DESIGN}$	15	0.093	0.094	40.2	40.65
Gravity thickener, combined sludge (primary + waste activated sludge), [Eqs (C-39) and (C-40)], $Q_E = 1.53\ Q_{DESIGN}$	50	0.241[h]	0.244	10.97	11.1
Anaerobic digester, [Eqs (C-43) and (C-44)], $Q_E = Q_{DESIGN}$	50	1.024	1.036	44.6	45.12
Belt filter press, [Eqs (C-47) and (C-48)], $Q_E = Q_{DESIGN}$	15	1.378	1.395	109.7	111.1
Landfilling of sludge, [Eqs (C-49) and (C-50)], $Q_E = Q_{DESIGN}$	20	0.146	0.148	53.9	54.6
Miscellaneous structures, administrative offices, laboratories, shops, and garage, [Eqs (C-53) and (C-54)]	50	0.464	0.470	107.6	108.96
Support personnel	—	—	—	12.37	12.52

TABLE 6-11 *Estimated Construction, and Operation and Maintenance Costs of Alternative D—cont'd*

System Components and Description	Service (years)	Construction Cost (millions of dollars) 1996[a]	Construction Cost (millions of dollars) 1998[b]	Annual O&M Cost (thousand of dollars) 1996[c]	Annual O&M Cost (thousand of dollars) 1998[b]
Subtotal 1, Cost of activated sludge plant		13.258	13.423	997.94	1010.12
Piping 10 percent, electrical 8 percent, instrumentation 5 percent, and site preparation 5 percent (total = 28 percent)		3.712	3.758	—	—
Subtotal 2	—	16.97	17.181	997.94	1010.12
Engineering and construction supervision 15 percent, contingencies 15 percent (total = 30 percent)	—	5.091	5.154	—	—
Total cost of activated sludge plant	—	22.061	22.335	997.94	1010.12
Total cost of alternative D	—	33.878	34.296	1872.44	1895.32

[a]Capital costs for new units are obtained from the cost equations given in Appendix C. Cost of unit renovation are developed from engineering judgment.
[b]Costs 1998 = cost 1996 × ENR Index 1998/ENR Index 1996; ENR Index May 1996 = 5572, the estimated ENR Index 1998 = 5640.
[c]O&M costs are obtained from cost equations given in Appendix C.
[d]Renovation cost is lump sum. For O&M cost, Q_E = (TDH 11.5 m/3.05 m) × Q_{DESIGN} = 3.77 Q_{DESIGN} [see Eq. (C-4)].
[e]Q_E for combined sludges is obtained from thickener loading of 43 $kg/m^2 \cdot d$ (8.8 b/sq ft·d) and design sludge mass of 0.22 kg/m^3 (1844 lb/10^6 gal), Q_E = 1.53 Q_{DESIGN} [see Eqs (C-39) and (C-40)].
[f]Q_E = (TDH 15.25 m/3.05 m) × Q_{DESIGN} = 5.0 Q_{DESIGN}.
[g]Capital and O & M costs are assumed 30% higher than that of conventional activated sludge process.
[h]Q_E for primary and waste-activated sludge is obtained from thickeners loading of 43 $kg/m^2 \cdot d$ (8.8 lb/sq ft·d) and design sludge mass of 0.22 kg/m^3 (1844 lb/10^6 gal), Q_E = 1.53 Q_{DESIGN} (see Eq. C-11).

TABLE 6-12 Estimated Construction, and Operation and Maintenance Costs of Alternative E

System Components and Description	Service (years)	Construction Cost (millions of dollars)		Annual O&M Cost (thousand of dollars)	
		1996[a]	1998[b]	1996[a]	1998[b]
A. 91-cm (36-in.) gravity diversion sewer to divert from stabilization ponds to aerated lagoon. Total trunk length 3.5 km (2.2 miles), average design flow 192 L/s (4.4 mgd) [Eqs (C-1) and (C-2)]	50	2.447	2.477	3.9	4.0
B. 76 cm (30 in.) gravity diversion sewer to divert flow from trickling filter plant to aerated lagoon. Total trunk length 2.9 km (1.8 miles), average design flow of 132 L/s (3 mgd)	50	1.639	1.659	2.9	2.9
Subtotal of gravity diversion sewer	—	4.086	4.136	6.8	6.9
Engineering and construction supervision 15 percent, and contingencies 15 percent (total = 30 percent)	—	1.226	1.241	—	—
Total costs of gravity diversion sewer	—	5.312	5.377	6.8	6.9
C. New Activated Sludge plant:					
Design average and peak flows are 440 L/s (10 mgd) and 1320 L/s (30 mgd), respectively.					
Lift station, [Eqs (C-3) and (C-4)]	15	5.334[c]	5.399	170.5[c]	172.6
Preliminary treatment, [Eqs (C-7) and (C-8)]	30	0.352	0.356	61.5	62.6
Primary sedimentation, [Eqs (C-9) and (C-10)], $Q_E = 0.92\ Q_{\mathrm{DESIGN}}$	50	0.871	0.882	70.5	71.4
Anaerobic-anoxic-aerobic system[d], [Eqs (C-13) and (C-14)]	30	4.037	4.086	273.5	276.8
Final clarifier, [Eqs (C-17) and (C-18)], $Q_E = 1.91\ Q_{\mathrm{DESIGN}}$	40	2.684	2.717	246.9	249.9
UV disinfection, Eqs (C-35) and (C-36)]	15	0.550	0.557	39.7	40.2
Gravity thickener, combiner sludge (primary + waste-activated sludge), [Eqs (C-39) and (C-40)], $Q_E = 1.84\ Q_{\mathrm{DESIGN}}$	50	0.349[e]	0.353	15.3[e]	15.5
Anaerobic digester, [Eqs (C-43) and (C-44)] $Q_E = 0.95\ Q_{\mathrm{DESIGN}}$	50	1.214	1.229	50.9	51.5

Continued

TABLE 6-12 Estimated Construction, and Operation and Maintenance Costs of Alternative E—cont'd

System Components and Description	Service (years)	Construction Cost (millions of dollars)		Annual O&M Cost (thousand of dollars)	
		1996[a]	1998[b]	1996[a]	1998[b]
Filter press, [Eqs (C-47) and (C-48)], $Q_E = 1.22\ Q_{DESIGN}$	15	1.80	1.822	133.1	134.7
Biosolids recycle, [Eqs (C-51) and (C-52)], $Q_E = 0.71\ Q_{DESIGN}$	20	0.128	0.130	47.7	48.3
Landfilling of residues[f]	20	0.013	0.013	4.77	4.83
Miscellaneous structures, administrative offices, laboratories, shops, and garage, [Eqs (C-53) and (C-54)]	50	0.568	0.575	127.9	129.4
Support personnel	—	—	—	15.97	16.16
Subtotal 1, cost of activated sludge plant	—	17.9	18.119	1258.24	1273.89
Piping, 10 percent, electrical 8 percent, instrumentation 5 percent, and site preparation 5 percent, (total = 28 percent)	—	5.012	5.073	—	—
Subtotal 2	—	22.912	23.192	1258.24	1273.89
Engineering and construction supervision 15 percent, and contingencies 15 percent (total = 30 percent)	—	6.874	6.958	—	—
Total cost of activated sludge plant	—	29.786	30.150	1258.24	1273.89
Total cost of alternative E	—	35.098	35.527	1265.04	1280.79

[a]Capital and O&M costs are obtained from the cost curves in Appendix C.

[b]Cost of 1998 = cost of 1996 × $\frac{\text{ENR Index 1998}}{\text{ENR Index 1995}}$; ENR Index May 1996 = 5572; ENR Index 1998 = 5640.

[c]$Q_E = \frac{\text{TDH 11.5 m}}{\text{3.05 m}} \times Q_{DESIGN} = 5.0\ Q_{DESIGN}$

[d]The capital and O&M costs are assumed 30% higher than that of conventional activated sludge plant.

[e]Q_E for combined sludge is obtained from thickeners loading of 43.7 kg/m^2·d (8.8 lb/sq ft·d) and design sludge mass of 0.269 kg/m^3 (1844 lb/10^6 gal), $Q_E = 1.84\ Q_{DESIGN}$ [see Eqs (C-39) and (C-40)].

[f]The screening, grit, scum and any unused or left over biosolids shall be landfilled. Cost is based on engineering judgment.

TABLE 6-13 Present Worth and Equivalent Annual Costs of Alternatives D and E

Alternatives	Total Capital Cost 1998 (millions of dollars)	Total Annual O&M 1998 (thousands of dollars)	Present Worth of Annual O&M Cost (millions of dollars)	Present Worth (millions of dollars)	Equivalent Annual Cost (millions of dollars)
D	34.296	1895.32	16.573[a]	50.869[b]	5.82[c]
E	35.527	1280.79	11.199	46.726	5.345

[a]Interest rate 7.652 percent, planning 15 years, present value of annuity = $[1 - (1 + i)^{-n}]/i = [1 - (1 + 0.07652)^{-15}]/0.07652 = 8.744$. Present worth of annual O&M cost = $(1895.32 \times 8.744/1000) = 16.578$ millions of dollars.

[b]Present worth = $16.573 + 34.296 = 50.869$ million dollars.

[c]Capital recovery factor = $[i\,(1 + i)^n/(1 + i)^n - 1] = 0.07652\,(1.07652)^{15}/[(1.07652)^{15} - 1] = 0.1144$. Equivalent annual cost = $50.869 \times 0.1144 = 5.82$ million dollars.

gram, which also includes available interest rates for project planning. It may be noted that the estimated ENR Index for 1998 and the discount interest rate for FY 1996 are used in these calculations. It is assumed that the discount interest rate will remain the same. The procedure is only for illustration. Design engineers must use the actual ENR Index and the discount interest rate for the same year for cost estimates.

The present worth and equivalent annual costs of alternatives D and E are summarized in Table 6-13. These costs must be updated prior to starting the construction program.

Effectiveness Evaluation of Alternatives. Economic analyses of wastewater treatment facilities were traditionally based on cost considerations. These evaluations utilized total costs incurred and any savings or benefits that could be expressed in monetary units. It has been recognized that combining costs and benefits into single measures will not necessarily indicate the most economically efficient alternative. Consequently, interest has been developed in evaluating the alternatives on effectiveness and then comparing with system economics.[10] Alternatives D and E have been evaluated in terms of six measures of effectiveness. Many parameters have been grouped together to simplify the analysis. The measures of effectiveness are presented in Table 6-14.

Evaluation Procedure. To rate the effectiveness, each alternative was scored for every measure of effectiveness by assigning a number 0–4. The scores 0 and 4 indicated low and high effectiveness ratings, respectively. Since the measures of effectiveness are not of equal importance, a relative weight was also assigned to each measure of effectiveness. The relative weights varied from 1 to 3, with 3 indicating extremely important. The effectiveness scores (0 to 4) were then multiplied by the relative weights. The final number was then used in the matrix evaluation. Finally, the weighted scores for each alternative were added to give the overall scores for comparison purposes. Such matrix evaluation for two alternatives is given in Table 6-15.

The assignment of relative weights to each measure of effectiveness and the rating of each alternative for different measures of effectiveness should be performed independently by a team of at least three professionals. Average values should be used to minimize the preferences and biases of the individuals.

Alternative Selection. The economic comparison of alternatives D and E indicate that alternative E is slightly cheaper than alternative D. However, the effectiveness evaluation of these alternatives in terms of six measures of effectiveness (Table 6-14) indicate that the alternative E is considerably superior to alternative D. These measures of effectiveness include factors such as site conditions, effluent quality, environmental considerations, regionalization, future expansion, energy conservation, operational flexibility and reliability, system compatibility, and sludge handling and disposal. Based on the results of cost-effectiveness evaluation, alternative E is more cost-effective and has been selected for implementation.

TABLE 6-14 Measures of Effectiveness Used in Effectiveness Evaluation of Alternatives D and E

Measures of Effectiveness	Description
Plant location	Wastewater treatment plant location should be sensitive to constraints imposed by system design, land cost, and availability for present construction and future expansion, and the nature of the surrounding developments both existing and planned. In this respect, alternative D utilizes two plant locations, while alternative E provides a single site at a remote location. Therefore, alternative E is rated higher.
System compatibility	Facilities that have similar and compatible multiple-treatment processes reduce system complexity and operational skills to operate, maintain, and minimize routine daily monitoring activities and reduce the available inventory of spare parts and equipment. Alternative D utilizes two independent treatment processes, namely, activated sludge and trickling filters. Therefore, it is rated lower than alternative E in this category.
Operational flexibility and reliability	Single-treatment process under shock loadings or upset conditions may produce inferior effluent quality. Multiple treatment processes, such as in alternative D, provide a backup system if one process is subjected to operational difficulties. For this reason in this category, alternative D is rated higher than alternative E.
Environmental quality	Environmental quality control is important for health and aesthetic reasons, which in turn directly affect public opinion and concern. Items of concern here are odors, effluent quality, and surface and subsurface pollution. Trickling filters, in general, provide inferior effluent quality and are also associated with odor problems. Therefore, alternative D is rated lower in this category.
Power requirement	Energy needed to operate a given system is included in power requirements. Low power requirements will reflect favorably in the cost analysis, but since power is a limited resource, low power requirements also contain dimensions that relate purely to effectiveness. Power is needed for pumping and treatment equipment. Trickling filters require lesser power than the activated sludge process to operate; however, the pumping head is greater at a trickling filter plant. Alternative D utilizes two pumping stations as compared with one pumphouse in alternative E. Aerobic sludge digestion for a portion of sludge in alternative D would require higher energy demand and will not yield methane. For these reasons, it is expected that alternative E will have less energy demand.
Sludge handling and disposal	Sludge from wastewater treatment facilities can cause serious odor problems if not handled properly. Alternative D provides sludge drying beds at the trickling filter plant and therefore may cause odor problems. Therefore, alternative E is rated higher in this category.

TABLE 6-15 Matrix Evaluation of Wastewater Treatment Alternatives

Measure of Effectiveness (1)	Assigned Weights to Each Measure of Effectiveness (2)	Alternative D Score (3)	Alternative D Weighted Score (4)	Alternative E Score (5)	Alternative E Weighted Score (6)
Plant location	3.0[a]	2.5[b]	7.5[c]	4.0[b]	12.0[d]
System compatibility	2.0	2.5	5.0	4.0	8.0
Operational flexibility and reliability	1.0	3.0	3.0	2.0	2.0
Environmental quality	3.0[a]	3.0	9.0	4.0	12.0
Power requirements	2.5	3.0	7.5	3.0	7.5
Sludge handling	2.5	3.0	7.5	4.0	10.0
Total			39.5		51.5

[a]Plant location and environmental quality have been assigned a weight of 3.0, indicating that these measures of effectiveness are extremely important.
[b]Alternatives D and E, when compared for plant location, received scores of 2.5 and 4.0, respectively, indicating that alternative E ranks much higher in this measure of effectiveness than alternative D.
[c]Product of values in columns (2) and (3).
[d]Product of values in columns (2) and (5).

6-9 FINANCIAL STATUS

The city has the legal, institutional, managerial, and financial capability to ensure adequate funds for construction, operation, and maintenance of the project proposed in this facility plan. The city is currently charging a fee on the users of water and sewer service. The sewer service rates are based on sewage flow and strength. The financial and statistical information about the city is normally provided in an attachment to the facility plan (see Sec. 6-15).

6-10 DESCRIPTION OF SELECTED WASTEWATER TREATMENT FACILITY

6-10-1 Design Considerations

The schematic flow diagram of the selected wastewater treatment facility is shown in Figure 6-4. The process train employs bar screen, pump station, grit removal, primary sedimentation, biological nutrient removal, UV disinfection, and effluent disposal by dilution. The sludge is thickened, digested, and dewatered. The biosolids are used over agricultural land. The scum, screenings, grit, and unused biosolids are landfilled. A summary of the design criteria and unit sizes of various collection and treatment systems is an essential part of a facility plan. Such information is given in Chapter 23. Design analysis and summary of design computations are normally submitted as an appendix to a facility plan (see Sec. 6-15).

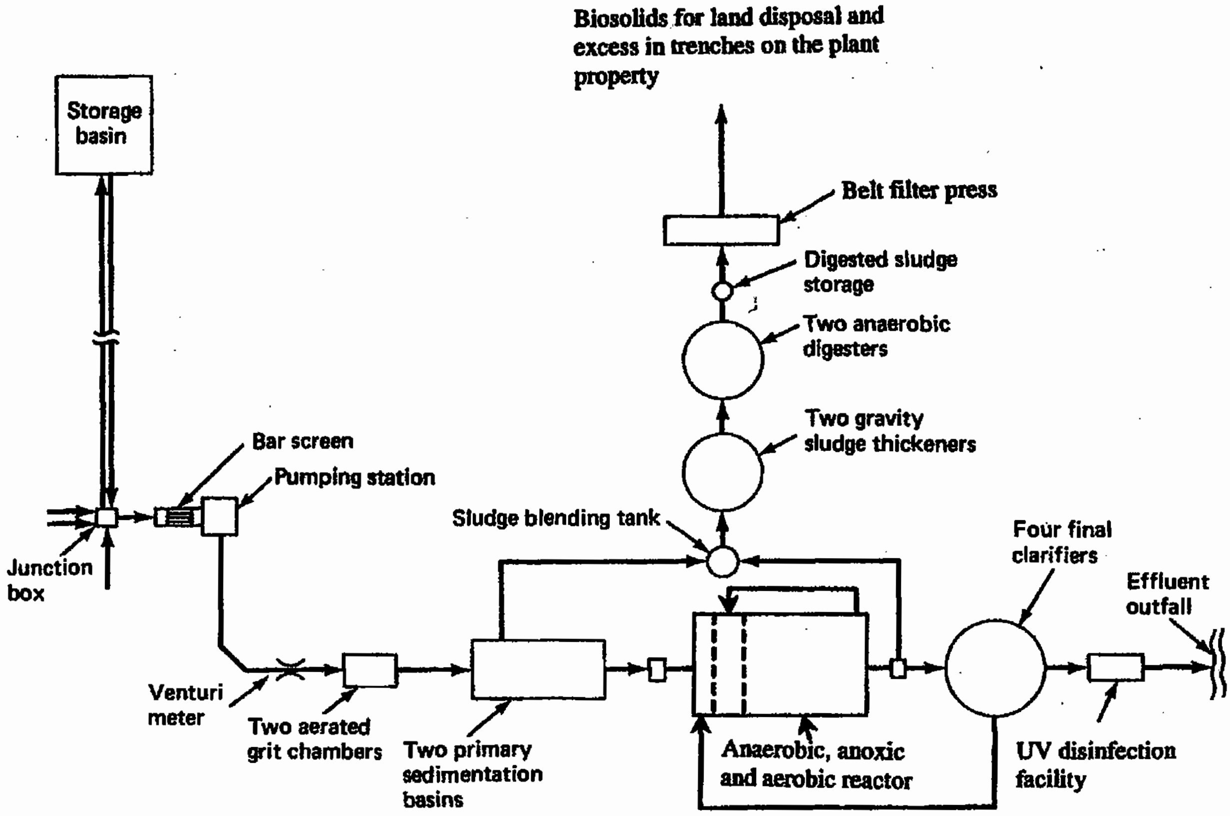

Figure 6-4 Schematic Flow Diagram of the Proposed Wastewater Treatment Plant.

6-10-2 Operation and Maintenance

The treatment facility will be operated by trained licensed operators on a 7-d-per-week basis. Supporting staff, including mechanics and electricians on a 24-h basis, will be provided. A laboratory facility for quality control, an administration building, and equipment maintenance shops will also be provided.

Each treatment unit will have an outlet for chlorine solution to wash the walls and weirs and remove the scum. General discussion on operation and maintenance requirements for the treatment facility is provided in subsequent chapters along with the detailed design and equipment specifications for each unit.

Control of Bypassing. The power supply in the area is very reliable. Interruption records for 8 years at the treatment plants revealed four power outages ranging in duration from 10 min to 1 h. Prolonged power failure is unlikely. Auxiliary power generators and multiple electrical primary power lines from different sources are considered unnecessary. In order to protect against overflow and subsequent stream pollution, the existing aerated lagoons will be used as storage ponds. An overflow structure shall be constructed to direct raw wastewater into the lagoons for storage. Existing lagoon volume will provide storage for approximately 35 h. The raw wastewater stored in the lagoon will be routed through the plant after restoration of power.

All treatment processes have been provided with multiple units, and pumps and blowers have sufficient standby units in case equipment breakdown occurs.

The proposed treatment facility will not tie into any existing system until the completion of the interceptors, pumping station, and treatment plant. After construction of the entire facility, the necessary connections will be made to the existing sewerage system to divert the wastewater to the new facility.

Flood Hazard Analysis. All treatment units except bar screen and pumping station will be constructed above ground level. A levee will be constructed around the entire plant area to at least 0.5 m above the 100-yr flood for protection against flooding. Provisions will be made for draining the area surrounded by the levees.

User's Charge. User's charge schedule and industrial cost recovery system shall be implemented in accordance with the federal and state guidelines.

6-11 ENVIRONMENTAL IMPACT ASSESSMENT

An environment impact assessment report of the project was prepared as part of the state Revolving Fund Loan application for the proposed project. This application is attached as an appendix to a facility plan. Only basic information pertinent to the environmental impact assessment is presented in the main text of this facility plan. The proposed project will not significantly impact current or future land uses, natural drainage, energy consumption, aquatic life, or water quality and supply, nor will it promote erosion. Land clearing or disruption of wildlife habitat will not be required. The detailed discussions on many specific areas of the environment are addressed in the environmental impact assessment report. It is expected that the proposed project will not have any significant primary adverse impacts on the following areas of the environmental concern:

- floodplains and wetlands
- plant and animal communities
- ecosystem
- threatened or endangered species
- unique or vulnerable environmental features
- unique archeological, historic, scientific, or cultural resources, parks, or stream corridors
- community growth pattern and land use
- air quality
- scenic views and aesthetics

Many federal and state agencies were contacted to review and comment on the proposed project for environmental impacts. These agencies included the U.S. Fish and Wildlife Service, State Parks and Wildlife Department, U.S. Soil Conservation Service, U.S. Army Corps of Engineers, State Air Control Board, State Water and Environment Control Board, and the State Historic Preservation Officer. Comments with these agencies revealed no significant environmental impacts associated with the project since the proposed improvements will be maintained in the existing property of the wastewater treatment plant utilizing aerated lagoons.

The proposed treatment facilities are expected to reduce health hazards, enhance surface water quality in the receiving stream, eliminate groundwater pollution potential, and aid in orderly development for the area served. The proposed action presents beneficial long-term use of the environment. It will achieve the goal of providing adequate wastewater treatment capacity for the planning area in order to maintain water quality standards in the receiving waters.

The city owns a large tract of land bounded by two streams (Fig. 6-2). Approximately 125 hectare will be allocated to the plant property. No relocation of people or existing structures will be required for the project. Also, no change in the value of the adjacent property is anticipated as a result of this project.

The minor adverse effects that cannot be avoided are those normally associated with the existence and operation of wastewater treatment facilities. The increased noise levels and possible occasional odors emanating from the facility will be minimized by modern design techniques and efficient plant operation. Some degree of disruption of the environment and inconveniences to citizens during construction of interceptors is unavoidable but will be reduced in severity by proper construction scheduling and techniques. At the plant site, short-term impacts to be mitigated during construction of the proposed project include dust production, noise level, and traffic disruption.

Fuels and other forms of energy will be utilized during construction and operation of facilities. Construction materials will be used for the project and remain permanently. Chemicals such as chlorine, sulfur dioxide, lime, ferric chloride, and organic polymers will be needed for normal plant operation. The land for treatment facilities will be permanently lost to agricultural production. However, the productivity of the area in current use as an aerated lagoon is minimal.

6-12 PUBLIC PARTICIPATION PROGRAM

During the entire facility-planning process, an active public participation program was maintained. The program utilized public meetings, workshops, mailouts, and press releases to keep the public aware of the progress and to receive their comments before any decisions were made. Various appointed environmental advisory committees held public meetings to review the work. Notices of each meeting were mailed to the interested citizens and were published in the newspapers at least 45 days prior to each meeting date. Documents considered at that meeting were placed for public review at the library, city hall, and the chamber of commerce at least 30 days before the meeting.[22]

The entire facility-planning process lasted for 57 weeks, as was originally proposed during the initial planning. The public participation work program that was conducted in relation to the project decision points is summarized in Table 6-16.

The final meeting is a public hearing, and all public comments are included in the future application to conduct the engineering and design of the improvements committed in the facility plan.

6-13 COST BREAKDOWN

The cost of performing facility planning and estimated cost for preparation of design plans and specifications are given in Tables 6-17 and 6-18. The cost estimates for prepa-

TABLE 6-16 Public Participation Work Program Conducted During Facility Planning Process

Decision Point	Tasks	Time Schedule in Weeks Since Start of Facility Planning	Target Audience
Preparation of public participation program since the start of facility planning	Select engineer, designate public liaison; develop mailing list, public participation work program, and fact sheet; and designate depository for key documents (public library, chamber of commerce, city hall).	1–6	
	Distribute public participation work program and fact sheet to media, depository, and mailing list.	7–8	General public
Assessment of present and future conditions	Send notice of proposed public meeting (press releases and mailout) 45 days prior to meeting.	11–12	General public
	Place relevant documents in designated depository for public review 30 days before meeting.	13–14	General public
	Contact planning staff and chairpersons of various committees for their advice on key issues, water quality problems, and public participation.	16–17	Planning staff and chairpersons of various committees
	Conduct the workshop with the city council and the environmental advisory committees, and interested citizens (representing neighborhood organizations) to explain major water quality problems, planning process, problems and issues, and community goals and objectives.	18–22	General public
	Publish news releases in local news papers to describe local water quality problems, objectives, goals, issues, and public participation program.	24–25	General public
	Briefings at various local group meetings.	26–28	Local citizen groups

	Prepare agency responsiveness on public comments and place at designated depository.	32–37	EPA and state
Development of alternatives	Prepare the fact sheets describing alternatives, environmental impacts, and cost of each alternative. Distribute fact sheets and notice of upcoming public meeting 45 days prior to the meeting (use the news media and mailing list).	37–39	General public
	Place relevant documents in designated depository for public review 30 days before public meeting.	40–41	General public
	Informal public meeting of various environmental advisory committees to discuss alternatives, answer questions, and identify areas of further investigation.	44–45	General public
Development and submission of final plan to the city	Develop fact sheet to highlight major elements of proposed plan, distribute fact sheet to all on mailing list, and send notice of public hearing in local newspaper and send to all on mailing list (45 days prior to meeting).	48–50	General public
	Place relevant documents in designated depository for public review 30 days before public hearing.	51–52	General public
	Conduct joint meeting of environmental review committees to present final plans and draft EIS.	54–55	General public, key citizens and officials
	Conduct public hearing.	55	General public
	Prepare final responsiveness summary outlining grantees response and evaluation of public hearing to citizen input. Place on file at designated depository.	56	EPA and state
	Publish notice of final decision with press releases and mailout.	57	General public

TABLE 6-17 Cost of Preparing Facility Plan

Item	Description	Estimated Cost[a] (Dollars)
1	Infiltration/inflow analysis	48,000
2	Environmental impact assessment	28,000
3	Facility plans	206,940
4	Subtotal	282,940
5	Legal and administrative costs[b]	2,830
6	Contingencies[c]	14,150
7	Total estimated cost of facility planning and report preparation, revision and submission	299,920

[a]These are estimated costs based on engineering judgment and information given in Ref. 24.
[b]Legal and administrative costs 1 percent of subtotal.
[c]Contingencies 5 percent of subtotal.
Total allowance for preparation of the facility plan will be included in the funding from the SRF Program.

ration of plans and specifications are developed using EPA Region VI guide.[23,24] It is estimated that 100 drawings will be needed for detailing the wastewater treatment facility, and 30 drawings will be needed for the design and details of diversion sewers. Methods for estimating fees for professional engineer services for various types of projects are given in Ref. 24. The estimated costs of the proposed project are summarized in Table 6-19.

TABLE 6-18 Estimated Cost for Preparation of Design Plans and Specifications

Item	Description	Estimated Cost (Dollars)[a]
1	Proposed diversion sewers	360,850
2	Proposed wastewater treatment plant	2,162,720
3	Subtotal	2,523,570
4	Legal and administration[b]	25,230
5	Contingencies[c]	126,200
6	Total estimated cost of preparation of design plans and specifications	2,675,000

[a]These are estimated costs based on engineering judgment and information given in Ref. 24.
[b]Legal and administrative costs 1 percent of subtotal.
[c]Contingencies 5 percent of subtotal.
Total allowance for preparation of the design plans and specifications will be included in the funding for SRF Program.

TABLE 6-19 Estimated Cost of the Project

Item	Description	Estimated Cost (Dollars)[a]
1	Proposed diversion sewers	5,377,000
2	Treatment including pumping station	30,150,000
3	Total estimated construction cost	35,527,000
4	Preparation of Facility Plan	299,920
5	Preparation of Design Plans, and specification and contract documents	2,675,000
6	Total Project cost	38,501,920

[a]Construction cost of the project must be updated prior to initiating the construction phase of the project. Total estimated eligible cost of the project will be financed by the SRF program.

6-14 TIME SCHEDULE FOR PROJECT MILESTONES

The project milestones for preparing design plans and specifications and construction programs are summarized in Tables 6-20 and 6-21. The scheduling of starting and completion periods given in these tables are only tentative.

6-15 ATTACHMENTS

The following information is normally submitted as separate attachments or appendixes to a facility plan.

1. "Letter of clearance" on project review in conformance with OMB circular from the clearinghouse
2. Infiltration/inflow report
3. Design analysis and summary of design computations
4. Environmental impact assessment
5. Effluent discharge permits (NPDES permit)
6. Correspondence from agencies

TABLE 6-20 Schedule of Starting and Completion of Design Plans and Specifications

Project Schedule	Estimated Action Time (d)
Initiated after authorization to proceed with design plans and specifications	30
Completed	200
Submitted to the state agency	15
Total	245

TABLE 6-21 Schedule of Starting and Completion of Construction

Project Schedule	Estimated Action Time (d)
Advertise for construction bids within the days indicated after authorization to advertise for bids.	30
Submit bid opening documents within the days indicated after the bid opening date.	30
Award construction contract within the days indicated after authorization to award the contract.	15
Initiate construction (within the days indicated) after the award of the contract.	30
Complete construction within the days indicated after initiation of construction.	480
Total	585

7. The review comments or approval of relevant state and local agencies indicating compatibility with state and local plans
8. Notice of public hearings, including a brief summary of the public hearings held to consider the proposed project
9. A statement demonstrating that the applicant has necessary legal, technical, financial, institutional, and managerial resources to ensure construction, operation, and maintenance of the proposed treatment works
10. Statement specifying that the requirements on Title VI of the Civil Rights Act of 1964 have been satisfied within the facility plan

6-16 PROBLEMS AND DISCUSSION TOPICS

6-1 From the 1980 and 1990 population data given in Table 6-2, estimate the population for the initial year (1995). Use geometric rate of increase.

6-2 Calculate the population in the design year 2015 for the data given in Table 6-2. Use logistic curve fitting and decreasing rate of increase methods.

6-3 A service area has a total population of 24,000 residents. The total length of the sewerage system is 15 m per resident. The typical length distribution of average sewer sizes are 20 cm, 70 percent; 38 cm, 15 percent; 50 cm, 9 percent; and 60 cm, 6 percent. Calculate the equivalent pipe diameter. If infiltration/inflow is 1400 liters per d per cm per km, express the infiltration/inflow in Lpcd.

6-4 If the service area in Problem 6-3 is 1600 ha, calculate infiltration/inflow contribution in m^3 per ha per d.

6-5 The existing wastewater treatment facilities for Modeltown are described in Table 6-4. These include stabilization ponds, trickling filter plant, and an aerated lagoon. Using the dimensions for these facilities and flow and wastewater characteristics given in Tables 6-4 and 6-6, calculate the following:

(a) Detention time (days) and organic loading (kg/ha·d) to the stabilization ponds

(b) Hydraulic loading (m^3/m^2·d) and organic loading (kg/m^3·d) to the trickling filters

(c) Aeration period and organic loading to the aerated lagoon. Assume side slope 4 horizontal to 1 vertical. Also calculate the oxygen transferred by the floating aerators in g oxygen per h per kW. Effluent BOD_5 = 20 mg/L, and BOD_5/BOD_L ratio = 0.68.

6-6 In option D the trickling filter plant is renovated to include nutrient removal. Draw the complete process train. Comment on the facilities that will provide nutrient removal.

6-7 In option D if the trickling filter plant is renovated as a conventional secondary plant followed by a constructed wetland (see Chapter 24) for nutrient removal and polishing of effluent. Draw the process train and comment on the feasibility of this option.

6-8 A conventional wastewater treatment plant was designed for an average flow of 19,000 m^3/d. The process train includes preliminary treatment, low lift pump station, primary sedimentation, conventional activated sludge, final clarifier, and chlorination and dechlorination facilities. Develop the construction cost of the plant and annual operation and maintenance costs. Add in the construction costs: 28% for piping, electrical, and site preparation, and 30% for engineering and construction supervision and contingencies. The sludge treatment costs are not included in this project.

6-9 A wastewater treatment plant was designed for a flow of 11,400 m^3/d (3.0 mgd). The construction cost of the facility is \$8.0 million. The annual O&M cost is \$300,000. Calculate the unit treatment cost in \$/m^3 and \$/1000 gallons. Amortize the construction cost over 20 years of useful life at an interest rate of 7 percent. Use capital recovery factor equation given in footnote of Table 6-13.

6-10 Visit your local wastewater treatment facility. Determine which year the facility was expanded. Obtain a copy of the facility plan. Review the facility plan and the accompanying attachments (see Sec. 6-15). Identify the following:

(a) How much infiltration/inflow was reaching the plant? Was it excessive? Was a rehabilitation survey needed?

(b) How many alternatives were evaluated? Draw a process diagram for each alternative.

(c) In your opinion, is the selected alternative most cost-effective?

(d) Was an environmental impact report or statement (EIR or EIS) prepared? Were the major environmental factors addressed? Have some controversial issues arisen since the completion of the construction?

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7

Design of Intercepting Sewers

7-1 INTRODUCTION

Sewers are underground conduits for conveying wastewater to the treatment facilities or to the point of disposal. There are three types of sewers: sanitary, storm, and combined. Sanitary sewers are designed to carry wastewater from residential, commercial, and industrial areas and a certain amount of infiltration/inflow that may enter the system because of deteriorated conditions of the sewers and manholes and roof conditions. Storm sewers are exclusively designed to carry the surface runoff. Combined sewers are designed to carry both the sanitary and storm flows. Combined sewers are undesirable because heavy precipitation may cause flows far exceeding the designed capacity of the treatment facilities. These days, combined sewers are seldom designed and constructed in the United States. A discussion on stormwater runoff and combined sewer overflow is given in Chapter 26.

The characteristics of municipal wastewater and volumes of infiltration/inflow (I/I) have been discussed extensively in Chapters 3 and 6. In this chapter an overview of the design and construction procedures of sanitary sewers is given. Furthermore, design examples covering step-by-step calculations, sewer profiles, construction details, and operation and maintenance of five sewer lines are also presented. Flow conditions in these lines were developed in the Model Facility Plan (Chapter 6). These sewer lines are (1) diversion sewer from the stabilization pond area, (2) diversion sewer from the trickling filter plant area, (3) existing intercepting sewer from the central part of the town, (4) final sewer line carrying the total flow from the junction box to the bar screen, and (5) the bypass sewer to direct the flow from the junction box to the storage basin during power outages when the pumping equipment is temporarily inoperable.

7-2 COMMUNITY SEWERAGE SYSTEM

The community sewerage system consists of (1) building sewers (also called building connections), (2) laterals or branch sewers, (3) collector or subcollector, (4) trunk sewers, and (5) intercepting sewers.

Building sewers connect the building plumbing to the laterals or to any other sewer lines mentioned above. Laterals or branch sewers convey the wastewater to the collector sewers. Several collector sewers connect to the interceptor sewers that convey the wastewater to the treatment plant. A typical layout of a municipal sewerage system is shown in Figure 7-1.

All sewers except the building sewers are constructed in streets, special easements, or available rights-of-way. They follow the natural ground elevations. The wastewater treatment facility is generally located near the low outskirts of the community if other conditions are acceptable. A tradeoff between the construction cost of laying deeper lines and the cost of lift stations and pumping costs must be established in determining the cost-effective sewerage system.

The diameter of a sewer line is generally determined from the peak flow that the line must carry and the local sewer regulations concerning the minimum sizes of the laterals and building connections. On the other hand, the total length of the sewer lines depends on the layout of the community. Based on the data for 97 cities in 21 states of the United States, the total average length of the sewerage system was 4.3–11.5 m per capita (14–38 ft per capita). The typical distribution of pipe sizes is summarized in Table 7-1.[1,2]

Pressure or vacuum sanitary sewers are also designed these days where gravity lines may not be economically feasible. Examples of such situations are lake communities, rocky terrain, and areas where groundwater table is high. Additional discussion on pressure and vacuum sewers may be found in Chapter 25 and in Refs. 3–6.

7-3 DESIGN AND CONSTRUCTION OF COMMUNITY SEWERAGE SYSTEM

Designing a community sewerage system is not a simple task. It requires considerable experience and a great deal of information to make proper decisions concerning the layout, sizing, and construction of a sewer network that is efficient and cost-effective. The design engineer needs to generally undertake the following tasks:

1. Define the service area.
2. Conduct preliminary investigations.
3. Develop preliminary layout plan and profile.
4. Review design and construction considerations.
5. Conduct field investigations and complete design and final profiles.
6. Prepare contract drawings and specifications.

Each of these tasks is discussed below.

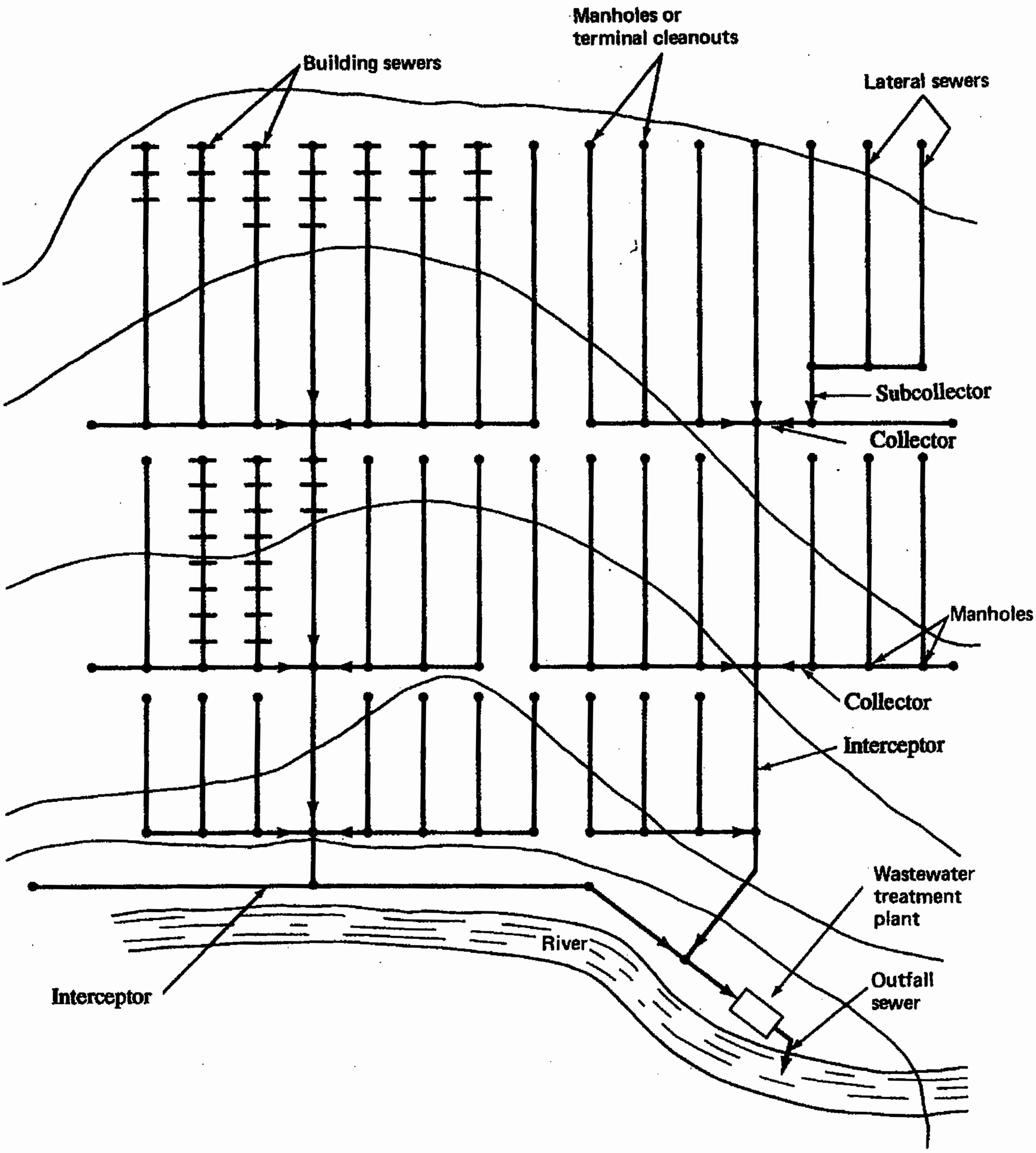

Figure 7-1 Sanitary Sewerage System.

7-3-1 Service Area

Service area or sewer district is defined as the total area that will eventually be served by the sewerage system. The service area may be based on natural drainage or political boundaries or both. It is generally a part of the areawide waste management plan. Additional discussion on the service area can be found in Chapter 2.

TABLE 7-1 Typical Distribution of Sewer Sizes

Sewer Diameter [cm (in.)]	Distribution (percent)
10, 12.5, 15 (4, 5, 6)	3.6
20 (8)	73.1
25, 30, 35, 37.5, 40 (10, 12, 14, 15, 16)	13.4
45, 50, 53 (18, 20, 21)	3.8
60 (24)	1.7
67.5 (27) and above	4.4
Total	100.0

Source: Adapted in part from Ref. 2.

7-3-2 Preliminary Investigations

The design engineer must conduct the preliminary investigations to develop a layout plan of the sewerage system. Site visits and contacts with the city and local planning agencies and state officials should be made to determine the land use plans, zoning regulations, and probable future changes that may affect both the developed and undeveloped land. Data must be developed on topography, geology, hydrology, climate, ecological elements, and social and economic conditions. Topographic maps with existing and proposed streets and other utility lines provide the most important information for preliminary flow routing. If reliable topographic maps are not available, field investigations must be conducted to prepare the contours and watershed boundaries; place benchmarks; locate buildings, streets, utility lines, drainage ditches, low and high areas, streams, and the like. All these factors influence the sewer layout.

7-3-3 Layout Plan

Proper sewer layout plan and profiles must be completed before design flows can be established. The following is a list of basic rules that must be followed in developing a sewer layout plan and profile:

1. Select the site for the wastewater treatment plant. Basic considerations for site selection of wastewater treatment plants have been presented in Chapter 2. For gravity system, the best site is generally the lowest elevation of the entire drainage area.
2. The preliminary layout of sewers is made from the topographic maps. In general, sewers are located on streets or on available rights-of-way and are sloped in the same direction as the slope of the natural ground surface.
3. The trunk sewers are commonly located in valleys. Each line is started from the intercepting sewer and extended uphill until the edge of the drainage area is reached and further extension is not possible without working downhill.
4. Main sewers are started from the trunk line and extended uphill, intercepting the laterals.

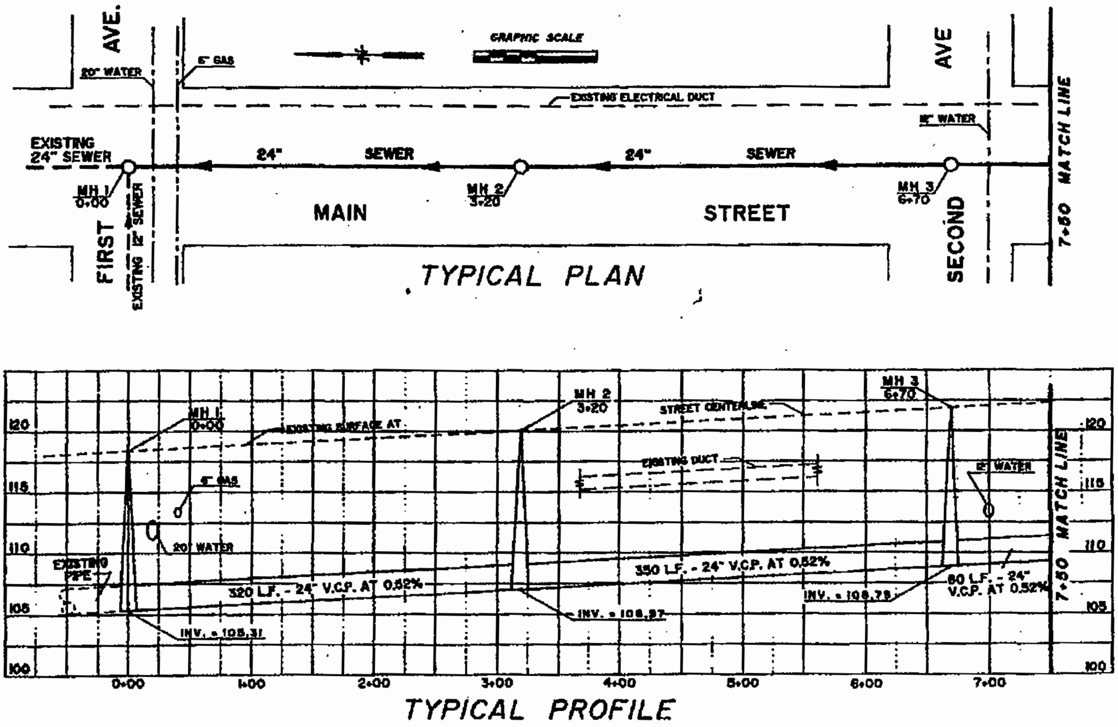

Figure 7-2 Typical Plan and Profile Drawings for Sanitary Sewer. 1 ft = 0.3048 m (from Ref. 7; used with permission of National Clay Pipe Institute).

5. All laterals or branch lines are located in the same manner as the main sewers. Building sewers are directly connected to the laterals.
6. Preliminary layout and routing of sewage flow is done by considering several feasible alternatives. In each alternative, factors such as total length of sewers and cost of construction of laying deeper lines versus cost of construction, operation, and maintenance of lift station, should be evaluated to arrive at a cost-effective sewerage system.
7. Sewers should not be located near water mains. State and local regulations must be consulted for appropriate separation distance between the sewers and water lines.
8. After the preliminary sewer layout plan is prepared, the street profiles are drawn. These profiles should show the street elevations, existing sewer lines, and manholes. These profiles are used to design the proposed lines.

Finally, these layout plans and profiles are revised after the field investigations and sewer designs are complete. A typical sewer profile is shown in Figure 7-2.[7]

7-3-4 Design and Construction Considerations

Many design and construction factors must be investigated before sewer design can be completed. Factors such as design period; peak, average, and minimum flows; sewer slopes and minimum velocities; design equations; sewer material; and joints and connections, appurtenances, and sewer installation are all important in developing sewer design. Many of the factors are briefly discussed below.

Design Period. Design period should be based on ultimate tributary population. It is not uncommon to design sewers for a design period of 25–50 years or more.

Design Population. Population projections must be made for the population at the end of the design year. Discussion on population projection techniques can be found in Chapter 2.

Design Flow. Sanitary sewers should be designed to carry peak residential, commercial, and industrial flows and the normal infiltration and inflow where unfavorable conditions exist.[8] Methods of estimating peak, average, and minimum flows and allowance for infiltration/inflow have been presented in Chapters 3 and 6. Laterals and submain sewers are designed for 1500 L per capita per day (400 gpcd), main and trunk sewers for 1330 L per capita per day (350 gpcd), and intercepting sewers for 1000 L per capita per day (260 gpcd). To all these values, industrial contributions and allowance for infiltration/inflow should also be made.

Minimum Size. Minimum sewer size recommended by Ten States Standards is 20 cm (8 in.).[9] Many states allow 15-cm (6-in.) lateral sewers.

Velocity. In sanitary sewers, solids tend to settle under low-velocity conditions. Self-cleaning velocities must be developed regularly to flush out the solids. Most states specify minimum velocity in the sewers under low flow conditions. A good practice is to maintain velocity above 0.3 m/s (1 ft/s) under low-flow conditions. Under peak dry weather condition, the lines must attain velocity greater than 0.6 m/s (2 fps). This way the lines will be flushed out at least once or twice a day. In depressed sewers (inverted siphons) self-cleaning velocities of 1.0 m/s (3.0 fps) must be developed to prevent accumulation of solids.

Velocities higher than 3.0 m/s (10 fps) should be avoided as erosion, and damage may occur to the sewers or manholes.

Slope. Flat sewer slopes encourage solids deposition and production of hydrogen sulfide and methane. Hydrogen sulfide gas is odorous and causes serious pipe corrosion.[10,11] Methane gas has caused explosions. Most states specify minimum slope for the sanitary sewers. These minimum slopes are such that a velocity of 0.6 m/s (2 fps) is reached when flowing full and $n = 0.013$. If sewer slopes of less than the recommended values are provided, the state agencies may require depth and velocity computations at minimum, average, and peak flow conditions. Minimum sewer slopes for different diameter lines are summarized in Table 7-2.[3,9]

Depth. The depth of sewers is generally 1–2 m below the ground surface. Depth depends on the water table, lowest point to be served (ground floor or basement), topography, and the freeze depth.

Appurtenances. Sewer appurtenances include manholes, building connections, junction chambers or boxes, terminal cleanouts, and others. These are discussed briefly below. Discussion of sewer appurtenances can be obtained in Refs. 1, 4, 7, and 11–13.

TABLE 7-2 Minimum Recommended Slopes of Sanitary Sewer

Diameter		
in.	mm	Slope (m/m)
6	150	0.0060
8	200	0.0040
10	250	0.0028
12	310	0.0022
14	360	0.0017
15	380	0.0015
16	410	0.0014
18	460	0.0012
21	530	0.0010
24	610	0.0008
27	690	0.00067
30	760	0.00058
36	910	0.00046
42	1050	0.00038
48	1200	0.00032
54	1370	0.00026

Source: Adapted in part from Refs. 3 and 9.

Manholes. Manholes for small sewers are generally 1.2 m (4 ft) in diameter. For larger sewers (exceeding 60 cm, or 24 in. in diameter) larger manhole bases are provided, although a 1.2-m barrel may still be used. Manholes should be of durable structure, provide easy access to the sewers for maintenance, and cause minimum interference to the sewage flow. Manholes should be located at the end of the line (called "terminal cleanout"), at the intersection of sewers, and at changes in grade and alignment except in curved sewers.[a] The maximum spacing of manholes is 90–180 m (300–600 ft), depending on the size of sewer and available size of sewer-cleaning equipment. Manholes, however, should not be located in low places where surface water may enter. If such locations are unavoidable, special watertight manhole covers should be provided. Typical manhole details are shown in Figure 7-3. Details of a terminal cleanout are shown in Figure 7-4.

Drop Manholes. The purpose of a drop manhole is to reduce the turbulence in the manhole if the elevation difference between incoming and outflow sewers is greater than 0.5 m (1.5 ft). The turbulence caused by a sudden drop of wastewater may cause splashing and release of gases that are corrosive and odorous, and can damage the manhole. Details of drop manholes are shown in Figure 7-3(c).

[a]Curved sewers are commonly used on curved streets. They eliminate manholes and construction is economical. Minimum radius is 30 m. Modern equipment makes cleaning of curved lines possible.

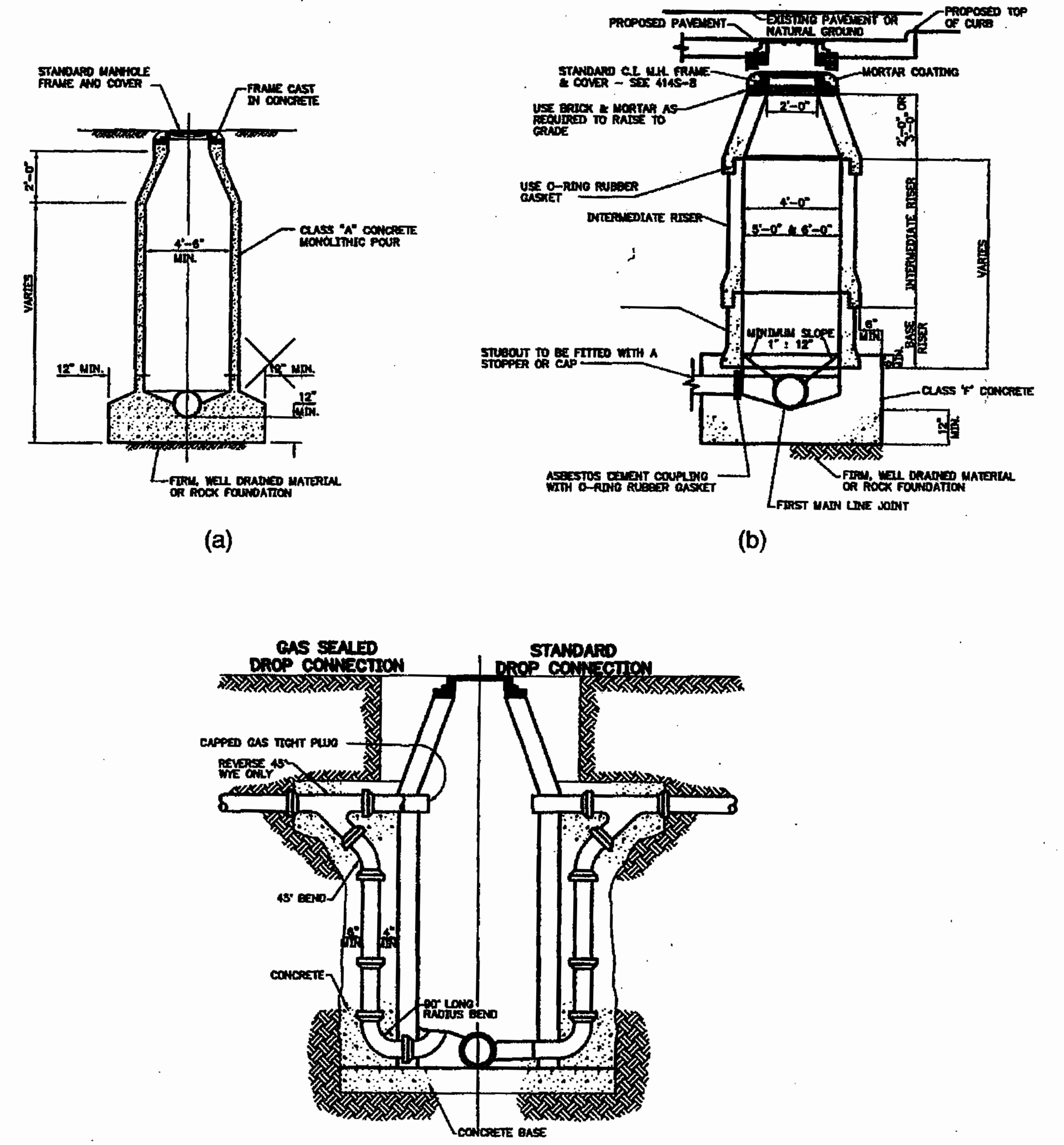

Figure 7-3 Typical Designs of Manholes: (à) standard cast-in-place manhole, (b) standard precast manhole, and (c) standard drop manhole connection (courtesy Chiang, Patel and Yerby, Inc.).

Building Connections. The building sewers are generally 10–15 cm (4–6 in.) in diameter and constructed on a slope of 0.02 m/m. Building connections are also called house connections, service connections, or service laterals. Service connections are generally provided in the municipal sewers during construction. While the sewer line is under construction, the connections are conveniently located in the form of wyes or tees and plugged tightly until service connections are made. In deep sewers, a vertical pipe or

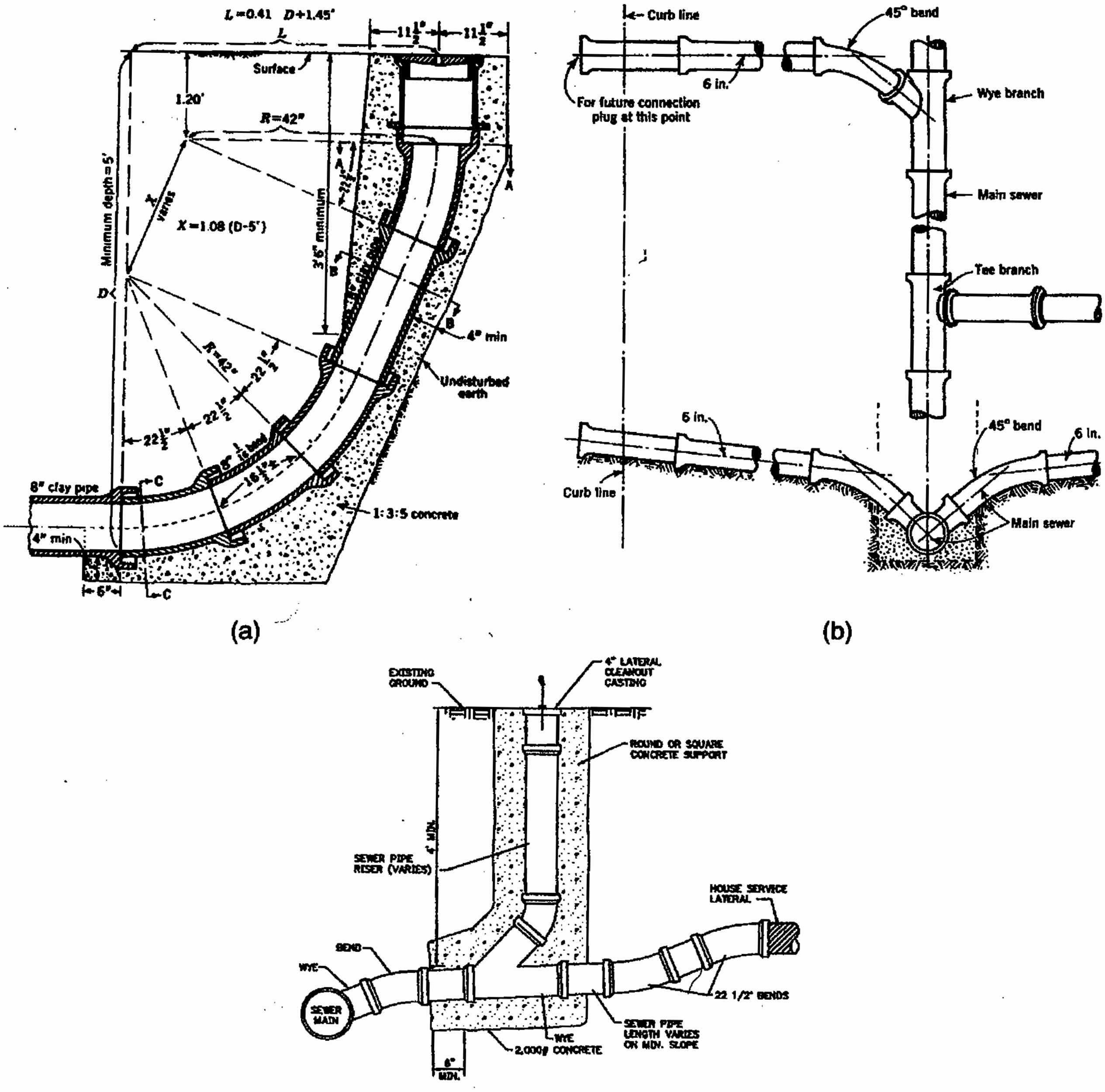

Figure 7-4 Typical Terminal Cleanouts and House Connections. 1 ft = 0.3048 m: (a) terminal cleanout (from Ref. 13; used with permission of National Clay Pipe Institute); (b) service connections for shallow sewer (from Ref. 11; used with permission of Water Environment Federation and American Society of Civil Engineers); (c) lateral sewer and house service lateral (courtesy Chiang, Patel & Yerby, Inc.).

riser encased in concrete (called a chimney) is provided for house connections. Details of terminal cleanout and house connections are given in Figure 7-4.

7-3-5 Design Equations and Procedure

Circular Sewer Flowing Full. Once the peak, average, and minimum flow estimates (including allowances for infiltration/inflow) are made and general layout and topographic features for each line are established, the design engineer begins to size the sewers. Design equations proposed by Manning, Chezy, Gangrullet, Kutter, and Scobey have been used for designing sewers and drains.[1,8,14] The Manning equation has received the most widespread application. The Manning equation in various forms is expressed below:

$$V = \frac{1}{n} R^{2/3} S^{1/2} \quad \text{(SI unit)} \tag{7-1}$$

$$Q = \frac{0.312}{n} D^{8/3} S^{1/2} \quad \text{(circular pipe flowing full, SI unit)} \tag{7-2}$$

$$V = \frac{1.486}{n} R^{2/3} S^{1/2} \quad \text{(U.S. customary unit)} \tag{7-3}$$

$$Q = \frac{0.464}{n} D^{8/3} S^{1/2} \quad \text{(circular pipe flowing full, U.S. customary unit)} \tag{7-4}$$

where

V = velocity for sewer flowing full, m/s (ft/s)
R = hydraulic mean radius = A/P (circular pipe flowing full, $R = D/4$), m (ft)
A = area of pipe flowing full ($\pi/4\ D^2$), m^2 (ft^2)
P = wetted perimeter ($\pi\ D$), m (ft)
D = diameter, m (ft)
Q = flow, m^3/s (ft^3/s)
S = slope of energy grade line or invert slope, m/m (ft/ft)
n = coefficient of roughness used in Manning equation

Coefficient of roughness depends on the material and age of the conduit. Commonly used values of n for different materials are given in Table 7-3.

TABLE 7-3 Common Values of Roughness Coefficient Used in the Manning Equation

Material	Commonly Used Values of n
Concrete	0.013 and 0.015
Vitrified clay	0.013 and 0.015
Cast iron	0.013 and 0.015
Brick	0.015 and 0.017
Corrugated metal pipe	0.022 and 0.025
Asbestos cement	0.013 and 0.015
Earthen channels	0.025 and 0.030

Various types of nomographs have been developed for solution of problems involving sewers flowing full. Nomographs based on the Manning equation for circular pipe flowing full and variable n values are provided in Figure 7-5.[1,7,8,14-16]

Circular Sewer Flowing Partially Full. Sanitary sewers are primarily designed to flow partially full. The hydraulic element equations for circular pipe flowing under part-full flow conditions are given by Eqs. (7-5) and (7-8) and are graphically shown in Figure 7-6.[8] The central angle is ϑ. It may be noted that the value of n decreases with the depth of flow (Figure 7-6). However, in most designs n is assumed constant for all flow depths. Also, it is common practice to use d, v, q, a, and p (lowercase) notations for depth of flow, velocity, discharge, area, and wetted perimeter under partial flow condition, while D, V, Q, A, and P (uppercase) notations are used for sewer flowing full. A sewer line that is flowing 80 percent full has a d/D ratio of 0.8.

$$\cos \frac{1}{2}\theta = \left(1 - \frac{2d}{D}\right) \tag{7-5}$$

$$a = \frac{D^2}{4}\left(\frac{\pi\theta}{360} - \frac{\sin\theta}{2}\right) \tag{7-6}$$

$$p = \frac{\pi D\theta}{360} \tag{7-7}$$

$$r = \frac{D}{4}\left(1 - \frac{360 \sin\theta}{2\pi\theta}\right) \tag{7-8}$$

Use of Eqs. (7-1) and (7-2) and Figures 7-5 and 7-6 are shown in the Design Example.

Organization of Computation. After the preliminary sewer layout plan and profile are prepared, the design computations are accomplished. Design computations for sewers are repetitious and therefore are best performed in a tabular format. Most textbooks provide computational tables that can be adapted for desired application. Computational procedures may be found in Refs. 1, 4, 11, 12, 15, and 16.

7-3-6 Construction Materials

Sewers. Sewers are made from concrete, reinforced concrete, vitrified clay, asbestos-cement, brick masonry, cast iron, ductile iron, corrugated steel, sheet steel, and plastic. Important factors in selection of sewer material include the following:[11-13,15,16]

1. Chemical characteristics of wastewater and degree of resistance to corrosion against acid, base, gases, solvents, etc.
2. Resistance to scour and flow (friction coefficient)
3. External forces and internal pressures
4. Soil conditions

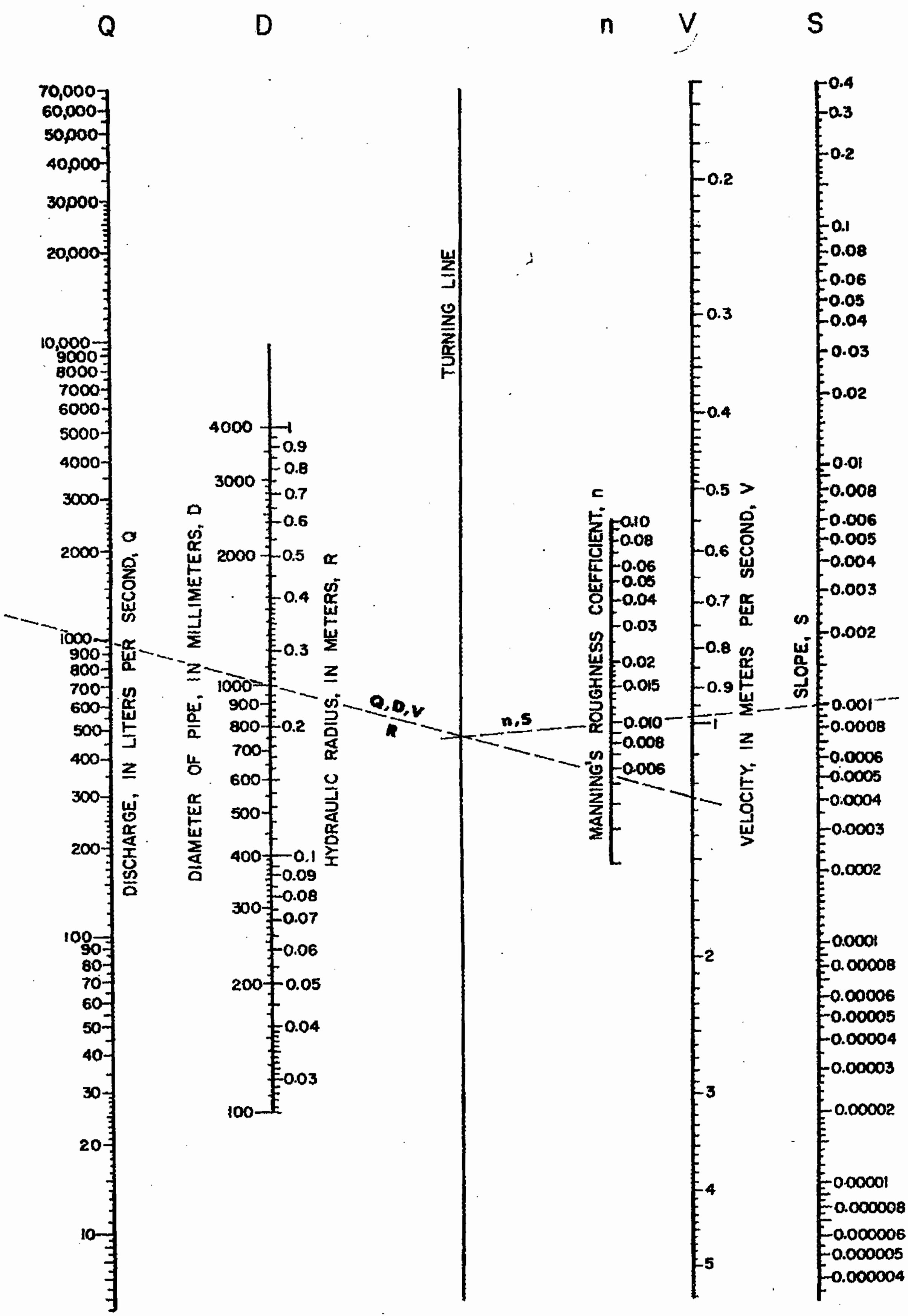

Figure 7-5 Diagram for Solution of the Manning Formula (from Practical Hydraulics for the Public Works Engineer; used with Permission of Public Works Journal Corporation Magazine).

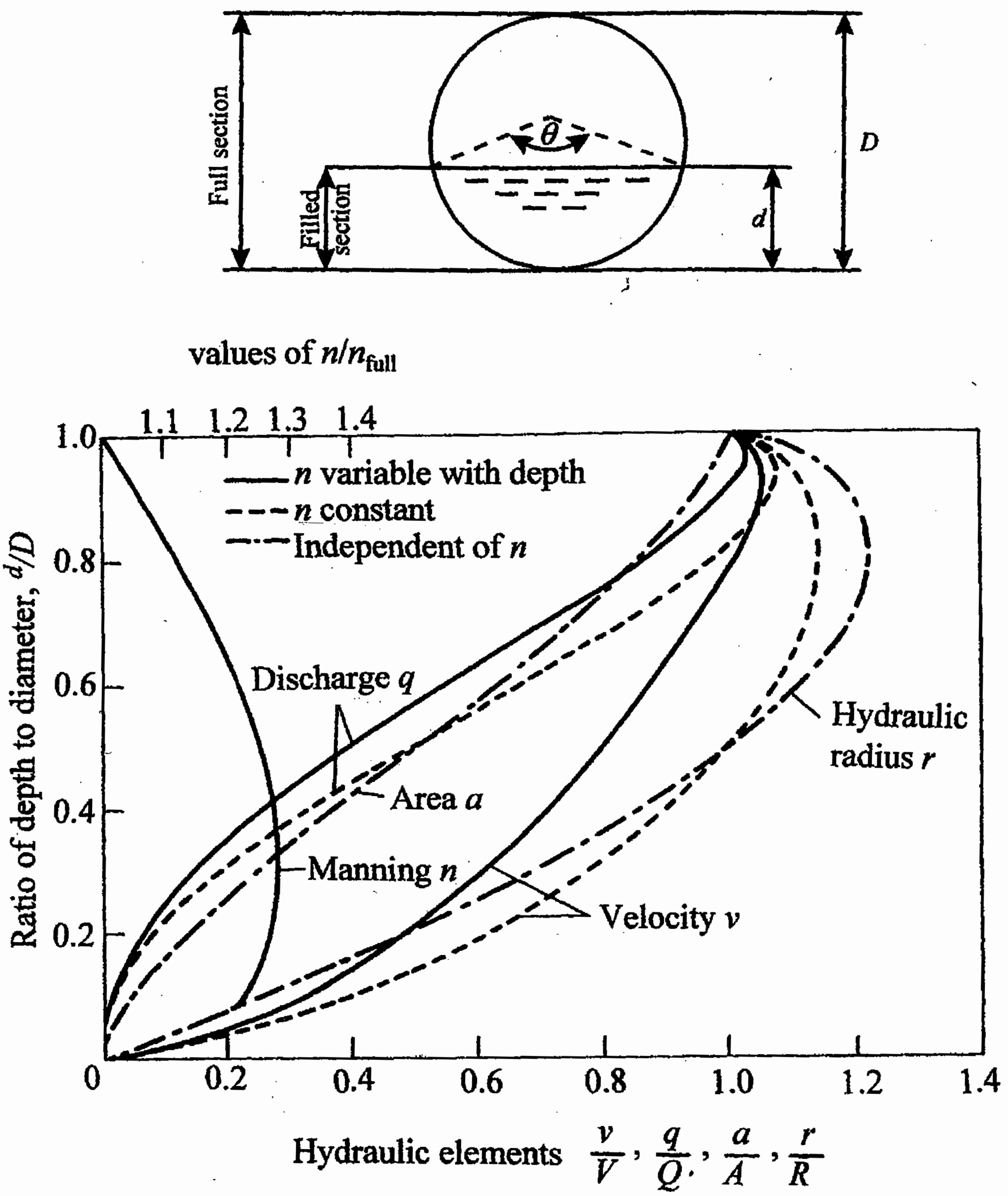

Figure 7-6 Hydraulic Properties of Circular Sewers: (a) partial flow condition and (b) hydraulic elements graph.

5. Type of backfill and bedding material to be used
6. Useful life
7. Strength and water tightness of joints required and effective control of infiltration and inflow
8. Availability in diameter, length, and ease of installation
9. Cost of construction and maintenance

Basic properties of sewer pipes of different materials are summarized in Table 7-4.

Manholes. Manholes are generally constructed from (1) brick masonry, (2) precast concrete, (3) cast-in-place concrete, (4) concrete block, and (5) fiberglass. Manhole cov-

TABLE 7-4 *Characteristics of Sewer Pipes Made from Common Construction Materials*

Construction Material	Available Sizes (dia.) cm	Available Sizes (dia.) in	Available Section Length m	Available Section Length ft	Reference Standard	Subject to Corrosion or Erosion	Strength	Type of Joint	Application
Concrete									
Plain	10–61	4–24	1.2–2.4	4–8	ASTM C14[a]	Yes	Good	Bell and spigot, or tongue and groove with rubber gasket, hot or cold poured bituminous material, cement mortar	Used for sewers, force mains, submerged outfalls, inverted siphons. Low cost, short lengths, many joints
Plain reinforced	30–305	12–120	1.2–2.4	4–8	ASTM C76	Yes	Good		
Reinforced concrete cylinder	30–366	12–144	1.2–7.3	4–24	ASTM C300–303	Yes	Good		
Mortar lined (& coated) steel	30–366	12–144	1.2–7.3	4–24	ASTM C200–210	Yes	Good		
Vitrified clay	10–122	4–48	0.9–1.8	3–6	ASTM C700	No	Brittle	Mortar, rubber gasket, compression	Used for sewers. Short lengths, many joints, susceptible to infiltration. Not a common material, for new pipes
Ductile iron (Optional-cement mortar lined) (Optional-epoxy coated)	8–163	3–64	5.5–6.1	18–20	ANSI[b]/AWWA[c] C151/821.51	Yes	Excellent	Bell and spigot, flanged, mechanical or grooved coupled, rubber or neoprene ring-type push on joint, restrained joint or ball and socket	Used for force mains, water mains, yard pipings, etc. Long laying lengths, tight joints, high pressure applications. Susceptible to soil corrosion
Corrugated and plain steel	5–792	2–312	6.4–12.8	21–42	AWWA C200	Yes	Good	Band or bolt, coating or lining, coupling	Used for sewers and drainage culverts

Plastic		4–15	4.0–6.1	13–20	ASTM D3034	No	Fair	Rubber cement, chemical weld sleeves, compression gasket	Light weight, tight joints, long laying lengths, thin walls, susceptible to sunlight and temperature which affect shape and strength. Susceptible to long term deflections.
Gravity	10–38	18–27	4.0–6.1	13–20	ASTM F679				
	46–69	21–48	4.0–6.1	13–20	ASTM F794				
	53–122								
Pressure	4–30	1.5–12	4.0–6.1	13–20	ASTM D2241	No	Fair		
	10–30	4–12	4.0–6.1	13–20	AWWA C900				
	36–122	14–48	4.0–6.1	13–20	AWWA C905				
Cast iron[d]	5–120	2–48	Long length	Long length	AWWA C100	Yes	Excellent	Bell and spigot, flanged, mechanical or groove coupled, rubber or neoprene ring-type push on joint, or ball and socket	Used for force mains, water mains, yard pipings, etc. Long laying lengths, tight joints, withstand high pressures. Susceptible to soil corrosion
Asbestos-cement[d]	10–110	4–42	—	—	AWWA C400	Yes	Good	Collar with rubber ring	Used for force mains, water distribution lines, sewers. Light weight, easy to transport and handle
Brick masonry[d]	Greater than 91	Greater than 36	Any length	Any length	None	Yes	Good	Mortar	Uncommon these days except for lining of concrete sewers

[a]ASTM—American Society for Testing and Materials.
[b]ANSI—American National Standards Institute.
[c]AWWA—American Water Works Association.
[d]These are not in common use in the United States.
Source: Courtesy of Freese and Nichols, Inc.

ers are made of cast iron and should be tightly fitted to eliminate inflow, uncovering, and rattling. They should also have provisions to vent gases and admit air. Manhole steps should be placed at intervals of 30–50 cm (12–16 in.) and be designed to prevent corrosion and slipping.

Joints and Infiltration. The method of making joints should be fully covered in the specifications. Joints should be designed to make sewers watertight, root-resistant, flexible, and durable. A leakage test should be specified. The leakage infiltration and exfiltration shall not exceed 0.2 m^3 per d per cm of pipe diameter per kilometer (200 gpd per in. of pipe diameter per mile). It has been experimentally demonstrated that joints made from rubber gasket and hot-poured bituminous material produced almost no infiltration, whereas cement mortar joints cause excessive infiltration.[9,17] The air test shall, as a minimum, conform to the test procedure described in ASTM for clay and concrete pipes.

7-3-7 Sewer Construction

Sewer construction involves excavation, sheeting and bracing of trenches, pipe installation, and backfilling. Each of these construction steps is discussed briefly below.

Determining Structural Requirements. The constructed sewer must be able to withstand the forces under which it must function. The structural design requires that the supporting strength of the conduit as installed, divided by a suitable factor of safety, must equal or exceed the loads imposed on it by the weight of earth and any superimposed loads. The supporting strength of a buried conduit is a function of installation conditions, as well as the strength of the pipe itself. The three most common conditions of installation are (a) in a trench in natural ground, (b) in an embarkment, and (c) in a tunnel. Loading conditions due to gravity earth forces for each installation are different and involve weight for the prism of earth directly over it, frictional shearing forces, cohesive forces, forces induced by backfill and settlement, and the like. Marston's equation and many modifications for different soils and installation conditions are used to check the structural integrity of the sewer line. Many types of nomographs have also been developed to facilitate the calculations. This topic is beyond the scope of this publication. Readers are referred to several sources for determining the structural requirements of sewer installation.[4,7,8,11-13,16]

Excavation. After the sewer alignment is marked on the ground, the trench excavation begins. Machinery such as backhoe, clamshell, dragline, front-end loader, or other specialized equipment is used. Hand excavation may be possible only for short distances. Hard rocks may be broken by drilling; explosives may also be used where situations permit.

Sheeting and Bracing. Trenches in unstable soil conditions require sheeting and bracing to prevent caving. Sheeting is placing planks in actual contact with the trench sides. Bracing is placing crosspieces extending from one side of the trench to the other. Sheeting and bracing may be of various types, depending on the depth and width of the trenches and the type of soils supported. Common types are stay bracing, poling boards, box sheeting, vertical sheeting, and skeleton (open) sheeting. Details on sheeting and bracing may be found in Refs. 4, 7, and 11–13. In many situations pumping may be necessary to dewater the trenches.

Sewer Installation. After the trench is completed, the bottom of the trench is checked for elevation and slope. In firm, cohesive soils the trench bottom is shaped to fit the pipe barrel and projecting collars. Often granular material such as crushed stones, slag, gravel, and sand are used to provide uniform bedding for the pipe.

The pipes are inspected and lowered, with particular attention being given to the joints. The pipe lengths are placed on line and grade with joints pressing together with a lever or winch. The joints are then filled per specifications. A typical pipe installation is shown in Figure 7-7.

Backfilling. The trenches are filled immediately after the pipes are laid. The fill should be carefully compacted in layers of 15 cm (6 in.) deep around, under and over the pipe. After completion of the filling, the surface is then finished.

7-3-8 Field Investigations and Completion of Design

Field work should be conducted to establish bench marks on all streets that will have sewer lines. Soil borings should be conducted to develop subsurface data needed for trenching and excavation. The depth of boring should be at least equal to the estimated depth of the sewer lines. Detailed plans should be drawn showing the following: (1) contours at 0.5-m intervals in map with scale 1 cm equal to 6 m (1 in. = 50 ft); (2) existing and proposed streets; (3) street elevations; (4) railroads, buildings, culverts, drainage ditches, etc.; (5) existing conduits and other utility lines; and (6) existing and proposed sewer lines and manholes. The sewer profiles should also be developed showing ground surface and sewer elevations, slope, pipe size and type, and location of special structures and the appurtenances. Profile drawings should be prepared immediately under the sewer plan for ready reference (see Figure 7-2).

7-3-9 Preparation of Contract Drawings and Specifications

It is important that the detailed drawings be prepared and specifications completed before the bids can be requested. The contract drawings should show (1) surface features, (2) depth and character of material to be excavated, (3) the existing structures that are likely to be encountered, and (4) the details of sewer and appurtenances to be constructed.

The specifications should be prepared by writing clearly and completely all work requirements and conditions affecting the contracts. As an example, technical specifications should cover items such as site preparation, excavation and backfill, concrete work, sewer materials and pipe laying, jointing, appurtenances, and acceptance tests (infiltration, exfiltration, smoke, or air tests).

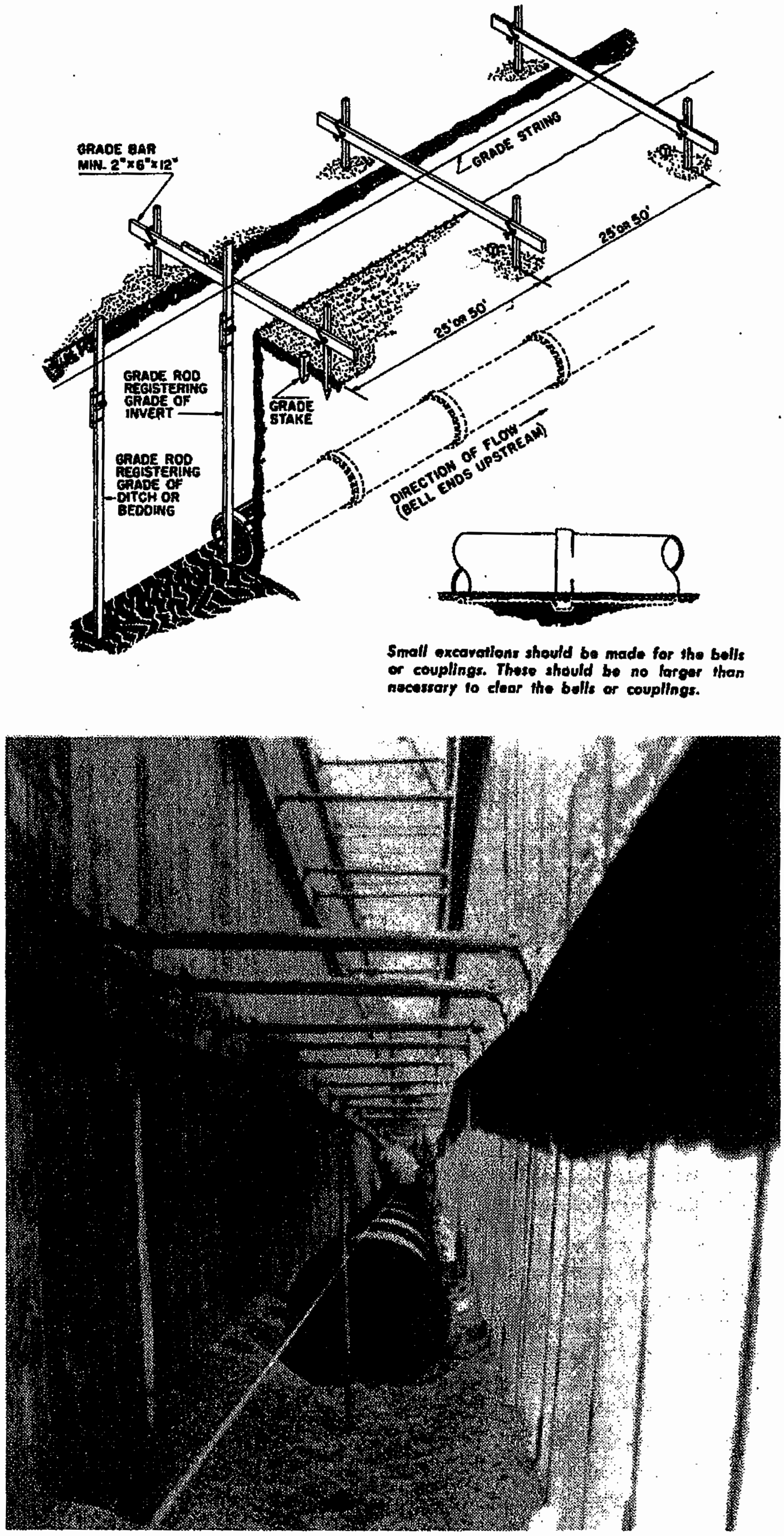

Figure 7-7 Sewer Trench and Installation: (a) laying clay pipe in line grade; (b) vitrified clay pipe installation in deep trench (from Ref. 13; used with permission of National Clay Pipe Institute).

7-4 INFORMATION CHECKLIST FOR THE DESIGN OF SANITARY SEWERS

Design of sanitary sewers involves preliminary investigations, a detailed field survey, design calculations, and field drawings. The design engineer should be familiar with the service area, the local and state design criteria, and the design procedures. Adherence to a carefully planned sequence of activities to develop sewer design minimizes project delays and expenditures. A checklist of design activities is presented below. These activities are listed somewhat in their order of performance. However, in many cases separate tasks can be performed concurrently or even out of the order given below:

1. Develop a sewer plan showing existing and proposed streets and sewers, topographic features with contour intervals of 0.5 m, elevations of street intersections, and location of permanent structures, and existing utility lines. Mark the proposed sewer lines and tentative slopes.
2. Locate manholes and number them in accordance with a convenient numbering system.
3. Prepare vertical profile showing ground surface, manhole locations, and elevation at the surface of each manhole.
4. Determine total land surface area that will eventually be served by different sewer lines.
5. Determine expected saturation population densities and average per capita wastewater flow rates.
6. Estimate peak design flow, and peak, average, and minimum initial flows.
7. Review design equations and develop hydraulic properties of the conduits.
8. Obtain state standards, sewer codes, or any design and maintenance criteria established by the concerned regulatory agencies.

7-5 DESIGN EXAMPLE

7-5-1 Design Criteria Used for Sewer Design

Five sewer lines are designed in this example. These lines are

1. Diversion sewer from stabilization pond area
2. Diversion sewer from the trickling filter plant area
3. Existing intercepting sewer from the central part of the town
4. Final sewer line from the junction chamber to the bar screen
5. Bypass sewer from the junction chamber to the storage basin

The layout plan of these lines are shown in Figure 6-2 and 6-4, and the initial and design flows were developed in Chapter 6 (Tables 6-8 and 6-9). Following are the basic design criteria used for the design of these lines:

1. All lines shall be designed to flow partially full under peak design flow.
2. The velocity under initial low-flow condition shall not be less than 0.45 m/s (1.5 ft/s).

TABLE 7-5 Peak, Average, and Minimum Flows for the Initial and Design Years for Five Sewer Lines[a]

Sewer Lines		Flow Conditions Initial Year (m^3/s)			Flow Conditions Design Year (m^3/s)		
		Peak	Avg.	Min.	Peak	Avg.	Min.
i	Diversion sewer from stabilization ponds	0.388	0.124	0.065	0.590	0.192	0.100
ii	Diversion sewer from tricking filter plant	0.257	0.086	0.041	0.385	0.132	0.063
iii	Existing intercepting sewer	0.274	0.094	0.047	0.331	0.116	0.058
iv	Final sewer to the plant	0.966	0.304	0.152	1.321	0.440	0.220
v	Bypass sewer	0.966	0.304	0.152	1.321	0.440	0.220

[a]Peak, average, and minimum flows were developed in Chapter 6 and summarized in Tables 6-8 and 6-9.

3. Provide concrete sewers and assume roughness coefficient $n = 0.015$.
4. The peak, average, and minimum flows for initial and design years are summarized in Table 7-5. These values are obtained from Tables 6-8 and 6-9.
5. The sewer layout plan and sewer profiles are shown in Figure 7-8 and 7-9.

7-5-2 Design Calculations

Step A: Sewer Design. The design computations for the sewers should be started from the upper reach of the line and carried downward in the direction of the flow. The cumulative flows between manholes are used to determine the sewer diameter and slope and ground and invert elevations. In this example, the step-by-step calculations for the diversion sewers and intercepting sewers are not shown in their entirety. Instead, the calculations for only the last section of these pipes are given with reference to the floor elevation of the rack chamber. This elevation was established from the old sewer line serving the existing aerated lagoon. The design procedure involves the following steps:

1. Select the diameter, slope, and roughness coefficient n for the sewer line.
2. Determine the discharge Q and velocity V from Eqs. (7-1) and (7-2) or from Figure 7-5 when the sewer is flowing full. Q should be larger than the peak design flow.
3. Calculate q/Q ratios at peak design flow and at minimum initial flow.
4. Determine d/D and v/V ratios at these flows from Figure 7-6.
5. Calculate velocity and depth of flow under these flow conditions.
6. The velocity at minimum initial flow should be greater than 0.45 m/s.
7. The design steps are summarized in Table 7-6.

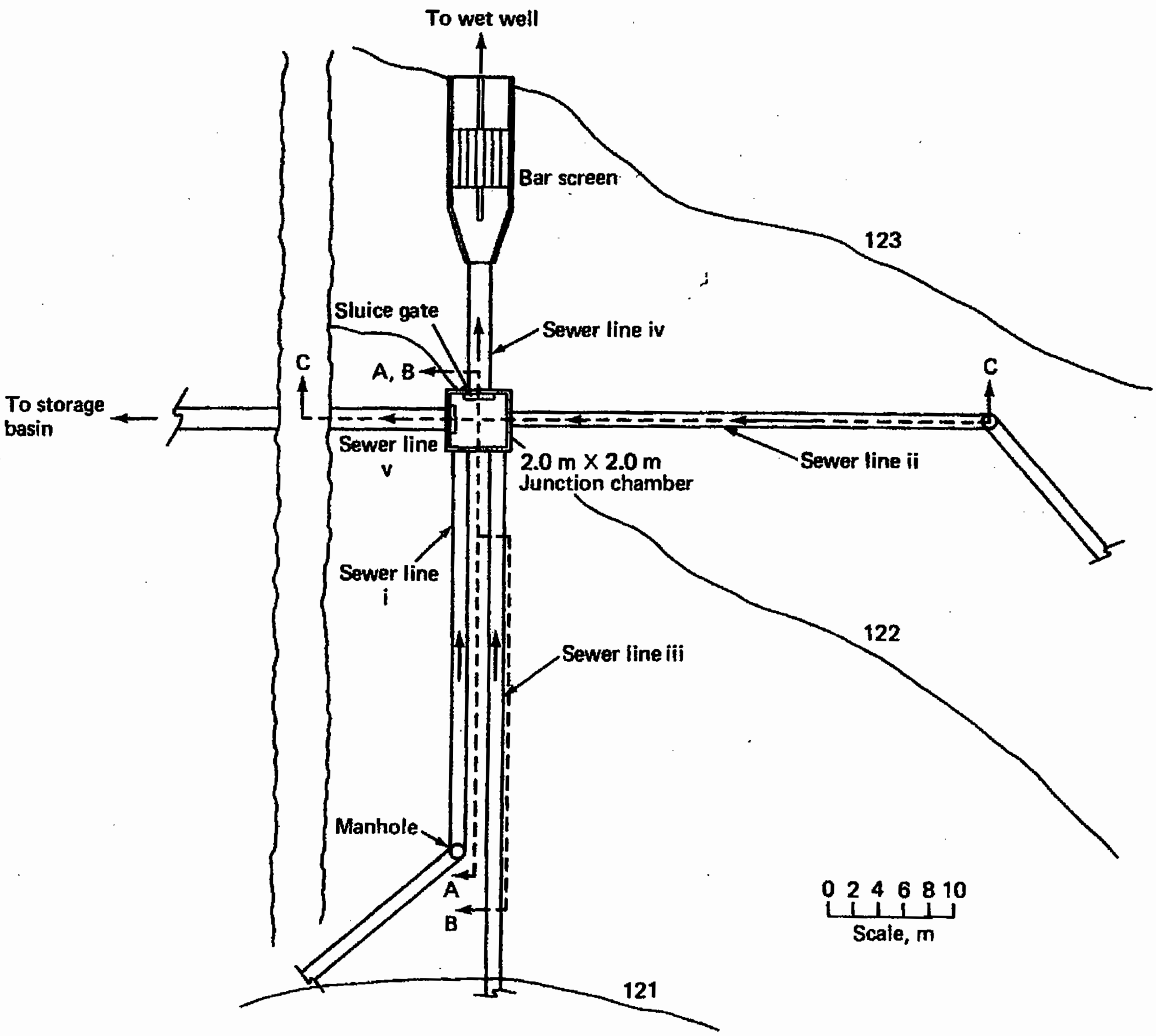

Figure 7-8 Sewer Layout for Design Example (see Tables 7-5 and 7-6 for sewer line identification number and details).

Step B: Junction Chamber. The junction chamber has internal dimensions of 2.0 m × 2.0 m. Three incoming sewers join at the junction chamber. A single line 1.53 m diameter carries the total flow to the treatment facility. A manually operated sluice gate is provided to close the line and divert the flow into a 1.22-m bypass sewer. The bypass sewer is designed to divert the incoming flow into the storage basin at the time of power outages. Design details of junction chamber are shown in Figures 7-8 and 7-9.

Step C: Hydraulic Profile. The water surface profiles for all sewers were developed from the water depth, slope of sewer, and the head losses encountered in the appurtenances. The head losses in a manhole or collection chamber are caused by (1) exit loss, (2) change in direction, (3) contractions and expansions, (4) transitions, and (5) entrance loss. All these losses may be conveniently expressed by Eq. (7-9):

$$h_L = K \frac{v^2}{2g} \tag{7-9}$$

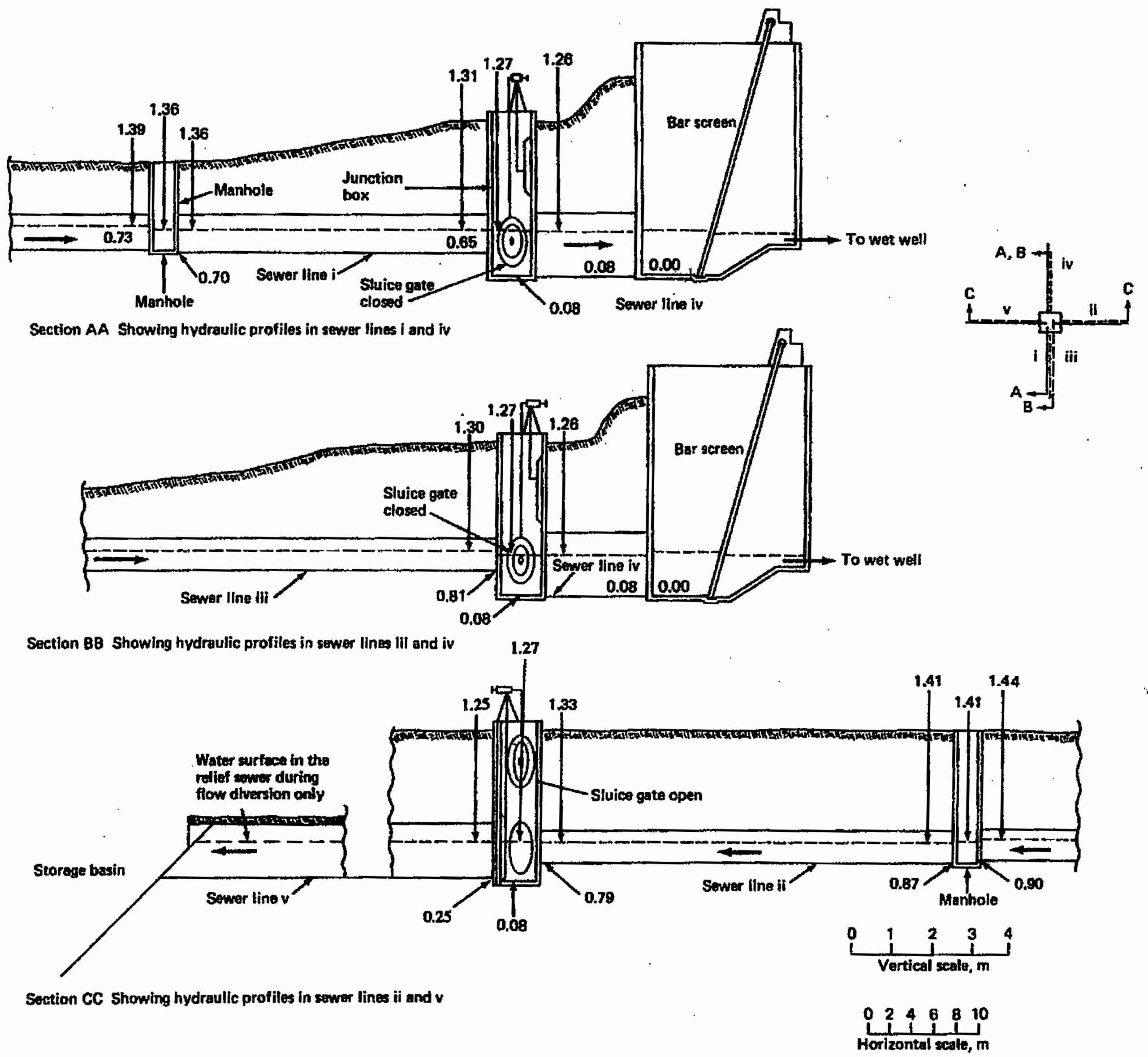

Figure 7-9 Sewer Profiles for the Design Example (see Tables 7-5 and 7-6 for sewer line identification number and details). All Elevations Are in Meters above the Floor of the Rack Chamber.

where

h_L = minor loss due to entrance, exit, or change in direction of the flow, m
v = velocity of flow, m/s
g = acceleration due to gravity, 9.81 m/s^2
K = head loss coefficient

The values of K for different conditions of open-channel flow and sewer appurtenances are summarized in Appendix B. The head loss calculations are given below, and hydraulic profiles for different sewer lines are shown in Figure 7-9. These calculations utilize velocities, depths of flow, and sewer slopes developed in Table 7-6. This simplified procedure gives conservative design if higher values of K are selected from Appendix B. An

TABLE 7-6 Computations for Sewer Design

Design Steps	Diversion Line from Stabilization Pond i	Diversion Line from Trickling Filter Plant ii	Existing Intercepting Sewer from Central Part of Town iii	Final Sewer from the Junction Chamber iv	Bypass Sewer from Junction Chamber v
Select diam (m)	0.91	0.76	0.76	1.53	1.22
Select slope (m/m)	0.00173	0.002	0.002	0.00047	0.00147
Select n	0.015	0.015	0.015	0.015	0.015
Q full from Eq. (7-2) (m^3/s)	0.673[a]	0.447	0.447	1.402	1.355
V full from Eq. (7-1) (m/s)	1.03[b]	0.98	0.98	0.76	1.16
q peak design flow (Table 7-5) (m^3/s)	0.590	0.385	0.331	1.321	1.321
q/Q	0.88[c]	0.86	0.74	0.94	0.98
d/D (Fig. 7-6, n constant)	0.72	0.71	0.64	0.77	0.82
v/V (Fig. 7-6, n constant)	1.13	1.13	1.09	1.18	1.14
d (m)	0.66	0.54	0.49	1.18	1.00
v (m/s)	1.17[d]	1.11	1.07	0.87	1.32
q' minimum initial flow (Table 7-5) (m^3/s)	0.065	0.041	0.047	0.152	0.152
q'/Q	0.10	0.09	0.11	0.11	0.11
d'/D (Fig. 7-6, n constant)	0.23	0.21	0.23	0.23	0.23
v'/V (Fig. 7-6, n constant)	0.62	0.58	0.62	0.62	0.62
d' (m)	0.21	0.16	0.18	0.35	0.28
v' (m/s)	0.64	0.57	0.61	0.47	0.72

[a] $Q = (0.312/0.015) \cdot (0.91)^{8/3}(0.00173)^{0.5} = 0.673$ m^3/s. Q may also be obtained from Figure 7-5.
[b] $V = (1/0.015)(0.91/4)^{2/3}(0.00173)^{0.5} = 1.03$ m/s. V may also be obtained from Figure 7-5.
[c] $(q/Q) = (0.590/0.673) = 0.88$.
[d] $v = 1.15 \times 1.03 = 1.18$

alternative method is to apply the energy equation at two sections and then solve for velocity or depth of flow by a trial-and-error procedure. Besides being tedious and time-consuming, the accuracy of the result depends on the judgment of the designer in selecting the head loss coefficient, which present many uncertainties. The use of energy equation for establishing the hydraulic profile is covered in Chapter 8 and in Refs. 1 and 18. Many designers prefer to match the crowns of all sewers meeting at a manhole. Since the outgoing sewer is always the largest, a drop in manhole invert is necessary. Some designers drop the invert of each manhole by about 0.03 m. These drops are generally more than that needed to compensate for the head losses due to the exit, entrance, and others. Under these conditions, the hydraulic profile calculations are not performed.

1. Compute the hydraulic profile from the bar screen to the junction box.
   The invert elevation at the bar screen chamber = 0.00 m.
   a. Set differential elevation of incoming sewer line iv at 0.08 m above the invert of the bar screen chamber (see Chapter 8 for more details)
   b. Exit loss at the discharge point[b] $= \dfrac{Kv^2}{2g} = \dfrac{0.09 \times (0.87\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = \text{small}$
   c. Depth of flow = 1.18 m (Table 7-6)
   d. Water surface elevation in the sewer line iv at the exit point = 0.00 m + 0.08 m + 1.18 m = 1.26
   e. Invert elevation of the sewer line iv at the junction
   $= 0.08\text{ m} + \text{length} \times \text{slope}$
   $= 0.08\text{ m} + 8\text{ m}^{\text{b}} \times 0.00047$
   $= 0.08\text{ m} + \text{small} = 0.08\text{ m}$
   f. Water surface elevation in the sewer line iv at the junction box = 0.08 m + 1.18 = 1.26 m
   g. Entrance loss into the sewer line iv[b] $= \dfrac{Kv^2}{2g} = \dfrac{0.3\,(0.87\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = 0.01\text{ m}$
   h. Water surface elevation in the junction box = 1.26 m + 0.01 m = 1.27 m

2. Compute hydraulic profile from the junction box to the manhole in the sewer line i
   a. Water surface elevation in the sewer line i at the exit point[b] = 1.27 m + exit loss
   $$= 1.27\text{ m} + \frac{Kv^2}{2g}$$

[b]Entrance and exit conditions and length may be seen in Figures 7-8 and 7-9.

$$= 1.27 \text{ m} + \frac{0.5 \times (1.17 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2}$$

$$= 1.27 \text{ m} + 0.04 \text{ m} = 1.31 \text{ m}$$

b. Invert elevation of the sewer line i at the junction box $= 1.31 \text{ m} - \text{water depth}$
$= 1.31 \text{ m} - 0.66 \text{ m} = 0.65 \text{ m}$

c. Invert elevation of sewer line i at the manhole $= 0.65 \text{ m} + \text{length} \times \text{slope}$
$= 0.65 \text{ m} + 30 \text{ m} \times 0.00173$
$= 0.65 \text{ m} + 0.05 \text{ m} = 0.70 \text{ m}$

d. Water surface elevation in the sewer line i $= 0.70 \text{ m} + 0.66 \text{ m} = 1.36 \text{ m}$

e. Entrance loss in the sewer line i at the manhole $= \dfrac{Kv^2}{2g} = \dfrac{0.05 \times (1.18 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2} = \text{small}$

f. Water surface elevation in the manhole $= 1.36 \text{ m}$

g. Water surface elevation in sewer line i upstream of manhole $= 1.36 \text{ m} + \text{losses due to exit and change in direction}$

$$= 1.36 \text{ m} + \frac{Kv^2}{2g}$$

$$= 1.36 \text{ m} + \frac{0.4 \times 1.17 \text{ m/s}^2}{2 \times 9.81 \text{ m/s}^2}$$

$$= 1.36 \text{ m} + 0.03 \text{ m} = 1.39 \text{ m}$$

h. Invert elevation of sewer line i upstream of manhole $= 1.39 \text{ m} - 0.66 \text{ m} = 0.73 \text{ m}$

3. Compute the water surface and invert elevations in the sewer line iii

a. Water surface elevation in sewer line iii $= 1.27 \text{ m} + \text{exit loss}$

$$= 1.27 \text{ m} + \frac{Kv^2}{2g}$$

$$= 1.27 \text{ m} + \frac{0.5\ (1.07 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2}$$

$$= 1.27 \text{ m} + 0.03 \text{ m} = 1.30 \text{ m}$$

b. Invert elevation $= 1.30 - \text{water depth}$
$= 1.30 \text{ m} - 0.49 \text{ m} = 0.81 \text{ m}$

4. Compute the hydraulic profile in the sewer line ii
   a. Water surface elevation $= 1.27\text{ m} + \text{exit loss}$

   $$= 1.27\text{ m} + \frac{1.0\,(1.11\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2}$$

   $= 1.27 + 0.06\text{ m} = 1.33\text{ m}$

   b. Invert elevation $= 1.33\text{ m} - \text{water depth}$
   $= 1.33\text{ m} - 0.54\text{ m} = 0.79\text{ m}$

   c. Invert elevation at manhole $= 0.79\text{ m} + \text{length} \times \text{slope}$
   $= 0.79\text{ m} + 40\text{ m} \times 0.002$
   $= 0.79\text{ m} + 0.08\text{ m} = 0.87\text{ m}$

   d. Water surface elevation in the sewer line ii at the manhole $= 0.87\text{ m} + 0.54\text{ m} = 1.41\text{ m}$

   e. Water surface elevation in the manhole $= 1.41\text{ m} + \text{entrance loss}$

   $$= 1.41\text{ m} + \frac{Kv^2}{2g}$$

   $$= 1.41\text{ m} + \frac{0.05 \times (1.11\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2}$$

   $= 1.41\text{ m} + \text{small} = 1.41\text{ m}$

   f. Water surface elevation in the sewer line ii upstream of manhole $= 1.41\text{ m} + \text{losses due to exit and change in direction}$

   $$= 1.41\text{ m} + \frac{0.4\,(1.12)^2}{2 \times 9.81}$$

   $= 1.41\text{ m} + 0.03\text{ m} = 1.44\text{ m}$

   g. Invert elevation of the sewer line ii upstream of manhole $= 1.44\text{ m} - \text{water depth}$
   $= 1.44\text{ m} - 0.54\text{ m}$
   $= 0.90\text{ m}$

5. Compute the hydraulic profile in the sewer line v.
   Assume the entrance-loss coefficient $K = 0.2$. The hydraulic profile in the sewer line v during flow bypass is shown in Figure 7-9.

7-6 OPERATION AND MAINTENANCE AND TROUBLESHOOTING IN SANITARY SEWERS

7-6-1 Common Problems

A well-designed sanitary sewerage system is expected to provide trouble-free operation. However, serious problems may develop because of lack of a preventive maintenance program or factors beyond the control of the maintenance crew. Common problems that may develop into collection systems include the following:

1. Explosions or severe corrosion caused by discharge of uncontrolled industrial wastes
2. Corrosion of sewer lines and manholes caused by generation of hydrogen sulfide gas
3. Collapse of sewer because of overburden or corrosion
4. Poor construction or workmanship or earth shifts may cause pipes to break or joints to open up. Excessive infiltration/exfiltration may occur. Also, tree roots, soil, gravel, etc., may enter the sewer resulting in sewer blockages.
5. Protruding taps in the sewers because of improper workmanship (called plumber taps or hammer taps) are common sources of problems. They substantially reduce line capacity and contribute to frequent blockages.
6. Excessive settling of solids in the manhole and sewer line may lead to obstruction, blockage, or generation of undesired gases.
7. The diameter of the sewer line may be reduced by accumulation of slime, grease, and viscous materials on the pipe walls. This may lead to obstruction and blockage of the line.
8. Faulty, loose, or improperly fit manhole covers, in addition to being noisy, can be a source of inflow. Ground shifting may cause cracks in the manhole walls or in the pipe joints at the manhole. These cracks become a source of infiltration or exfiltration. Debris (rags, sand, gravel, sticks, etc.) may collect in the manhole and block the lines. Also, tree roots (like in sewer lines) may enter the manholes through the cracks, joints, or faulty cover and cause serious blockages.

7-6-2 Maintenance Program

Strict enforcement of sewer ordinances and timely maintenance of sanitary sewerage systems is the fastest and most efficient way to prevent interruptions in sewer services. Inspection and maintenance on a regular basis will minimize the possibility of damage to private property by sewer stoppages, as well as minimize the legal responsibility of the sewer authority from such damages.

For trouble-free operation of sanitary sewers, a maintenance program is necessary and should be developed. An effective maintenance program involves four steps: (1) sewer ordinance, (2) inspection, (3) preventive maintenance, and (4) repairs. Each of these steps is discussed briefly below.

Sewer Ordinance. Under the pretreatment regulations of the Clean Water Act of 1977, all industrial users are prohibited from discharging pollutants into the sewerage system that can cause fire or explosion, corrosion, or obstruction to the sewers (see Chapter 2 for pretreatment standards). Also, protruding taps in the sewer system result from lack of inspection during installation of service taps/connections, unauthorized tapping, and ineffective enforcement of plumbing codes. Many cities have passed ordinances designed to protect sewers against injury and blockage. More specifically, these ordinances (1) prohibit discharge of corrosive, inflammable, and explosive liquid, gases, and vapors, and garbage or dead animals; (2) require installation of grease traps; and (3) prohibit house connections by plumbers or require city inspection after the connection is made by the plumbers.

Inspection. Sewer inspection on a regular basis should be performed to determine the general physical condition of the sewers and manholes, as well as the need for repairs. Sewers with steep slopes or carrying large flows have adequate velocities and offer little chance for solids to deposit. On the other hand, lines carrying low discharge on flat slopes will accumulate solids and require more frequent cleaning.

In many cities sewer inspections are made and work done only when problems occur. In other cities strict inspection schedules are followed. As an example, sewers on flat grades or lines with a history of root intrusions are examined every three months; sewers with no recorded troubles once a year; intercepting sewers once or twice per month; inverted syphons monthly to weekly; and so on. Inspection is made during low flows by observing the condition of the manhole, and looking through the sewer toward an explosion-proof flashlight placed in the downstream manhole. Such observations clearly indicate evidence of corrosion, cracks, and deteriorated conditions of manholes; improperly fitted covers; damaged cover rings; and partial stoppages of the lines. A strong rotten egg odor from manholes indicates hydrogen sulfide gas, which is generated under excessive turbulence, and anaerobic decomposition of settled solids and slimes.

Sewers may also be inspected by close circuit television cameras to determine the actual conditions in the sewer lines. TV inspection is an effective method of detecting exact locations of leaks, corrosion, intrusions, failure, and partial blockages.

Preventive Maintenance. The sewer maintenance personnel should be properly trained to identify the problem and apply the appropriate remedial measures. Basic information concerning preventive maintenance and repairs is given below. When emergency situations occur (such as collapsed pipes, blockage, sewer backups, etc.), they should receive top priority and prompt action.

Equipment. The requirements for sewer maintenance equipment depend on the size and type of sewers serviced. A typical list of equipment for all types of sewers in large cities is provided in Table 7-7.

Sewer Cleaning. Sewer cleaning is a preventive measure to protect certain lines from stoppages. It can be achieved by the following procedures:

1. Hydraulic flushing is done by attaching hose to fire hydrant or sewer-cleaning machines where high-pressure pumps are used. High-pressure water jet cleans the lines. Care should be taken since high-pressure jet may damage the corroded, cracked, or broken lines. Backing up of sewers should be avoided during flushing operation.
2. Sewer balls are effective means of removing deposits, slime, and grease from the lines. The ball is inserted in the upstream manhole and is pulled downstream by a rope. The ball adjusts itself to the irregularities of the pipe and water held behind the ball rushes around the ball thus flushing grease, slime, and deposits. Roots up to pencil size are broken off and removed.[4]

TABLE 7-7 Typical List of Sewer Maintenance Equipment for Municipalities

Major Equipment	Minor Equipment	Safety Equipment
Maintenance truck	Combination wrench set	Safety helmet
Winch truck	Hex key set	Rubber gloves and boots
Back hoe	Three-way puller	Gas mask
Sewer ball cleaner	Punch and chisel set	Oxygen deficiency indicator
Hinged sewer cleaner	Pipe wrenches	Hydrogen sulfide detector
Jointed wood cleaning rods	Pipe cutters	Carbon monoxide detector
Rotating steel rodding machine	Tubing cutter set	Chlorine detector
Bucket cleaning machine	Hammers and crow bar	Portable air blower
Spring-cable sewer cleaner	Aluminum level	Protective clothing
High velocity jet cleaner (hydraulic)	Hack saw and blades	Safety harness and life lines
Manhole-cleaning unit (suction type)	Shovels, picks, rake, and wheelbarrow	Fire extinguisher
Closed circuit television	Hydraulic jack	First aid and life saving kit
Propane torch kit	Water hose and hose nozzles	Explosion proof portable light
Bench vise	Double branch chain sling	Warning signs
High speed drill set	Circular drum dollie	
Ball bearing bench grinder	Portable pump with engine	
Air compressor	Portable blower	
	Portable ladder	

Source: Adapted in part from *Operation and Maintenance Manual for the Wastewater Facilities in Big Spring, Texas,* prepared by Gutierrez, Smouse, Wilmut & Assoc., Inc., Consulting Engineers, Dallas, Texas.

3. Greater accumulations are removed by special buckets or scoops. The bucket is pulled down the line until the operator feels that it is full. It is then pulled backwards, removed, and emptied. A bucket does not work well on pipes that are out of line because of differential settlement.

Stoppage Clearing. Stoppage occurs when a line is partially or completely blocked. Stoppage may result from large objects being dropped into manholes; accumulation of sand, grit, or grease; tree roots; or collapsed line. Most stoppages (except collapsed lines) can be cleared by a rodding machine. Flexible metal rods rotated by a motor on the ground turn various cleaning and cutting devices inserted into the lines in the direction of the flow. The equipment rods should not be operated upstream because they can damage wyes and service connections. Rodding-collapsed pipes can damage the cleaning equipment.

Odor Control or Prevention of Hydrogen Sulfide Gas. Several physical and chemical methods have been utilized in controlling odors and generation of H_2S in collection systems. These methods include (1) avoidance of flat slopes of sewer when designing; (2)

avoidance of excessive turbulence such as that in drop manholes; (3) ventilation and aeration of sewers and manholes; and (4) application of chemicals such as solution of chlorine, hydrogen peroxide, ozone, or other chemicals. Detailed discussion on odor and hydrogen sulfide control may be found in Refs. 10 and 19.

Repairs. Collapsed and severely cracked pipes need excavation and replacement. This constitutes major repairs. Grouting and insertion-renewal (slip-lining) techniques are effective methods of rehabilitating old, corroded, and damaged pipes. Slip lining is done from manhole to manhole with plastic pipes made especially for this purpose.[1,4] Defective joints that admit roots and infiltration should be uncovered and repaired. All protruding taps must be redone since they reduce the capacity of a line and contribute to frequent blockages.

Manhole covers having cracks or improper fit should be repaired or replaced. If the cover ring or gasket is damaged, they should also be replaced. The manhole steps should be repaired or replaced whenever they weaken or show excessive deterioration. Damaged or missing steps can cause serious accidents. Corroded, cracked, or damaged manhole walls may require patching with cement bricks or cement mortar. If damage to the manhole walls is extensive, the entire manhole should be replaced.

7-7 SPECIFICATIONS

General specifications for a wastewater collection system are briefly summarized below. These specifications are meant only to be illustrative of the design completed. They are neither complete nor definitive.

7-7-1 Excavations

The contractor shall excavate all trenches to the depths shown on the drawings. Excavated material not required for backfill shall be disposed of by the contractor. The contractor shall do everything needed to keep excavations free of water (may need pumping), providing at all times adequate barricades and policing to prevent outsiders and employees from harm. The contractor shall do all bracing, sheeting, and shoring necessary to protect all excavations as required for safety. Where pipes cross roads and walks, excavations shall be filled before nightfall and proper barricades and lighting shall be maintained at all times until the work has been completed.

The contractor shall carefully backfill and compact all excavations and replace all paving for parking areas, roadways, and walks, including curbs, to the condition that existed before the work began. The collection system shall be laid out so that major trees will not be disturbed, but where it is necessary to temporarily remove shrubs or small trees, the contractor shall remove, ball, and then replace them.

Sewer installation between manholes must proceed with utmost rapidity, and testing shall be done in segments in accordance with the work plan.

7-7-2 Sewer Pipe

Reinforced concrete sewer pipes shall be constructed, tested, and inspected at the point of manufacture in accordance with the specified test requirements and shall bear the initials or name of the manufacturer, the date of manufacture, and the testing laboratory's stamp of approval.

The connecting joints of the reinforced concrete sewer pipe shall be manufactured to ensure accurate and concentric bell and spigot joint. All joints shall be made with approved flexible compression-type rubber gasket. The rubber gasket shall truly conform in size to the spigot end of the pipe. In laying the rubber gasket joints in concrete pipes, care shall be exercised in handling the pipe to prevent damaging the rubber gaskets and ends of the pipes. The inside of the bells and the outside of the spigots shall be clean of dirt or foreign matter. The grooves or bell end shall be coated with approved lubricating material and then pipes pulled with sufficient force to cause the spigot to enter the bell and be forced completely home. On all sizes larger than 61 cm (24 in.) in diameter, the inside annular space shall be completely filled with plastic or portland cement to effect a smooth flow.

All sewer lines shall be tested as specified for tightness, infiltration, and exfiltration between the consecutive manholes. All joints shall be tight under a 1.2-m head of water.

7-7-3 Manhole

All manholes shall be of precast reinforced concrete. Shop drawings of all manholes shall be approved before manufacture and shall be constructed as shown in the respective drawings. No steps or ladders in manholes shall be permanently casted. They may corrode and become dangerous and may encourage intruders. Maintenance crews shall bring portable ladders to be lowered into the manholes for use. Manhole covers shall be new, heavy-duty, and constructed in accordance with standard guidelines. The covers shall be gastight and with machined bearing surfaces and neoprene gaskets.

7-7-4 Junction Chamber

The junction chamber shall be 2.0 m × 2.0 m, brought to the depth as shown in the drawing, and constructed in-place of reinforced concrete. The concrete shall be of approved aggregates and cement in specified proportions and mixed in an approved-type, power-operated batch mixer to yield the specified ultimate compressive strength at the age of 28 days. Reinforcing steel shall conform to the standard specifications and shall conform in size and position to the requirements of the drawings. The forms shall conform to the lines, dimensions, and shapes of concrete indicated on the drawings. They shall be tight to prevent leakage of concrete, shall be securely braced and maintained in position to prevent movement after concrete is poured, and shall have adequate strength to safely support the concrete. All concrete shall be placed with the aid of mechanical vibration equipment, and curing and finishing shall be in accordance with the American Concrete Institute (ACI) standards. The corners, dead spaces, and the bottom of the junction chamber shall be contoured properly for free flow of incoming wastewater into the exit sewer.

7-8 PROBLEMS AND DISCUSSION TOPICS

7-1 A 60-cm diameter (24-in.) sewer is designed to carry a peak design flow of 0.01 m^3/s at a slope of 0.0008 m/m. Calculate the actual depth of flow and velocity. Assume constant value of $n = 0.015$.

7-2 Repeat Problem 1 for variable value of n. Assume $n = 0.015$ when the line is flowing full.

7-3 Calculate the depth of flow and velocity into the sewer line iv in the Design Example, at average and minimum design flows. The diameter and slope of the line are given in Table 7-6. The flow conditions are provided in Table 7-5. Assume $n = 0.015$.

7-4 Calculate the diameter of a trunk sewer to carry 50 L/s when flowing 60 percent full. The velocity in the line at partial full condition must be 0.6 m/s ($n = 0.013$).

7-5 Discuss the basic design factors that must be considered in developing the preliminary design and layout of the sanitary sewerage system for a community.

7-6 Prepare a checklist of an effective maintenance program for the sanitary sewerage system for a community.

7-7 An intercepting sewer is 400 mm in diameter and is serving a population of 5000 residents. The average wastewater flow is 450 Lpcd. Calculate the depth of flow and velocity at minimum flow. The slope of the line is 0.003. Assume $n = 0.013$ and minimum flow is one-third of the average flow.

7-8 An intercepting sewer is 61 cm in diameter and has a slope of 0.0013. It carries a flow of 113.3 L/s. The sewer enters a standard manhole that is 76 cm in diameter. The outlet sewer is also 61 cm in diameter and has a slope of 0.0014. Using $n = 0.015$, draw the hydraulic profile and determine the invert elevation of the incoming sewer. Assume that the coefficients of exit and entrance are 0.4 and 0.1, respectively, and the manhole invert is in line with the outlet sewer.

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8

Screening

8-1 INTRODUCTION

Screening is normally the first unit operation used at wastewater treatment plants. The general purpose of screens is to remove large objects such as rags, paper, plastics, metals, and the like. These objects, if not removed, may damage the pumping and sludge removal equipment; hang over weirs; and block valves, nozzles, channels, pipelines, and appurtenances, thus creating serious plant operation and maintenance problems. In recent years there has been renewed interest in all types of screens. The applications range from removal of settleable solids such as grit and primary sludge to polishing of effluent from final clarifiers. The reasons for the renewed interest is because of more reliable screening equipment and the need for more compact wastewater treatment plants.

In this chapter different types of screening devices and the design criteria for bar racks are covered. The step-by-step design procedure, operation and maintenance, and equipment specifications for bar racks are presented in the Design Example.

8-2 TYPES OF SCREENS

Screening devices can be broadly classified as coarse or fine. Screens may be manually or mechanically cleaned. Various types of coarse and fine screens are discussed below.

8-2-1 Coarse Screen

Coarse screens are used primarily as protective devices and therefore are used as the first treatment unit. Common types of protective devices include bar racks (or bar screens), coarse woven-wire screens, and comminutors. A screen composed of parallel bars or rods is called a bar rack. A brief description of racks and comminutors is provided in Table 8-1.[1-3] Installations of bar racks and comminutors are shown in Figure 8-1.

8-2-2 Fine Screen

The fine screens are now available in openings ranging from 0.035 to 6 mm. They are usually mechanically cleaned. The purpose of fine screens is to provide pretreatment or primary treatment.[2] Various types of microscreens have been developed in recent years that are used for upgrading effluent quality from secondary treatment plants.

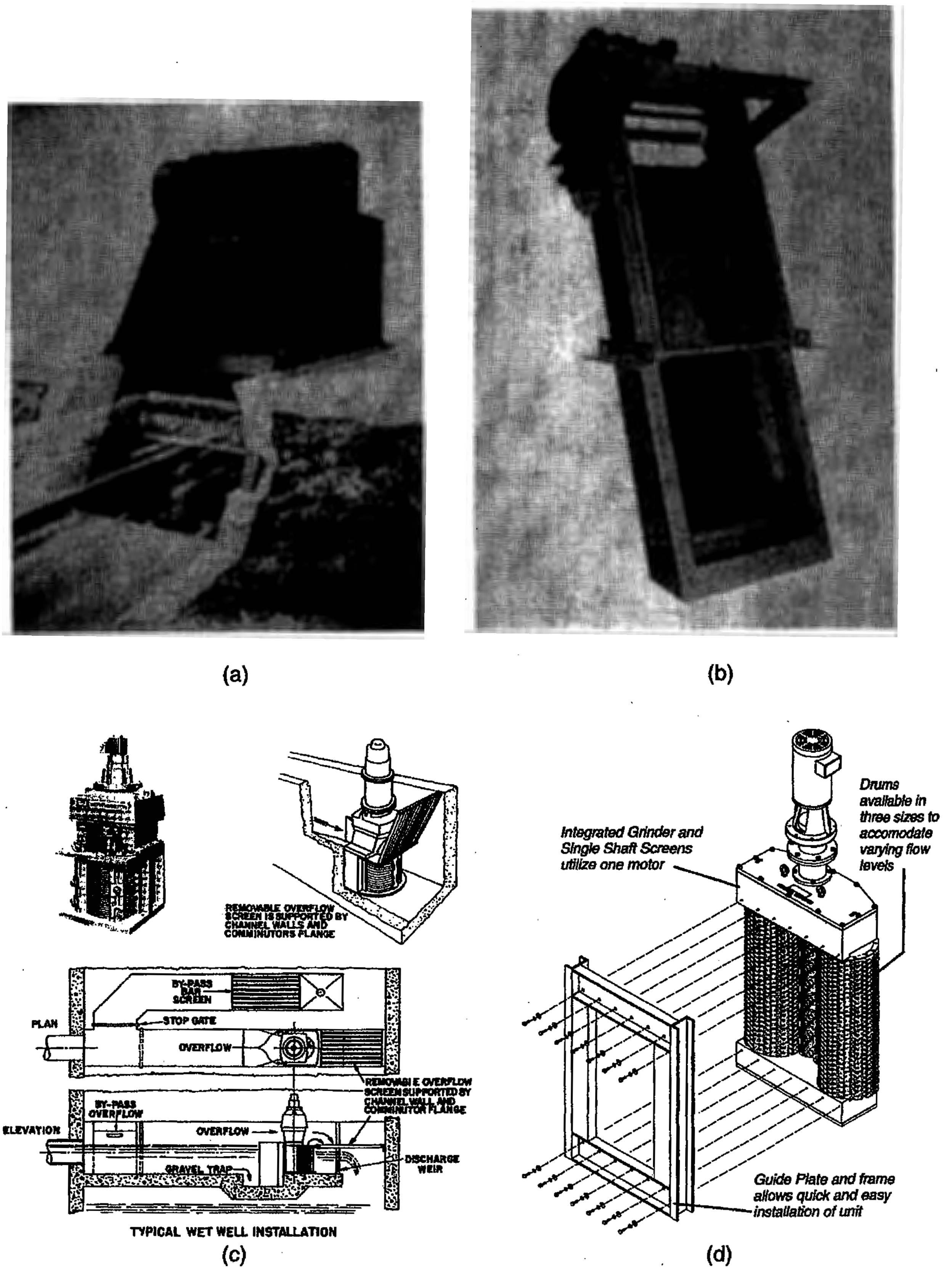


Figure 8-1 Bar Rack and Comminutor Assembly: (a) inclined rack, front cleaned (courtesy Envirex, U.S. Filter); (b) cable-operated bar rack (courtesy Envirex, U.S. Filter); (c) comminutor assembly and typical wet well installation (courtesy Worthington Pump Corp.); and (d) double drum integrated grinder and single shaft screen and motor (courtesy JWC Environmental).

TABLE 8-1 Description of Coarse Screen and Communitor

Type	Location	Description
Bar racks or bar screens	Ahead of pumps and grit removal facilities	Bar racks may be manually cleaned or mechanically cleaned. Manually cleaned racks are provided at small wastewater treatment facilities. The clear spacings of the bars may range from 1.5 cm to 4.0 cm. Bar rack installations are shown in Figure 8-1(a) and (b).
Grinder or comminutor	Used in conjunction with coarse screens	Grinders or comminutors are used to grind or cut up the screenings. They utilize cutting teeth or shredding devices on a rotating or oscillating drum that passes through stationary combs, screen, or disks. Large objects are shredded that pass through thin openings or slots 0.6–1 cm. Manufacturers' rating tables are available for different capacity ranges, channel dimensions, submergence, and power requirements. Provision to bypass the device is always made. Comminutor installation is shown in Figure 8-1(c). A double drum integrated grinder and screen is shown in Figure 8-1(d).

Fine screens consist of fixed, movable, and centrifugal screens. The fixed or static screens are permanently set in vertical, inclined, or horizontal position and must be cleaned by rakes, teeth, or brushes. Movable screens are cleaned continuously while in operation. Centrifugal screens utilize the rotating screens that separate effluent, and solids are concentrated.

Most fine screens are capable of removing 20–35 percent suspended solids and BOD_5.[1] They also remove grease and tend to increase the dissolved oxygen (DO) levels of the wastewater. The moving screens, however, exhibit less head loss, but require more power for operation than the fixed screens. Different types of fine screens, their operation range, and applications are summarized in Table 8-2.[4-7] Installation and configuration of several fine screens are shown in Figure 8-2.

8-3 DESIGN FACTORS FOR BAR RACKS (BAR SCREENS)

Bar racks are the most commonly used devices at medium- and large-sized wastewater treatment facilities. They contain a screen chamber with inlet and outlet structures and a screening device that has an arrangement for cleaning and removing the screenings. The design velocity, bar spacing, bar size, angle of inclination, and allowable head losses through the racks are summarized in Table 8-3. The screen chamber (with inlet and outlet arrangements), methods for calculating head losses through the screen, equipment for cleaning and removing the screenings, and methods of estimating the quantities of screenings are described below.

TABLE 8-2 Types of Medium and Fine Screens

Type of Screen	Description	Opening Size	Application
Inclined (fixed)	These are inclined fixed screens. Flat-, basket-, cage-, or disk-type screens remove smaller particles [Fig. 8-2(a)].	0.25–2.5 mm	Primary treatment
Band	Band of screen consists of an endless perforated band that passes over upper and lower sprocket. Brushes may be installed to remove the material retained over the screen. Water jet may also be used to flush the debris [Fig. 8-2(b)].	0.8–2.5 mm	Pretreatment
Wing or shovel screens	These screens consist of circular perforated radial vans that slowly rotate on a horizontal axis. The vans scoop through the channel.	1.0–5.0 mm	Pretreatment
Drum screen or strainer	The screens consist of a rotating cylinder that has screen covering the circumferential area of the drum. The liquid enters the drum and moves radially out. The solids deposited are removed by a jet of water from the top and discharged into a trough. The screen openings may range in size giving coarse medium and fine screens. The microstrainers have very fine screens and are used to polish the secondary effluent or remove algae from the effluent of stabilization ponds [Fig. 8-2(c)].	1.0–5.0 mm 0.25–2.5 mm 6–40 μm (0.04 mm)	Pretreatment Primary treatment Polishing secondary effluent
Rotary disc	These are rotary disc covered with mesh. The disc rotates slowly in a specially contoured chamber. The effluent passes through the mesh and exits as filtrate, while waste liquid containing captured solids is discharged through a chute that has an adjustable weir [Fig. 8-2(d)].	0.05–1.0 mm	Pretreatment Primary treatment

Rotary sheer screen	The influent enters over the adjustable weir and falls over a rotating screen. The screening action takes place inside the cylinder. The shearing action of the rotating screen separates the tumbling solids. A series of diverters moves solids to the end of the cylinder for removal [Fig. 8-2(e)].	0.05–1.0 mm	Primary treatment
Rotary basket screen	The wastewater flows into a semicircular screening basket where solids are retained. As the liquid level in the basket rises to a predetermined level, the rake begins to rotate and clean the screen bars. The rake's teeth pass between the bars of the screen to remove the screened material. The raked solids drop into the central screw conveyor. The screw conveyor then transports the screened material that is washed, compacted, and dewatered before discharge into a chute [Fig. 8-2(f)].	0.25–5 mm	Primary treatment
Centrifugal screen	The influent enters at the bottom of the center feed column, travels upward, and feeds the horizontal distribution pipes. The nozzles at the end of the distribution pipes cause liquid to impinge tangentially against the inner surface of the revolving screw in a series of overlapping thin layers of liquid. A low centrifugal force is also generated by the rotating screen cage. The effluent is thus forced through the screen. The concentrate moves downward into an outlet pipe [Fig. 8-2(g)]. The screen is used successfully in industrial waste applications.	0.025–0.5 mm	Primary treatment and polishing secondary effluent
Step screen	The step screens have step-shaped laminae, where every other lamina is connected to one fixed and one moveable part on which a carpet of solids is formed. This gives a high sieving efficiency. The rotating step-shaped laminae convey the solids upwards, step-by-step. The circular movement is self-cleaning, and the solids are automatically off-loaded at the discharge point [Fig. 8-2(h)].	1–6 mm	Pretreatment

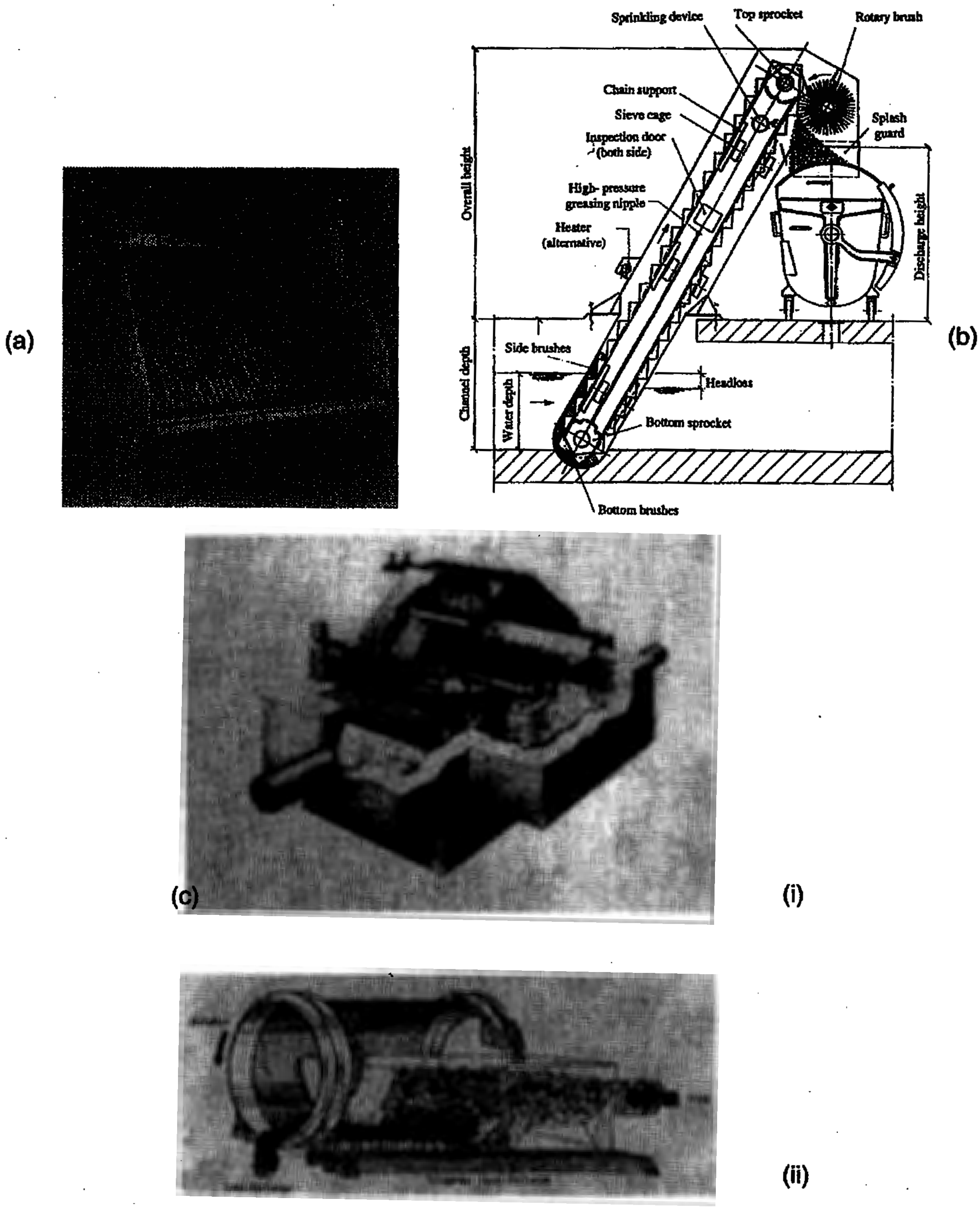
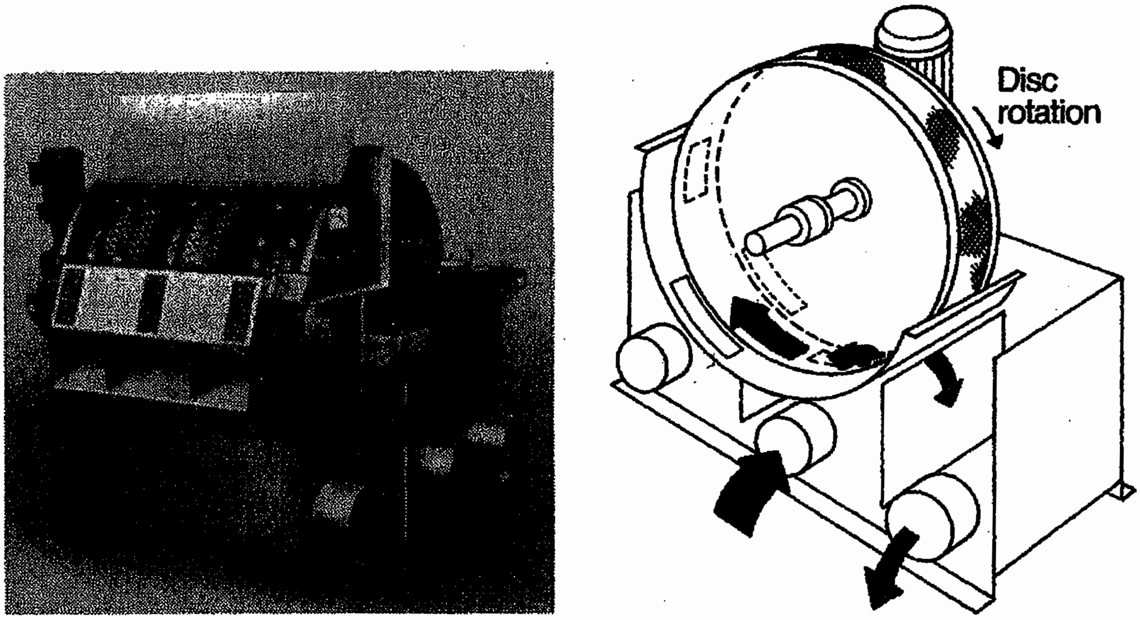
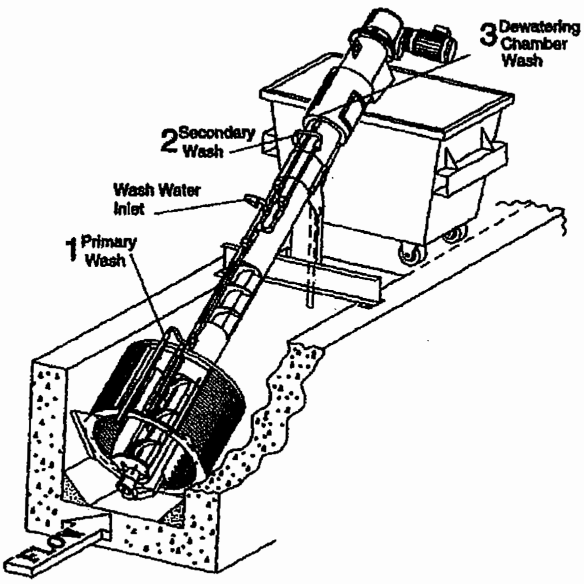


Figure 8-2 Various Types of Screens in Common Use: (a) Inclined Static Wedgewire Screen (courtesy Hycore Corp.); (b) endless traveling vertical or inclined band screen (courtesy Jones & Attwood Inc.); (c) rotating drum screen [(i) courtesy Envirex, U.S. Filter, (ii) courtesy Andritz Inc.].



(d)

(e)

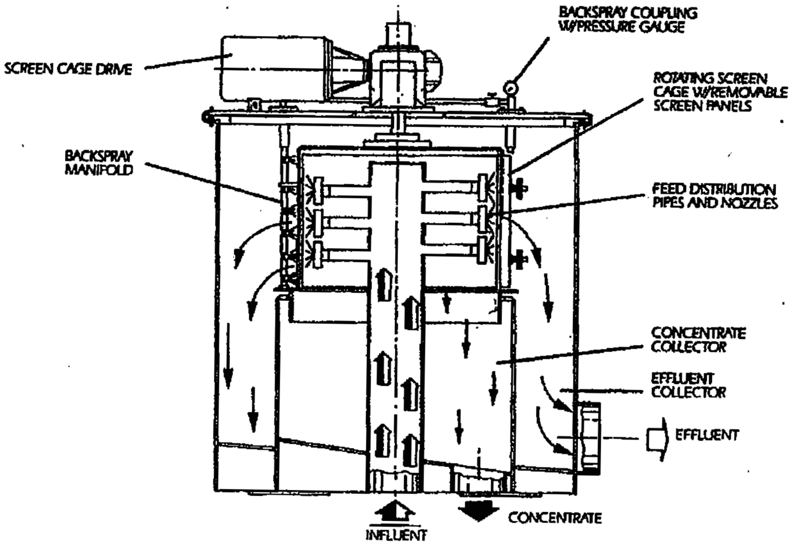


(f)

Figure 8-2—cont'd (d) revolving disc screen (courtesy Hycore Corp.); (e) rotary shear screen (courtesy Hycore Corp.); (f) rotary basket screen (courtesy Lakeside Equipment Corp.).

Continued

(g)



(h)

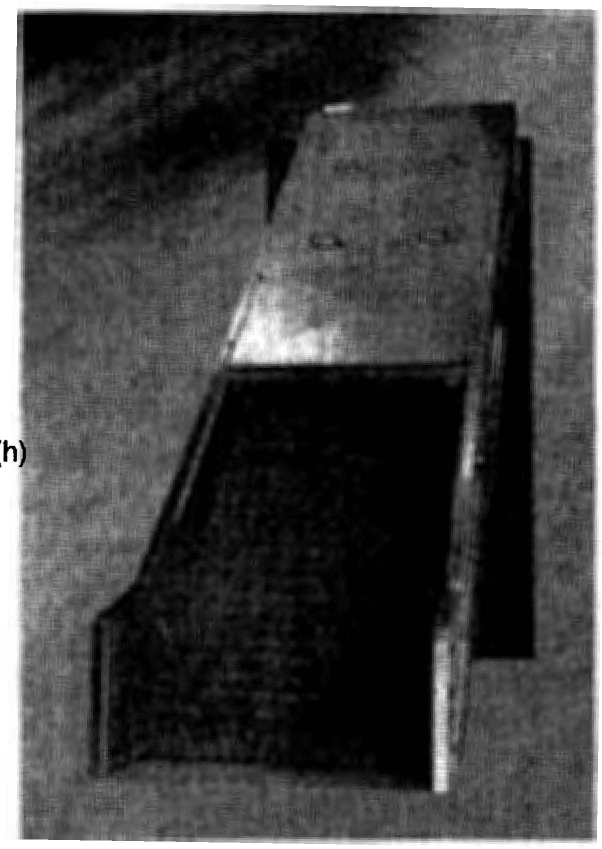


Figure 8-2—cont'd (g) centrifugal screen (courtesy SWECO, Inc.); and (h) step screen L (courtesy Hydropress Wallander & Co. AB).

8-3-1 Screen Chamber

A screen chamber consists of a rectangular channel. The floor of the channel is normally 7–15 cm lower than the invert of the incoming sewer. Also, the channel floor may be flat (horizontal) or at a desired slope. The screen channel is designed to prevent the accumulation of grit and other heavy materials into the channel.[2,3] The channel is normally pro-

TABLE 8-3 Design Factors for Manually Cleaned and Mechanically Cleaned Bar Racks

Design Factor	Manually Cleaned	Mechanically Cleaned
Velocity through rack (m/s)	0.3–0.6	0.6–1.0
Bar size		
Width (mm)	4–8	8–10
Depth (mm)	25–50	50–75
Clear spacing between bars (mm)	25–75	10–50
Slope from horizontal (degrees)	45–60	75–85
Allowable head loss, clogged screen (mm)	150	150
Maximum head loss, clogged screen (mm)	800	800

Source: Adapted in part from Refs. 1–4.

vided with a straight approach, perpendicular to the screen, to assure uniform distribution of screenings over the entire screen area.

At least two bar racks, each designed to carry the designed peak flow, must be provided in case one is out of service. Arrangements for stopping the flow and draining the channel should be made for routine maintenance. The entrance structure should have a smooth transition or divergence in order to minimize the entrance losses as wastewater is discharged from the interceptor into the channel and to prevent settling and accumulation of grits. Likewise, the effluent structure should have uniform convergence. The effluent from individual rack chambers may be combined or kept separate as necessary. The bar rack chambers, as well as influent and effluent structures, are shown in Figure 8-3. In all cases, however, an account must be made for head losses due to exit, bends, expansion, contraction, and entrance.

8-3-2 Head Loss

The head loss through the bar rack is calculated from Eqs. (8-1), (8-2), or (8-3).[1,2] Equation (8-1) is used to calculate head losses through clean or partly clogged bars, while Eq. (8-2) is used to calculate head loss through clean screen only.[1,2] Equation (8-3) is the common orifice formula and is also used to calculate head loss through fine screens.[1]

$$h_L = \frac{V^2 - v^2}{2g}\left(\frac{1}{0.7}\right) \tag{8-1}$$

$$h_L = \beta\left(\frac{W}{b}\right)^{4/3} h_v \sin\theta \tag{8-2}$$

$$h_L = \frac{1}{2g}\left(\frac{Q}{C_d}\right)^2 \tag{8-3}$$

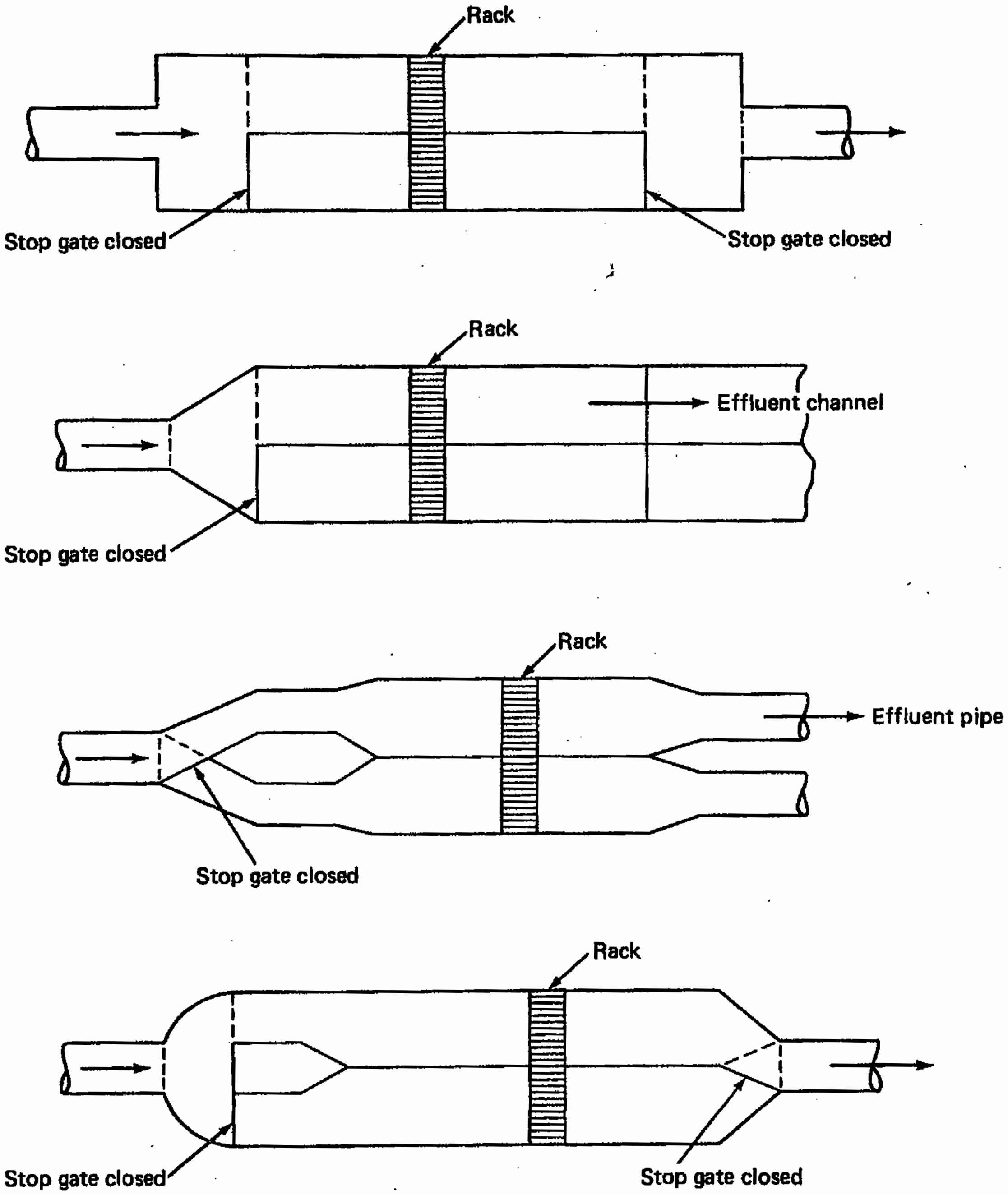


Figure 8-3 Double Chamber Bar Rack and Influent and Effluent Arrangements.

h_L = head loss through the rack, m
V, v = velocity through the rack and in the channel upstream of the rack, m/s
g = acceleration due to gravity, 9.81 m/s^2
W = maximum cross-sectional width of the bars facing the direction of flow, m
b = minimum clear spacing of bars, m
h_v = velocity head of the flow approaching the bars, m
θ = angle of bars with horizontal
Q = discharge through screen, m^3/s
A = effective submerged open area, m^2
C_d = coefficient of discharge = 0.60 for clean rack

β = bar shape factor; the values of bar shape factors for clean rack are summarized as follows:

Bar Type	β
Sharp-edged rectangular	2.42
Rectangular with semicircular upstream face	1.83
Circular	1.79
Rectangular with semicircular upstream and downstream faces	1.67
Tear shape	0.76

8-3-3 Removal of Screenings

Manually cleaned bar racks have sloping bars that facilitate hand raking. The length of a hand-cleaned rake should not exceed 3 m for convenience in racking. The screenings are placed on a perforated plate for drainage and storage. Adequate area for accumulation of screenings between the racking operations must be provided at the bars.

The mechanically cleaned bar racks are divided into four types: (a) chain-operated, (b) reciprocating, (c) catenary, and (d) cable. The chain-operated racks are the most common type and are front-cleaned or back-cleaned. In both cases the traveling rake moves the screenings upward and drops them into a collection bin or a conveyor. The back-cleaned raking device has the advantage that it does not jam easily because of obstruction at the base of the screen. In both types the raking is performed continuously by means of endless chains operating over sprockets. The operation can be made intermittent by means of a time clock or actuated by a preset differential head loss across the screen.

The reciprocating catenary and cable-operated mechanisms have different operations of the raking mechanisms and are discussed in Ref. 2. For detailed information on each system, equipment manufacturers should be contacted (see Appendix D).

8-3-4 Quantities and Composition of Screenings

The quantity of screenings depends on type of wastewater, geographic location, weather conditions, and type and size of screens. The quantity of screenings removed by bar racks vary from 3.5 to 80 $m^3/10^6\ m^3$ (0.5–11 ft^3/million gallons), with an average of about 20 $m^3/10^6\ m^3$ (2.7 ft^3/million gallons).[2] The average and maximum amounts of screenings collected from mechanically cleaned bar racks with respect to size of the opening are shown in Figure 8-4.

The screenings contain approximately 80 percent moisture and normally weigh 960 kg/m^3 (60 lb/ft^3). The screenings are odorous and attract flies. Disposal of screenings is achieved by landfilling or incineration. Often, screenings are discharged into grinders where they are ground and returned to the wastewater treatment plant.

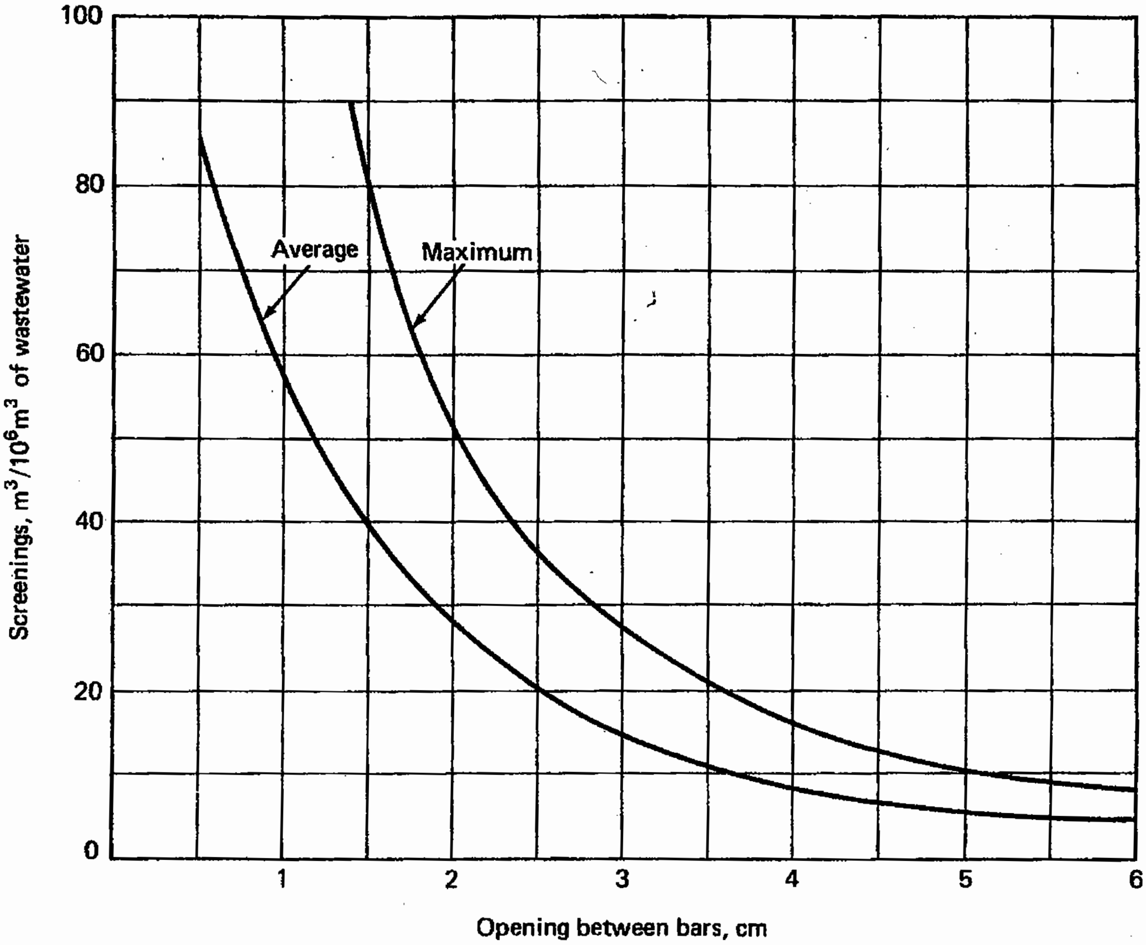


Figure 8-4 Quantities of Screenings Collected from Mechanically Cleaned Bar Racks (courtesy Envirex, U.S. Filter).

8-4 EQUIPMENT MANUFACTURERS OF BAR RACKS

A number of equipment manufacturers supply mechanically cleaned bar racks. The names and addresses of many manufacturers of screening devices are given in Appendix D. The design engineer must make many decisions in advance of selecting the equipment. Information such as width and water depth in the channel, clear spacings between the bars, velocity through screens, type of cleaning equipment (that is, front-cleaned or back-cleaned), etc., are necessary for making these decisions. Once these decisions are made, the design engineer must work in cooperation with the equipment manufacturers to select the equipment best suited for the project needs. Important considerations for equipment selection are also covered in Sec. 2-10.

8-5 INFORMATION CHECKLIST FOR DESIGN OF BAR RACK

Bar rack design should not be started until many important design decisions are made and the necessary data developed. The following is a listing of important items:

1. Flow data including peak wet weather, peak dry weather, and average design flows
2. Hydraulic and design data for the influent conduit

3. Treatment plant design criteria prepared by the concerned regulatory agencies
4. Velocities through the bars
5. Equipment manufacturers and equipment selection guide (catalog)
6. Information on existing facility if the plant is being expanded
7. Existing site plan with contours
8. Bar spacings and the head loss constraints through the rack and through the entire plant
9. Velocities through the screen chamber

8-6 DESIGN EXAMPLE

8-6-1 Design Criteria Used

The following design criteria are used for the design of bar racks:

1. Provide two identical bar racks, each capable of handling maximum flow conditions and each equipped with a mechanical cleaning device, $\theta = 75°$.
2. One screen chamber could be taken out of service for routine maintenance without interrupting the normal plant operation.
3. Bar spacing (clear) = 2.5 cm
4. Peak design wet weather flow = 1.321 m^3/s
   Maximum design dry weather flow = 0.916 m^3/s
   Average design dry weather flow = 0.441 m^3/s
5. Provide approximately the following velocities through the rack at different flows:
   - Velocity through rack at peak design wet weather flow equals 0.9 m/s.
   - Velocity through rack at maximum design dry weather flow should not be less than 0.6 m/s.
   - Velocity through rack at average design dry weather flow should not be less than 0.4 m/s.

8-6-2 Design Calculations

Step A: Flow Conditions in the Incoming Conduit

1. Diameter of the conduit = 1.53 m (Table 7-6)
2. Slope of the conduit = 0.00047 (Table 7-6)
3. Velocity at peak design flow, v = 0.87 m/s (Table 7-6)
4. Depth of flow in the conduit at peak design flow, d = 1.18 m (Table 7-6)

Step B: Design of Rack (Screen) Chamber

1. Compute bar spacings and the dimensions of the bar rack chamber.
   The rack chamber is designed for peak wet weather flow. The velocities through the rack and channel, and depth of flow in the channel, are also checked for average and minimum design flows.

Assume that the depth of flow in the rack chamber is the same as that in the incoming conduit (1.18 m).

a. Clear area through the rack openings $= \dfrac{\text{peak design flow}}{\text{velocity through rack}}$

req'd clear area

$$= \frac{1.321 \text{ m}^3/\text{s}}{0.9 \text{ m/s}} = 1.47 \text{ m}^2$$

b. Clear width of the opening at the rack $= \dfrac{\text{area}}{\text{depth of flow}}$

$$= \frac{1.47 \text{ m}^2}{1.18 \text{ m}} = 1.25 \text{ m}$$

c. Provide 50 clear spacings at 25 mm

d. Total clear openings in the rack chamber $= 50 \times 25\text{mm} \times \dfrac{1}{1000 \text{ mm/m}} = 1.25 \text{ m}$

e. Total number of bars $= 49$

f. Provide bars with 10 mm width

g. Width of the chamber $= 1.25 \text{ m} + 10 \text{ mm} \times 49 \times \dfrac{1}{1000 \text{ mm/m}}$

$$= 1.74 \text{ m}$$

h. The plan, longitudinal section, the cross section of the chamber including datum, water surface at peak design flow, bar arrangement, and spacings are shown in Figure 8-5.

2. Calculate the efficiency coefficient.

Efficiency coefficient $= \dfrac{\text{clear opening}}{\text{width of the chamber}}$

$$= \frac{50 \times 25 \text{ mm}}{1740 \text{ mm}} = 0.72$$

(Many manufacturers for a given clear spacing and bar thickness give an efficiency coefficient. This coefficient may also be used to calculate the width of the chamber.)

3. Compute the actual depth of flow and velocity in the rack chamber at peak design flow.
   a. The actual depth of flow in the rack chamber upstream of the bar rack is calculated by using the energy equation. Energy equation with respect to two sections of a channel is expressed by Eq. (8-4) [see sections (1) and (2) in Figure 8-5(b)]

$$Z_1 + d_1 + \frac{v_1^2}{2g} = Z_2 + d_2 + \frac{v_2^2}{2g} + h_L \qquad (8\text{-}4)$$

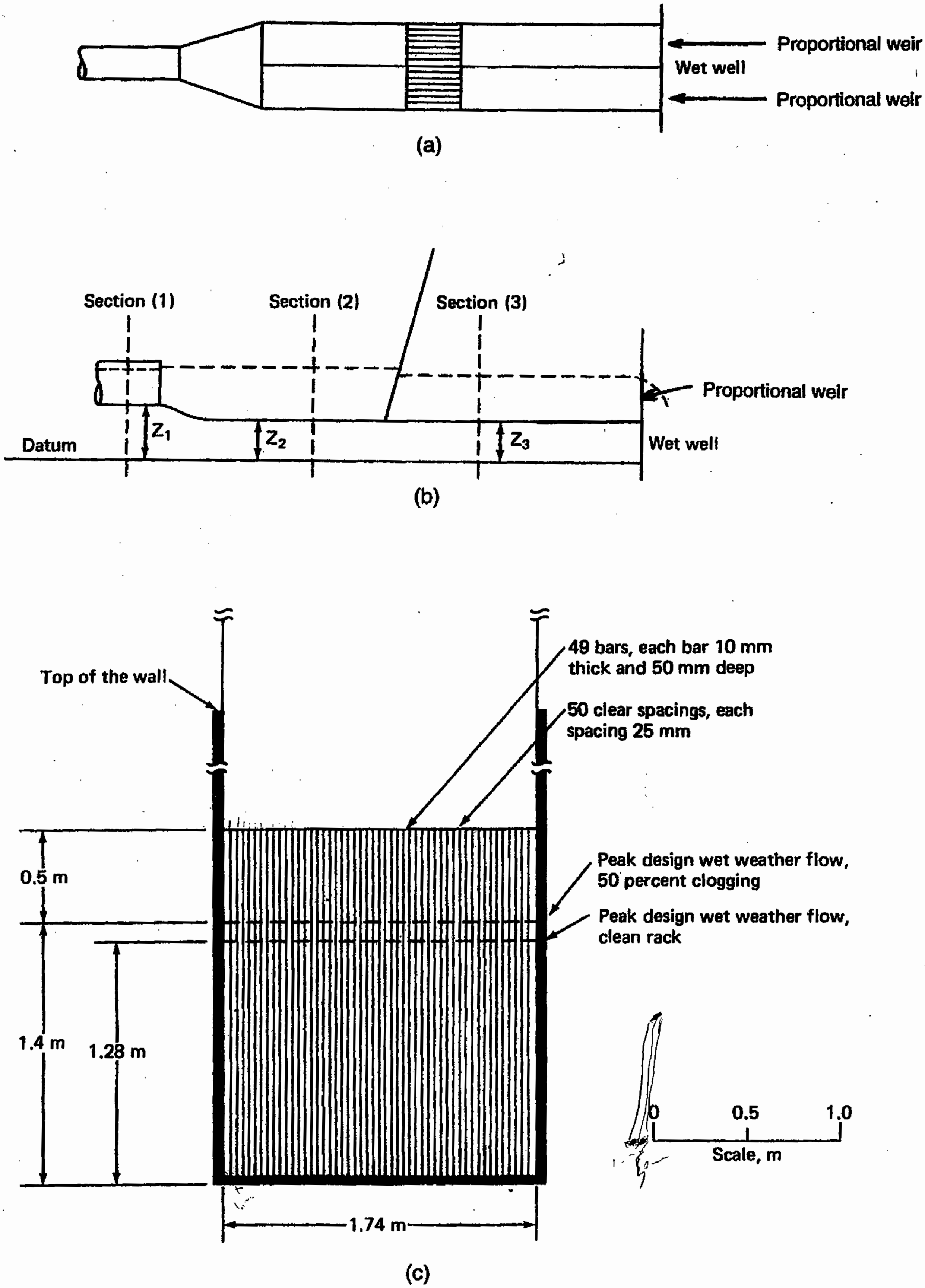


Figure 8-5 Details of Rack Chamber: (a) plan; (b) longitudinal section through the rack chamber; and (c) cross section showing bar arrangement, channel section, and depths of flow.

where

$$
\begin{aligned}
Z_1 \text{ and } Z_2 &= \text{height above datum, m} \\
v_1 \text{ and } v_2 &= \text{velocity at sections (1) and (2), m/s} \\
h_L &= \text{total head loss, m} \\
d_1 \text{ and } d_2 &= \text{depth of flow at sections (1) and (2), m}
\end{aligned}
$$

In this example

$$h_L = K_e \left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g} \right) \tag{8-5}$$

where

K_e = coefficient of expansion (see Appendix B). The value of K_e is dependent upon the configuration of the approach section.

b. Using the energy equation between section (1) in the incoming conduit and section (2) in the rack chamber upstream of the rack [Figure 8-5(b)], the actual depth of flow and velocity in the chamber are determined. The energy equation is written using the following conditions:
   - The floor of the chamber is horizontal.
   - Reference datum is at the floor of the chamber ($Z_2 = 0$).
   - The invert of the incoming conduit is 8 cm above the reference datum
   - $K_e = 0.3$.

$$
\begin{aligned}
0.08\text{ m} + 1.18\text{ m} + \frac{(0.87\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} \\
&= d_2\text{ m} + \frac{\left(\dfrac{1.321\text{ m}^3/\text{s}}{1.74\text{ m} \cdot d_2\text{ m}} \right)^2}{2 \times 9.81\text{ m/s}^2} \\
&\quad + 0.3 \left[\frac{(0.87\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} - \frac{\left(\dfrac{1.321\text{ m}^3/\text{s}}{1.74\text{ m} \cdot d_2\text{ m}} \right)^2}{2 \times 9.81\text{ m/s}^2} \right]
\end{aligned}
$$

c. Simplifying this equation:

$$d_2^3 - 1.288 d_2^2 + 0.021 = 0$$

d. Solving by trial and error:

$$d_2 = 1.28\text{ m, and } v_2 = \frac{1.321\text{ m}^3/\text{s}}{1.74\text{ m} \times 1.28\text{ m}} = 0.59\text{ m/s}$$

4. Compute velocity V through the clear openings at the bar rack:

$$
\begin{aligned}
V &= \frac{\text{flow}}{\text{net area at the rack}} = \frac{1.321\text{ m}^3/\text{s}}{1.25\text{ m}^{a} \times 1.28\text{ m}} \\
&= 0.83\text{ m/s}
\end{aligned}
$$

[a] 50 spaces × 0.025 m clear spacings = 1.25 m.

The actual velocity through the rack at peak design flow is 0.83 m/s. This is slightly less than the design value of 0.9 m/s but is still in the permissible range (Table 8-3). Some designers may redesign the rack with 48 or 49 clear openings. The width of the chamber will be reduced and higher velocity through the screen will be encountered (see Problem 8-2, Sec. 8-9).

5. Compute head loss through the bar rack.
The head loss through the bar rack for clean condition is calculated from Eq. (8-1) or (8-2). The head loss calculations from these equations are given below:

$$h_L \text{ [using Eq. (8-1)]} = \frac{(0.83 \text{ m/s})^2 - (0.59 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2} \times \frac{1}{0.7} = 0.025 \text{ m}$$

$$h_L \text{ [using Eq. (8-2)]} = 2.42 \times \left(\frac{49 \times 10 \text{ mm}}{50 \times 25 \text{ mm}}\right)^{4/3}$$

$$\times \frac{(0.83 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2} \times \sin 75°$$

$$= 0.024 \text{ m}$$

6. Compute the depth of flow and velocity in the rack chamber below the rack.
   a. The depth and velocity in the chamber is calculated by trial and error from the energy equation:

$$d_2 + \frac{v_2^2}{2g} = d_3 + \frac{v_3^2}{2g} + h_L$$

where

d_2 and v_2 = depth of flow and velocity in the chamber upstream of the rack
d_3 and v_3 = depth of flow and velocity in the chamber downstream of the rack
h_L = head loss through the rack

$$1.28 \text{ m} + \frac{(0.59 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2} = d_3 \text{ m} + \frac{\left(\dfrac{1.321 \text{ m}^3\text{/s}}{1.74 \text{ m} \cdot d_3 \text{ m}}\right)^2}{2 \times 9.81 \text{ m/s}^2} + 0.025 \text{ m}$$

   b. Simplifying this equation:

$$d_3^3 - 1.273\, d_3^2 + 0.029 = 0$$

   c. Solving this equation by trial and error, $d_3 = 1.25$ m and $v_3 = 0.61$ m/s.

7. Compute the head loss through the rack at 50 percent clogging.
   a. At 50 percent clogging of the rack, the clear area through the rack is reduced to half and the head loss through the rack is obtained from the energy equation.

$$d_2' + \frac{v_2'^2}{2g} = d_3 + \frac{v_3^2}{2g} + h_{50}$$

where

d_2' and v_2' = depth of flow and velocity in the chamber upstream of the rack at 50 percent clogging

h_{50} = head loss through the rack at 50 percent clogging

b. The velocity and depth of flow in the channel below the rack are governed by the condition in the outlet channel (the channel that carries the screened wastewater). Since the conditions in the outlet channel do not change, it can be assumed that d_3 and v_3 are the same as calculated for the clean rack.

c. $$h_{50} = \frac{(\text{velocity through rack opening})^2 - v_2'^2}{2g} \times \frac{1}{0.7}$$

d. $$\text{Velocity through rack openings at 50 percent clogging} = \frac{1.321\ \text{m}^3/\text{s}}{1.25\ \text{m} \times 0.5 \times d_2'\ \text{m}} = \frac{2.114}{d_2'}\ \text{m/s}$$

e. $$v_2' = \frac{1.321\ \text{m}^3/\text{s}}{1.74\text{m} \times d_2'\ \text{m}} = \frac{0.759}{d_2'}\ \text{m/s}$$

f. $$d_2'\ \text{m} + \frac{\left(\dfrac{0.759}{d_2'}\ \text{m/s}\right)^2}{(2 \times 9.81\ \text{m/s}^2)}$$

$$= 1.25\ \text{m} + \frac{(0.61\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2}$$

$$+ \frac{\left[\left(\dfrac{2.114}{d_2'}\ \text{m/s}\right)^2 - \left(\dfrac{0.759}{d_2'}\ \text{m/s}\right)^2\right]}{2 \times 0.7 \times 9.81\ \text{m/s}^2}$$

g. Simplifying this equation:

$$d_2'^3 - 1.269 d_2'^2 - 0.254 = 0$$

h. Solving this equation by trial and error:

$$d_2' = 1.40\ \text{m}$$

$$v_2 = \frac{1.321\ \text{m}^3/\text{s}}{1.74\ \text{m} \times 1.40\ \text{m}}$$

$$= 0.54\ \text{m/s}$$

i. The head loss under 50 percent clogging:

$$h_{50} = 1.40\ \text{m} - 1.25\ \text{m}$$
$$= 0.15\ \text{m}^{b}$$

[b] h_{50} can also be calculated from Eq. (8-1):

$$h_{50} = \frac{(1.51\ \text{m/s})^2 - (0.54\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2 \times 0.7} = 0.15\ \text{m}$$

TABLE 8-4 Summary of Depth of Flow, Velocity, and Head Loss through Bar Rack at Design Peak Flow

Conditions	Upstream Channel Depth of Flow (m)	Upstream Channel Velocity (m/s)	Velocity through Rack (m/s)	Downstream Channel Depth of Flow (m)	Downstream Channel Velocity (m/s)	Head Loss (m)
Clean rack	1.28	0.59	0.83	1.25[a]	0.61[a]	0.03[b]
50 percent clogging	1.40	0.54	1.51	1.25	0.61	0.15

[a]See Sec. 8-6-2, Step B, 6, IIc.
[b]Actual head loss = 0.025 m (see Sec. 8-6-2, Step B, 5).

j. Velocity V' through rack openings:

$$V' = \frac{2.114}{1.40}\ \text{m/s}$$
$$= 1.51\ \text{m/s}$$

k. Many designers use approximate analysis. When the screen openings become half-plugged, the area of flow is assumed to be half, and velocity through the screen is doubled:

$$h_{50} = [(2 \times 0.83\ \text{m/s})^2 - (0.59\ \text{m/s})^2]\,\frac{1}{2 \times 9.81\ \text{m/s}^2 \times 0.7}$$
$$= 0.18\ \text{m}$$

The head loss calculated by this method is slightly larger than that previously obtained (0.15 m). Higher allowance of head loss gives more conservative design.

l. The increase of head loss is about 15 cm (5.9 in.) as the screen becomes plugged. The need for accurate control of the clean-up cycle and protection against surge loads is thus demonstrated. A freeboard of over 0.5 m is normally provided for protection against flooding and overflow. In this design the bar screen is deep in the ground, and therefore flooding and overflow is not a consideration.

m. A summary of depth of flow, velocity, and head loss through the bar rack under clean and 50 percent clogging is given in Table 8-4.

Step C: Effluent Structure. In the above calculations the depth of flow and velocity at sections (1), (2), and (3) [Figure 8-5(b)] were calculated by assuming normal flow conditions in the channel. Since the flow from the bar screen has a free fall into the wet well, the actual depth into the channel will be significantly smaller than the normal depths calculated earlier. Furthermore, the velocities through the screen will also be significantly larger because of smaller depth in the channel. It is therefore important to design a control section at the effluent end of the screen chamber to maintain normal depth. In the earlier edition of this book, the bottom of the channel was raised in order to maintain the previously calculated depths into the screen chamber. The calculation procedure utilized critical depth equation for rectangular channel. The procedure for designing the channel with a raised floor may be found in Ref. 8. Raising the floor of the channel would encourage deposition of solids during low flows. A better design would be to provide a *proportional weir* or *Parshall flume.* This topic is discussed in detail in Step C, 2. The critical depth and velocity calculations are very common in treatment plant design. Therefore, Eq. (8-6) is introduced here for these applications.[9] The procedure for calculating the critical depth in a rectangular channel may be found in Chapters 11, 12, and 13. Problems 8-5 and 8-6 in Sec. 8-9 also utilize critical depth and critical velocity calculations.

$$Q = A_c \sqrt{g\, d_c} = b\sqrt{g}\, d_c^{3/2} \tag{8-6}$$

where

d_c = critical depth, m
b = width of the channel, m
A_c = area of cross section at critical depth, m^2
Q = flow in the channel, m^3/s

1. Utilize the free fall.
   A proportional weir or Parshall flume is specially beneficial as a control section if a free fall is available on the downstream side. In this design a free fall into the wet well is necessary; therefore, a proportional weir or a Parshall flume will be beneficial. A Parshall flume requires converging and diverging sections, which may not be practical in this case. A proportional weir will be ideal because of the following reasons: (a) no converging or diverging sections are needed, (b) raising of the floor may not be necessary, (c) the velocity in the channel and through the screen will remain fairly uniform even at lower flows, and (d) the head over the proportional weir can be calibrated for flow measurement. Therefore, a proportional weir at the effluent end of the bar screen channel is selected in this design. The theory and design of the proportional weir is given below. The design procedure for a Parshall flume is shown in Chapter 14.
2. Select equation for designing proportional weir.
   A proportional weir is used to maintain a uniform velocity in a channel at variable flow conditions. It is a combination of a weir and orifice. The weir crest is straight edge, and the sides of the orifice are so curved that the cross-sectional area of the orifice diminishes by three-halves power of the head over the crest. The flow through a proportion weir is given by Eq. (8-7).[10c]

$$Q = 1.57 C_d \sqrt{2g}\, L H^{3/2} \tag{8-7}$$

where

Q = flow through the proportional weir, m^3/s
H = head over weir, m
C_d = coefficient of discharge = 0.6–0.9. Because of slime growth and obstructions, a commonly used value is 0.6

[c]Other design equations of proportional weir and Sutro weir may be found in Refs. 2, 3, and 10–12.

L = length of the weir opening at a height H above the weir crest, m
g = acceleration due to gravity = 9.81 m/s^2

Substituting the values of C_d and g Eq. (8-7) is transformed into Eq. (8-8).

$$Q = 4.173\,[LH^{1/2}]H \qquad (8\text{-}8)$$

3. Develop design of proportional weir.
In order that a nearly uniform velocity be maintained through the screen chamber under variable-flow conditions, the depth of flow in the chamber must be proportional to the flow through the chamber or the head H over the weir. This is achieved by keeping the factor $LH^{1/2}$ in Eq. (8-7) constant. The design calculations of the proportional weir, as well as the velocity through the channel, are summarized below:
   a. Compute the length of the proportional weir opening at peak design flow.
   Most equipment manufacturers require a minimum depth of water in the channel below the screen. The purpose is to keep the bearings lubricated. Therefore, set the weir crest 15 cm above the channel floor. The maximum depth in the channel = d_3 or 1.25 m. Since the crest of the weir is kept 0.15 m above the floor of the channel, the maximum head over the proportional weir at peak design flow is 1.10 m (1.25 m − 0.15 m). The length of the weir opening L at the maximum head over the weir is calculated from Eq. (8-8):

$$L = \frac{Q}{4.173\,H^{3/2}} = \frac{1.321\ \text{m}^3/\text{s}}{4.173\,(1.10)^{3/2}} = 0.27\ \text{m}$$

   b. Compute the geometric profile of the proportional weir.
   The design details of the proportional weir are provided in Table 8-5. In these calculations the factor $LH^{1/2}$ at different sections of the proportional weir is kept constant.

$$LH^{1/2} = 0.27 \times \sqrt{1.10} = 0.283$$

   The ends of the weir are cut off at a height of 5 cm above the crest. The weir crest may be lowered slightly to compensate for the end area of the proportional weir that is lost due to the ends of the weir being cut off. The design details of the proportional weir are shown in Figure 8-6(a).

Step D: Hydraulic Profile through the Bar Rack. The hydraulic profile through the bar rack at peak design flow and design of proportional weir are illustrated in Figure 8-6. The elevation of the channel floor is assumed to be 0.00. A total of 0.03-m head loss occurs at peak design flow when the rack is clean. At 50 percent clogging, the total head loss through the rack is 0.15 m. These values may be found in Table 8-4.

Step E: Quantity of Screening. The quantity of screenings is obtained from Figure 8-4. For clear spacing of 2.5 cm, the average amount of screenings produced at the design average and peak flows are 20 and 36 m^3 per million m^3 of the flow. The design average flow is 0.411 m^3/s.

TABLE 8-5 Design Calculations of Proportional Weir and Depth and Velocity at Different Flow Conditions

Total Flow (m^3/s)	Flow Condition	Head over Weir (m)	Weir Length (m)	Depth in the Channel (m)	Velocity in the Channel (m/s)
1.321[a]	Peak design flow	1.10[b]	0.27[c]	1.25	0.61[d]
0.916[a]	Maximum design dry weather flow	0.78[e]	0.32[f]	0.93[g]	0.57[h]
0.441[a]	Average design dry weather flow	0.37	0.47	0.52	0.49
0.220[a]	Minimum design dry weather flow	0.19	0.67	0.34	0.37
0.152[a]	Minimum flow, initial year	0.13	0.79	0.28	0.31
0.05[i]	Flow to calculate the length of weir crest	0.05	1.27	0.20	0.14

[a]These flows are given in Table 6-9.
[b]Head over weir = maximum depth of flow in the channel, minus the height of the weir crest above the channel floor (see Step C, 3, a).
[c]Calculated above.
[d]1.321 m^3/s/ 1.74 m × 1.25 m = 0.61 m/s. This is same as v_3 in Table 8-4.

[e]$H_1 = \dfrac{0.916}{4.173\ LH^{1/2}} = \dfrac{0.916}{4.173 \times 0.283} = 0.78$ m

[f]$L_1 = \dfrac{LH^{1/2}}{H_1^{1/2}} = \dfrac{0.283}{(0.78)^{1/2}} = 0.32$ m

[g]Depth in the channel = 0.78 m + 0.15 m = 0.93 m

[h]Velocity in the channel = $\dfrac{0.916\ m^3/s}{1.74\ m \times 0.93\ m} = 0.57$ m/s

[i]This flow is not expected to occur.

1. Estimate the quantity of screenings at average flow.

$$\text{Average quantity of screenings} = 20\ m^3/10^6\ m^3 \times 0.441\ m^3/s \times 3600\ s/h \times 24\ h/d = 0.76\ m^3/d$$

2. Estimate the quantity of screenings at maximum flow.

$$\text{Maximum quantity of screenings} = 36\ m^3/10^6\ m^3 \times 0.441\ m^3/s \times 3600\ s/h \times 24\ h/d = 1.37\ m^3/d$$

Step F: Disposal of Screenings. The screenings will be disposed of by sanitary landfilling. General discussion, design details, land requirements, and equipment selection for sanitary landfill are provided in Chapter 19.

Step G: Design Details. The design details of the bar rack are provided in Figure 8-7.

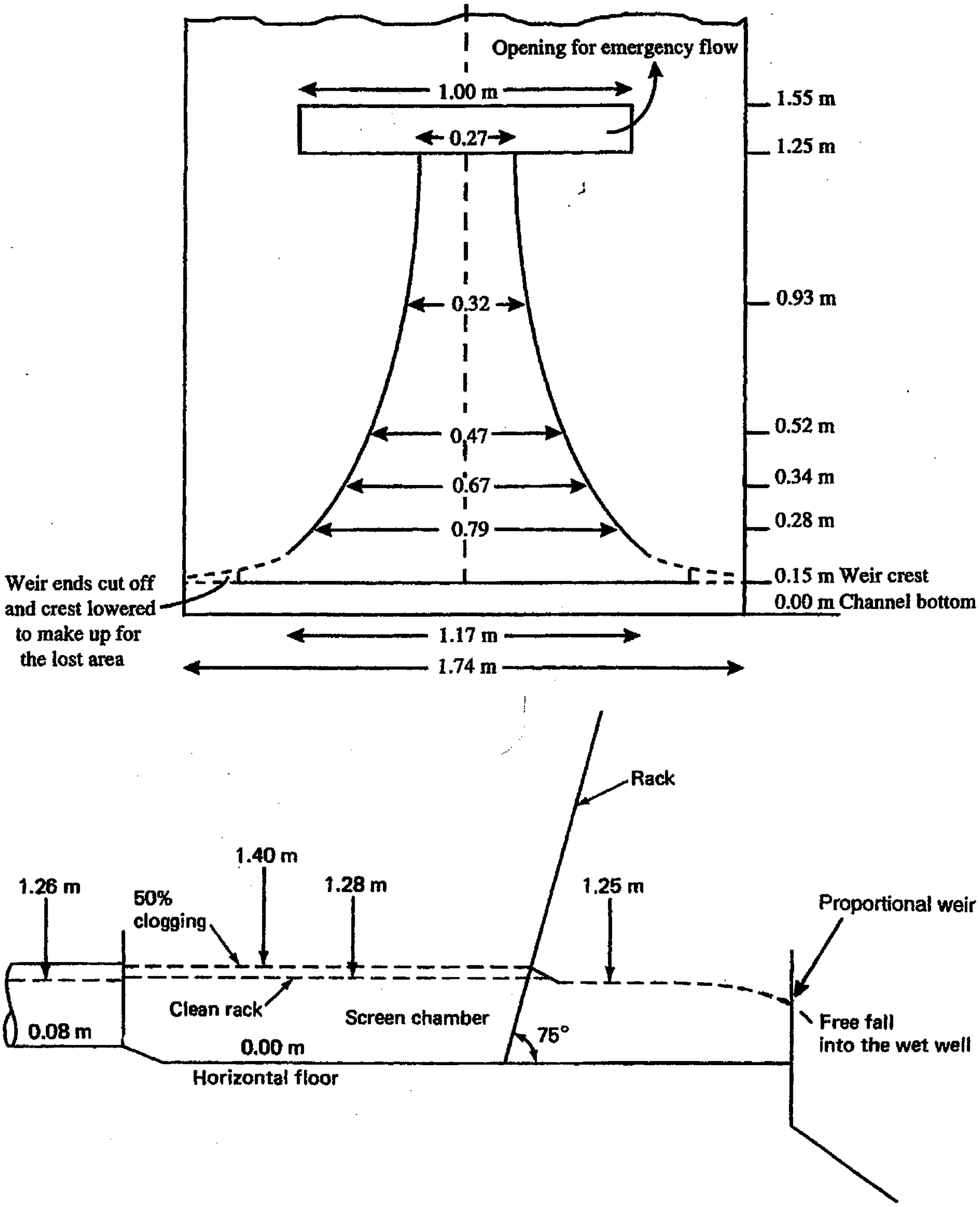


Figure 8-6 Proportional Weir and Hydraulic Profile: (a) design details of the proportional weir and (b) hydraulic profile through the bar rack at peak design flow when rack is clean and at 50 percent clogging. All Elevations Are with Respect to the Datum at the Floor of the Rack Chamber.

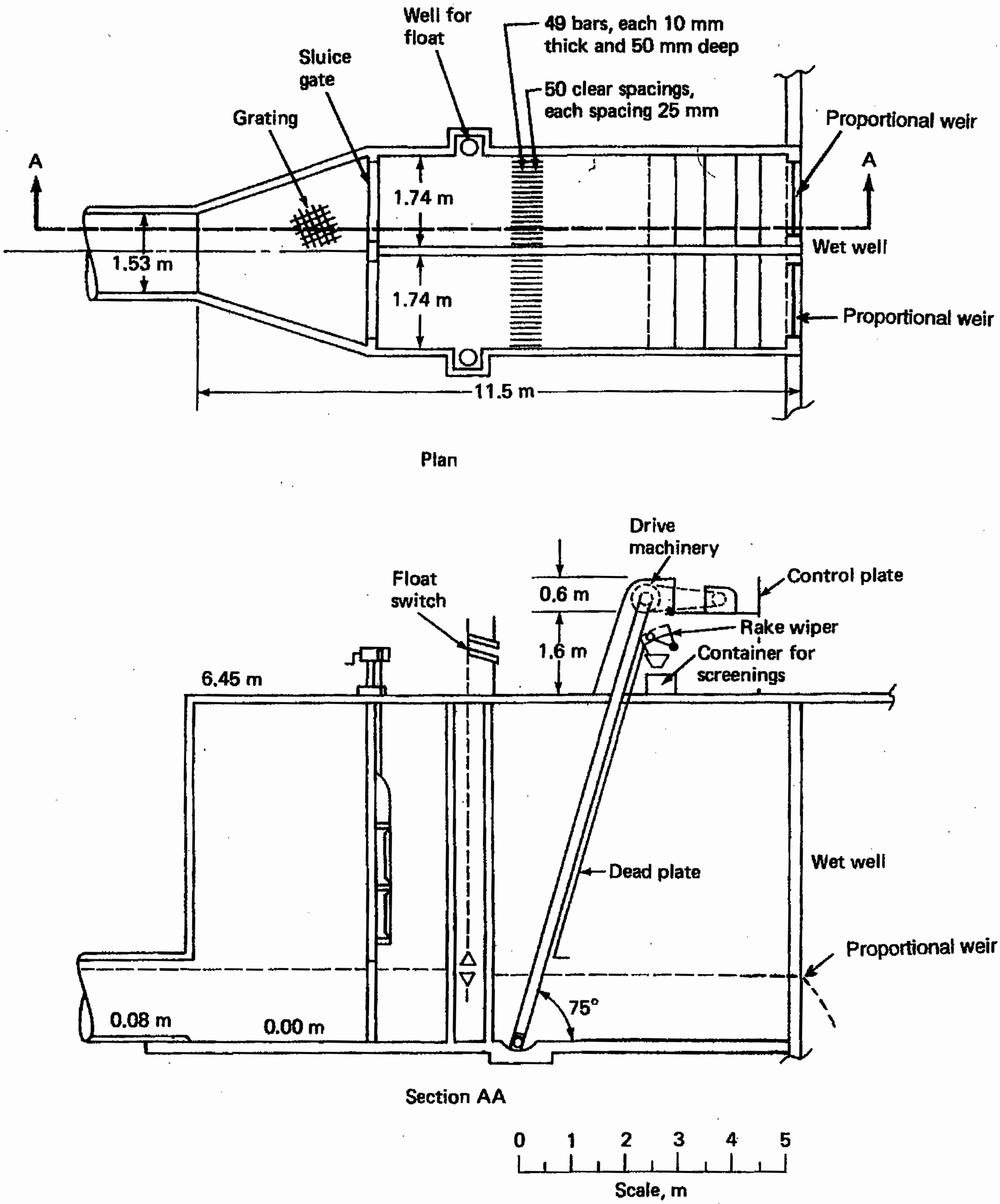


Figure 8-7 Design Details of Bar Rack. All Elevations Are with Respect to the Datum at the Floor of the Rack Chamber.

8-7 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT MECHANICALLY CLEANED BAR RACK

Debris in wastewater, if not removed properly, will damage equipment and interfere with normal operation of treatment processes. Often, high repair records of many processes and equipment may be attributed to poor performance of the bar rack. Common operation and maintenance problems that may develop at a bar rack and the procedure to correct them are summarized below.

8-7-1 Common Operating Problems and Suggested Solutions

The following items may be considered as troubleshooting guides:[8,13,14]

1. Obnoxious odors, flies, and other insects around the bar rack indicate prolonged storage of screenings at the facility. Increase frequency of removal and disposal of screenings.
2. Excessive screen clogging is an indication of (a) an unusual amount of debris in the wastewater, (b) low velocity through the rack, or (c) the automatic clock-operated screen rakes do not remove the debris fast enough. Possible solutions include identifying the source of waste causing excessive discharge of debris and stopping it at the source; providing a coarser rack; and resetting the timer cycle or installing level controller override.
3. Excessive grit accumulation in the chamber is due to low velocity in the channel. The possible solutions to this problem are remove bottom irregularity, reslope the bottom, rake the channel, or flush regularly with a hose.
4. A jammed raking mechanism may render the mechanical rake inoperable, and the circuit breaker will not reset. Remove the obstruction immediately.
5. A broken chain or cable, or a broken limit switch will render the rake inoperable, but the motor will run. Inspect chain and switches, and replace them as necessary.
6. A defective remote control circuit or motor will render the rake inoperable without any visible problem. Check remote control circuit and motor and replace them as necessary.

8-7-2 Facility Maintenance

Screenings removed from municipal wastewater treatment plants are odorous and attract flies and insects. The screenings should be stored in covered containers and hauled daily for disposal. The area should be thoroughly hosed off daily with chemical solution (chlorine or hydrogen peroxide).

The bar screen raking mechanism (raking chain; sprocket, teeth, and other moving parts) should be inspected daily. All moving parts should be lubricated and adjusted as recommended by the manufacturer.

Each bar screen should be taken out of service for maintenance on a routine basis. The unit should be dewatered and components checked for painting; cable, chain, or teeth replacement; removal of obstructions; straightening of bent bars; and so forth.

8-8 SPECIFICATIONS[d]

Brief specifications of several components of bar racks are presented in this section. The purpose of these specifications is to describe many components that could not be fully covered in the Design Example. These specifications should only be used as a guide for understanding the equipment.

8-8-1 General

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Each unit shall consist of bar rack, side frame, cleaning rakes, dead plate, rake-cleaning device, head section, and drive unit. The manufacturer shall have sufficient experience in the manufacture of mechanically cleaned bar racks of a similar size and design.

8-8-2 Bar Rack

The bar rack shall be inclined 75° from the horizontal and shall be held firmly in channel. The steel bars (10 mm thick and 50 mm wide) formed straight and true are held firmly and accurately in place with 25-mm clear openings by means of welded spacers at each end. The bar rack shall extend from the bottom of the channel to a height of 0.5 m above the maximum water level.

8-8-3 Cleaning Rake

The cleaning rake shall be front-cleaned jam proof,[e] entering the bar rack from the upstream side, mounted on two strands of chain running over two sets of sprocket wheels at a speed of about 2 rpm. The cleaning rakes shall be of steel plate with teeth of suitable shape to effectively clean the top and side of the bars. The screenings shall be conveyed over dead plate and deposited into a suitable receptacle within the head section.

8-8-4 Wiper and Dead Plate

An automatic rake wiper shall be provided for the screen, designed and placed so that the screenings do not wrap around the rake or the wiper during screening removal. The dead plate shall be constructed of at least 6-mm-thick steel, shall be bolted to the side

[d]Adapted in part from Refs. 3–8, 13, and 14.

[e]Some designers prefer back-cleaning devices because they have less tendency to jam at the bottom.

frames of the screen, and shall extend from a point 23 cm above maximum liquid level to 75 cm below the center line of the dead shaft in the enclosed housing. A 6-mm-thick steel lip shall be provided at the discharge point of the dead plate.

8-8-5 Screen Chains, Sprockets, Shafting, and Drive Unit

Screen chains shall be of specified ultimate strength with proper attachment links, heat-treated steel pins, and rivets.

The sprockets for drive and screen chains shall be semisteel, cast in a chill of specified hardness and number of teeth, and accurately ground to fit the chain. The driving sprockets shall be keyed firmly to shafts, and the foot shaft sprocket shall be suitable for underwater service under gritty conditions. All shafting shall be of steel, straight and true, of sufficient size, and properly keyed to transmit power as required.

The drive unit shall consist of a speed reducer and an electric motor assembly with each bar screen having a separate drive unit. The motor shall be totally enclosed, fan-cooled, and with a constant speed of ample power for starting and operating the mechanism under normal operating conditions without overloading. The motor shall conform to the specified volt, phase, and cycle current requirements.

8-8-6 Control System

The mechanically cleaned bar screens shall be operated by an independent, motor-driven time clock adjustable to give cycles ranging from 0 to 150 min. The control panel shall be enclosed and shall contain starters, circuit breaker, control relays, and transformers. In addition, the control panel shall have the following pushbuttons: hoist, lower, reset, manual, automatic, stop, and start, for proper operation and resetting. A float shall be provided in a stilling well upstream of the rack for an automatic high water level override to start the raking device should the liquid in the channel reach a predetermined depth.

8-9 PROBLEMS AND DISCUSSION TOPICS

8-1 All calculations in the Design Example were performed at the peak design flow of 1.321 m^3/s. Check the depth of flow and velocity in the chamber upstream and downstream of the bar rack at the average design flow of 0.441 m^3/s. Also calculate the velocity through the clear openings at the bar rack and compare your result with the design criteria given in Sec. 8-6-1. Use the screen chamber dimensions given in Figure 8-5.

8-2 Redesign the bar rack in the Design Example using 47 bars instead of 49. Calculate the velocity through the clear openings at the bar rack and compare your results with those calculated in the Design Example.

8-3 The average design flow at a wastewater treatment facility is 1.0 m^3/s. The bar rack is mechanically cleaned with an average clear spacing of 1.5 cm. Estimate the average quantity of screenings that will be collected each day.

8-4 A bar rack precedes a grit channel. The outlet of the grit channel is controlled by a proportional weir such that the depth of flow in the channel is directly proportional to the rate of flow. A 91-cm (36-in.) intercepting sewer brings the flow to the screen chamber. The depth of flow and velocity in the sewer at peak design flow of 0.43 m^3/s are 57.5 cm and 1.0 m/s, respectively. The bar rack has 47 bars. Each bar is 6.4 mm (¼ in.) wide and clear spacing is 2.5 cm (1 in.). The floor of the bar rack chamber is horizontal and is 7.6 cm below the invert of the intercepting sewer. Calculate the depth and velocity in the bar rack chamber upstream and downstream of the rack and through the rack. Also calculate the head loss when the rack is clean and 50 percent clogged.

8-5 A rectangular channel has a width of 1.5 m and is flowing at a normal depth of 1 m at a section. The slope of the channel is 0.00032 m/m, and Manning's $n = 0.018$. Calculate the discharge. The channel has a free fall approximately 1 km from the section at which normal depth was observed. Calculate the critical depth and critical velocity that will be encountered near the free fall. Sketch the longitudinal section of the channel. Do not compute the backwater curve.

8-6 A rectangular bar rack chamber is 1.74 m wide. The water depth downstream of the rack is 1.25 m. The wastewater drops into a wet well. The chamber bottom is raised to maintain a nearly constant depth near the outfall. The bar screen is designed for a flow of 1.321 m^3/s. Calculate the height of the raised floor above the bottom of the channel. Draw the longitudinal section. (Note: The solution may be found in Ref. 8). Draw a sketch.

8-7 Design a proportional weir for velocity control in the bar-screen chamber. The screen chamber is 1.74 m wide, and the depth of flow downstream of the bars is 1.25 m when peak design flow of 1.321 m^3/s is passing through the chamber. The crest of the weir is at the bottom of the chamber. The proportional weir is located at the edge of the wet well so that there is a free fall into the wet well.

8-8 Calculate the depth of flow and velocity upstream of the bar rack in the Design Example. The flow through the bar rack is 0.916 m^3/s. The depth of flow and velocity in the chamber downstream of the rack and corresponding to this flow are given in Table 8-5. Assume clean screen and Eq. (8-1) applies for head loss calculation. Also calculate the velocity through the rack.

8-9 Calculate the depth of flow and velocity of the bar rack in Problem 8-7 when the flow in the chamber is 0.441 m^3/s. Use the dimensions of the proportional weir developed in Problem 8-3.

8-10 Repeat the calculations of Problem 8-8 when the flow in the chamber is 0.220 m^3/s.

8-11 Prepare a list of operation and maintenance problems at various wastewater treatment units that may be directly attributable to the screenings that may pass through a bar rack.

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L = length of the weir opening at a height H above the weir crest, m
g = acceleration due to gravity = 9.81 m/s^2

Substituting the values of C_d and g Eq. (8-7) is transformed into Eq. (8-8).

$$Q = 4.173\,[LH^{1/2}]H \tag{8-8}$$

3. Develop design of proportional weir.
In order that a nearly uniform velocity be maintained through the screen chamber under variable-flow conditions, the depth of flow in the chamber must be proportional to the flow through the chamber or the head H over the weir. This is achieved by keeping the factor $LH^{1/2}$ in Eq. (8-7) constant. The design calculations of the proportional weir, as well as the velocity through the channel, are summarized below:
   a. Compute the length of the proportional weir opening at peak design flow.
   Most equipment manufacturers require a minimum depth of water in the channel below the screen. The purpose is to keep the bearings lubricated. Therefore, set the weir crest 15 cm above the channel floor. The maximum depth in the channel = d_3 or 1.25 m. Since the crest of the weir is kept 0.15 m above the floor of the channel, the maximum head over the proportional weir at peak design flow is 1.10 m (1.25 m − 0.15 m). The length of the weir opening L at the maximum head over the weir is calculated from Eq. (8-8):

$$L = \frac{Q}{4.173\,H^{3/2}} = \frac{1.321\text{ m}^3/\text{s}}{4.173\,(1.10)^{3/2}} = 0.27\text{ m}$$

   b. Compute the geometric profile of the proportional weir.
   The design details of the proportional weir are provided in Table 8-5. In these calculations the factor $LH^{1/2}$ at different sections of the proportional weir is kept constant.

$$LH^{1/2} = 0.27 \times \sqrt{1.10} = 0.283$$

   The ends of the weir are cut off at a height of 5 cm above the crest. The weir crest may be lowered slightly to compensate for the end area of the proportional weir that is lost due to the ends of the weir being cut off. The design details of the proportional weir are shown in Figure 8-6(a).

Step D: Hydraulic Profile through the Bar Rack. The hydraulic profile through the bar rack at peak design flow and design of proportional weir are illustrated in Figure 8-6. The elevation of the channel floor is assumed to be 0.00. A total of 0.03-m head loss occurs at peak design flow when the rack is clean. At 50 percent clogging, the total head loss through the rack is 0.15 m. These values may be found in Table 8-4.

Step E: Quantity of Screening. The quantity of screenings is obtained from Figure 8-4. For clear spacing of 2.5 cm, the average amount of screenings produced at the design average and peak flows are 20 and 36 m^3 per million m^3 of the flow. The design average flow is 0.411 m^3/s.

TABLE 8-5 Design Calculations of Proportional Weir and Depth and Velocity at Different Flow Conditions

Total Flow (m^3/s)	Flow Condition	Head over Weir (m)	Weir Length (m)	Depth in the Channel (m)	Velocity in the Channel (m/s)
1.321[a]	Peak design flow	1.10[b]	0.27[c]	1.25	0.61[d]
0.916[a]	Maximum design dry weather flow	0.78[e]	0.32[f]	0.93[g]	0.57[h]
0.441[a]	Average design dry weather flow	0.37	0.47	0.52	0.49
0.220[a]	Minimum design dry weather flow	0.19	0.67	0.34	0.37
0.152[a]	Minimum flow, initial year	0.13	0.79	0.28	0.31
0.05[i]	Flow to calculate the length of weir crest	0.05	1.27	0.20	0.14

[a]These flows are given in Table 6-9.
[b]Head over weir = maximum depth of flow in the channel, minus the height of the weir crest above the channel floor (see Step C, 3, a).
[c]Calculated above.
[d]1.321 m^3/s/ 1.74 m × 1.25 m = 0.61 m/s. This is same as v_3 in Table 8-4.

[e] $$H_1 = \frac{0.916}{4.173\,LH^{1/2}} = \frac{0.916}{4.173 \times 0.283} = 0.78 \text{ m}$$

[f] $$L_1 = \frac{LH^{1/2}}{H_1^{1/2}} = \frac{0.283}{(0.78)^{1/2}} = 0.32 \text{ m}$$

[g]Depth in the channel = 0.78 m + 0.15 m = 0.93 m

[h]Velocity in the channel $= \dfrac{0.916 \text{ m}^3/\text{s}}{1.74 \text{ m} \times 0.93 \text{ m}} = 0.57$ m/s

[i]This flow is not expected to occur.

1. Estimate the quantity of screenings at average flow.

$$\begin{aligned}\text{Average quantity of screenings} &= 20 \text{ m}^3/10^6 \text{ m}^3 \times 0.441 \text{ m}^3/\text{s} \times 3600 \text{ s/h} \times 24 \text{ h/d} \\ &= 0.76 \text{ m}^3/\text{d}\end{aligned}$$

2. Estimate the quantity of screenings at maximum flow.

$$\begin{aligned}\text{Maximum quantity of screenings} &= 36 \text{ m}^3/10^6 \text{ m}^3 \times 0.441 \text{ m}^3/\text{s} \times 3600 \text{ s/h} \times 24 \text{ h/d} \\ &= 1.37 \text{ m}^3/\text{d}\end{aligned}$$

Step F: Disposal of Screenings. The screenings will be disposed of by sanitary landfilling. General discussion, design details, land requirements, and equipment selection for sanitary landfill are provided in Chapter 19.

Step G: Design Details. The design details of the bar rack are provided in Figure 8-7.

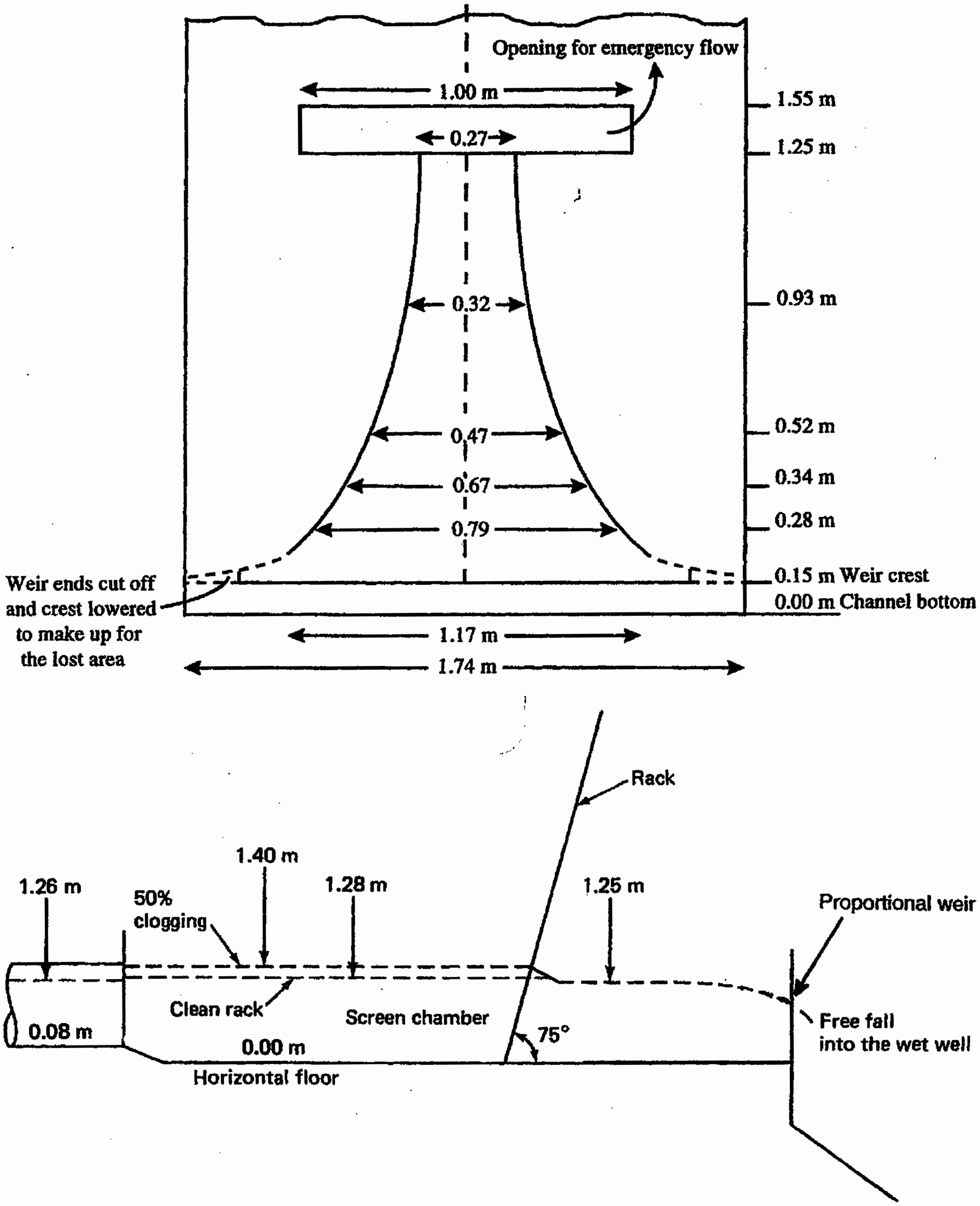

Figure 8-6 Proportional Weir and Hydraulic Profile: (a) design details of the proportional weir and (b) hydraulic profile through the bar rack at peak design flow when rack is clean and at 50 percent clogging. All Elevations Are with Respect to the Datum at the Floor of the Rack Chamber.

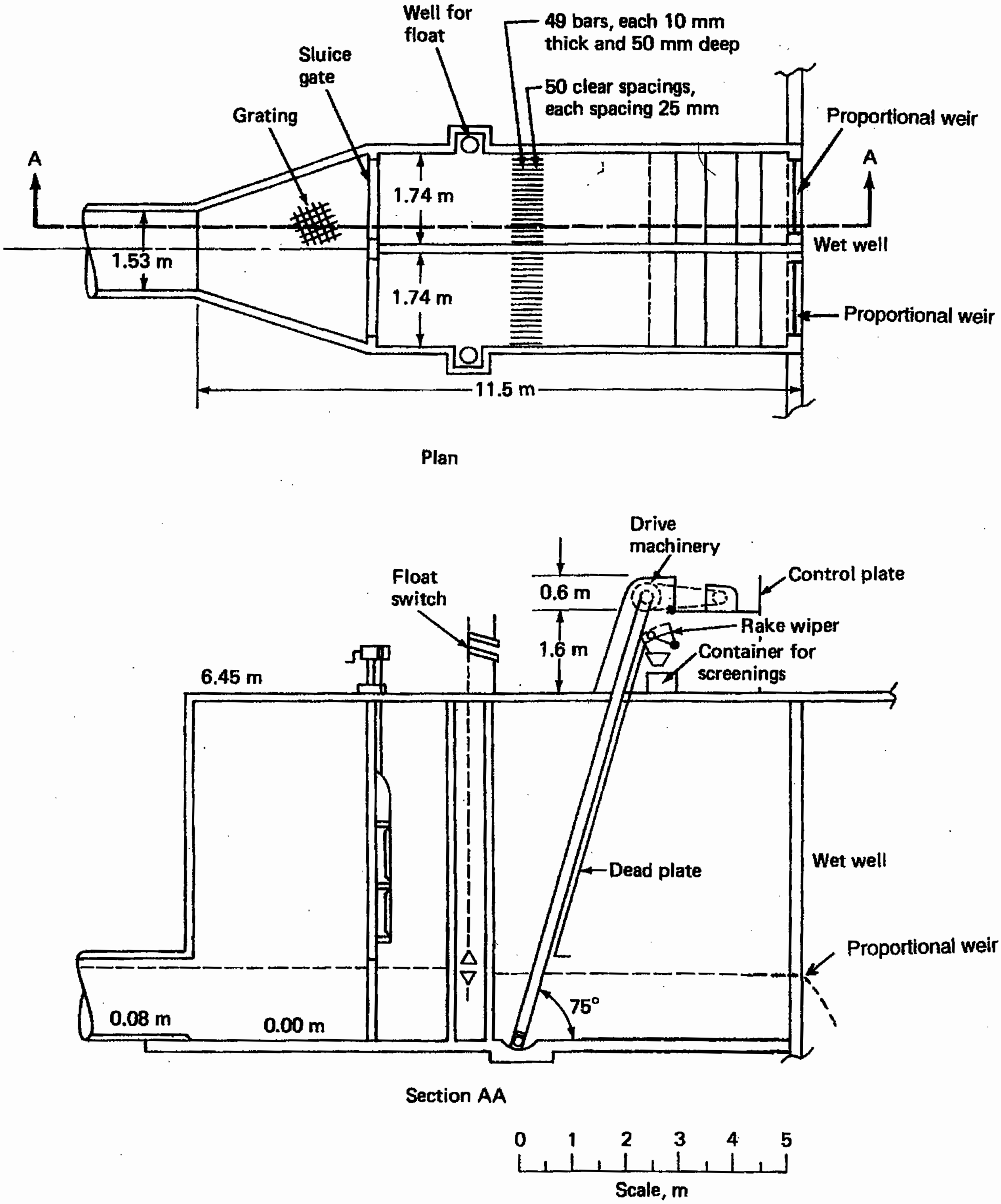

Figure 8-7 Design Details of Bar Rack. All Elevations Are with Respect to the Datum at the Floor of the Rack Chamber.

8-7 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT MECHANICALLY CLEANED BAR RACK

Debris in wastewater, if not removed properly, will damage equipment and interfere with normal operation of treatment processes. Often, high repair records of many processes and equipment may be attributed to poor performance of the bar rack. Common operation and maintenance problems that may develop at a bar rack and the procedure to correct them are summarized below.

8-7-1 Common Operating Problems and Suggested Solutions

The following items may be considered as troubleshooting guides:[8,13,14]

1. Obnoxious odors, flies, and other insects around the bar rack indicate prolonged storage of screenings at the facility. Increase frequency of removal and disposal of screenings.
2. Excessive screen clogging is an indication of (a) an unusual amount of debris in the wastewater, (b) low velocity through the rack, or (c) the automatic clock-operated screen rakes do not remove the debris fast enough. Possible solutions include identifying the source of waste causing excessive discharge of debris and stopping it at the source; providing a coarser rack; and resetting the timer cycle or installing level controller override.
3. Excessive grit accumulation in the chamber is due to low velocity in the channel. The possible solutions to this problem are remove bottom irregularity, reslope the bottom, rake the channel, or flush regularly with a hose.
4. A jammed raking mechanism may render the mechanical rake inoperable, and the circuit breaker will not reset. Remove the obstruction immediately.
5. A broken chain or cable, or a broken limit switch will render the rake inoperable, but the motor will run. Inspect chain and switches, and replace them as necessary.
6. A defective remote control circuit or motor will render the rake inoperable without any visible problem. Check remote control circuit and motor and replace them as necessary.

8-7-2 Facility Maintenance

Screenings removed from municipal wastewater treatment plants are odorous and attract flies and insects. The screenings should be stored in covered containers and hauled daily for disposal. The area should be thoroughly hosed off daily with chemical solution (chlorine or hydrogen peroxide).

The bar screen raking mechanism (raking chain; sprocket, teeth, and other moving parts) should be inspected daily. All moving parts should be lubricated and adjusted as recommended by the manufacturer.

Each bar screen should be taken out of service for maintenance on a routine basis. The unit should be dewatered and components checked for painting; cable, chain, or teeth replacement; removal of obstructions; straightening of bent bars; and so forth.

8-8 SPECIFICATIONS[d]

Brief specifications of several components of bar racks are presented in this section. The purpose of these specifications is to describe many components that could not be fully covered in the Design Example. These specifications should only be used as a guide for understanding the equipment.

8-8-1 General

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[d]Adapted in part from Refs. 3–8, 13, and 14.

[e]Some designers prefer back-cleaning devices because they have less tendency to jam at the bottom.

9

Pumping Station

9-1 INTRODUCTION

Wastewater pumping stations are used to lift or elevate the liquid from a lower elevation to an adequate height at which it can flow by gravity or overcome hydrostatic head. There are many pumping applications at a wastewater treatment facility. These applications include pumping of (1) raw or treated wastewater, (2) grit, (3) grease and floating solids, (4) dilute or well-thickened raw sludge or digested sludge, (5) sludge or supernatant return, and (6) dispensing of chemical solutions. Pumps and lift stations are also used extensively in the collection system. Each of the various pumping applications is unique and requires specific design and pump selection considerations.

The subject matter on pumping, pump selection, and design is very broad. Many books and technical papers have been written on the subject that cover the theory and practice in detail.[1-8] In this chapter various types of pumps and their applications are briefly discussed. Because centrifugal pumps are most commonly used for raw wastewater pumping, discussions on centrifugal pumps and pump selection, design procedure for raw wastewater pumping station, operation and maintenance, and specifications of pumping equipment and pumphouse are presented in detail.

9-2 PUMP TYPES AND APPLICATIONS

According to the Hydraulic Institute, all pumps may be classified as kinetic energy pumps or positive displacement pumps.[1] Brief descriptions and application of many types of pumps in these two classes are given in Table 9-1. A typical pump application chart is illustrated in Figure 9-1. Basic configurations of many types of pumps are also shown in Figure 9-2. Detailed discussions on the subject may be obtained in Refs. 1–9.

9-3 TYPES OF PUMPING STATIONS

The pumping stations are classified as wet well and dry well. The wet well stations employ either suspended or submersible pumps. Suspended pumps have the motor mounted above the liquid level in the wet well while the pump remains submerged. Submersible pumps have integral motors with special seals suitable for operation below liquid level. The suspended and submersible pumping arrangements in wet well are illustrated in Figures 9-2(a) and (b).

TABLE 9-1 Pump Types and Major Applications in Wastewater

Major Classifications	Pump Type	Brief Description	Major Pumping Applications
Kinetic	Centrifugal	Consists of an impeller enclosed in a casing with inlet and discharge connections. The head is developed principally by centrifugal force.	Raw wastewater, secondary sludge return and wasting, settled primary and thickened sludge, effluent
	Peripheral (torque-flow or vortex)	Consists of a recessed impeller on the side of the casing entirely out of the flow stream. A pumping vortex is set up by viscous drag.	Scum, grit, sludge and raw wastewater
Positive displacement	Rotary	Consists of a fixed casing containing gears, vanes, pistons, cams, screws, etc., operating with minimum clearance. The rotating element pushes the liquid around the closed casing into the discharge pipe.	Lubricating oils, gas engines, chemical solutions, small flows of water and wastewater
	Screw	Uses a spiral screw operating in an inclined case	Grit, settled primary and secondary sludges, thickened sludge, raw wastewater
	Diaphragm	Uses flexible diaphragm or disk fastened over edges of a cylinder	Chemical solutions
	Plunger	Uses a piston or plunger that operates in a cylinder. The pump discharges a definite quantity of liquid during piston or plunger movement through each stroke.	Scum, and primary, secondary, and settled sludges. Chemical solutions
	Airlift	Air is bubbled into a vertical tube partly submerged in water. The air bubbles reduce the unit weight of the fluid in the tube. The higher unit weight fluid displaces the low unit weight fluid, forcing it up into the tube.	Secondary sludge circulation and wasting, grit
	Pneumatic ejector	Air is forced into the receiving chamber, which ejects the wastewater from the receiving chamber.	Raw wastewater at small installation (100 to 600 L/min)

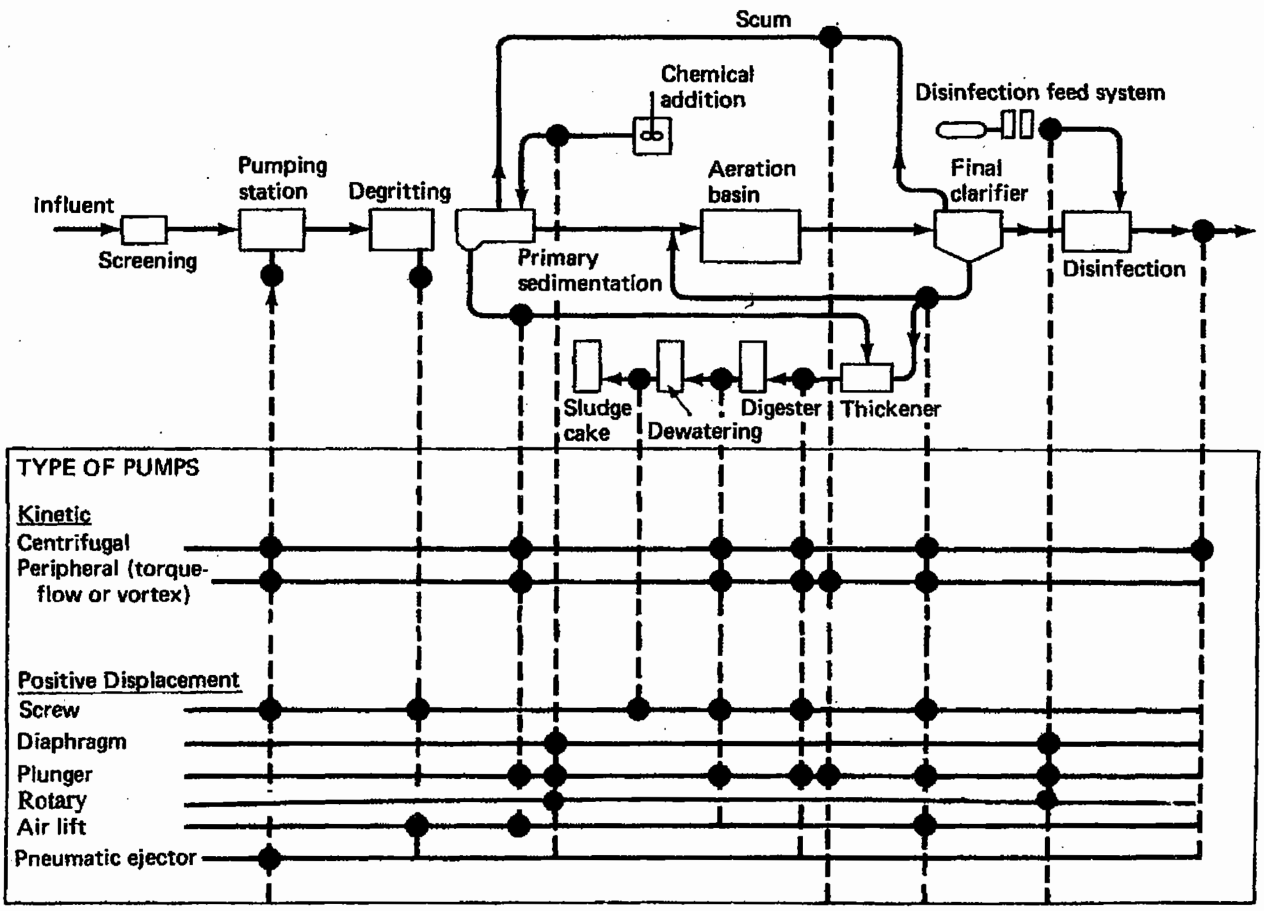


Figure 9-1 Typical Wastewater Pumping Applications.

The dry well stations employ either dry well or self-priming centrifugal pumps. The dry well centrifugal pumps operate within a dry well adjacent to the wet well. The volute of the pump is positioned below the low water level in the wet well to ensure a positive suction or prime as shown in Figures 9-2(c). In the other case a recessed dry section may be located within a wet well as shown in Figure 9-2(d). The pumps are specially designed to provide a suction lift and are self-priming. Other types of pumping stations utilize screw, air lift pumps, or pneumatic ejectors. These types of pumping systems are also illustrated in Figures 9-2(e)–(g).

9-4 HYDRAULIC TERMS AND DEFINITIONS COMMONLY USED IN PUMPING

Common terms used in pumping and pump analysis include (1) head, (2) capacity discharge or flow rate, and (3) work power and efficiency. These terms are briefly discussed in this section.

9-4-1 Head

Head describes hydraulic energy (kinetic or potential) equivalent to a column of liquid of specified height above a datum. Head and pressure may be expressed in terms of the

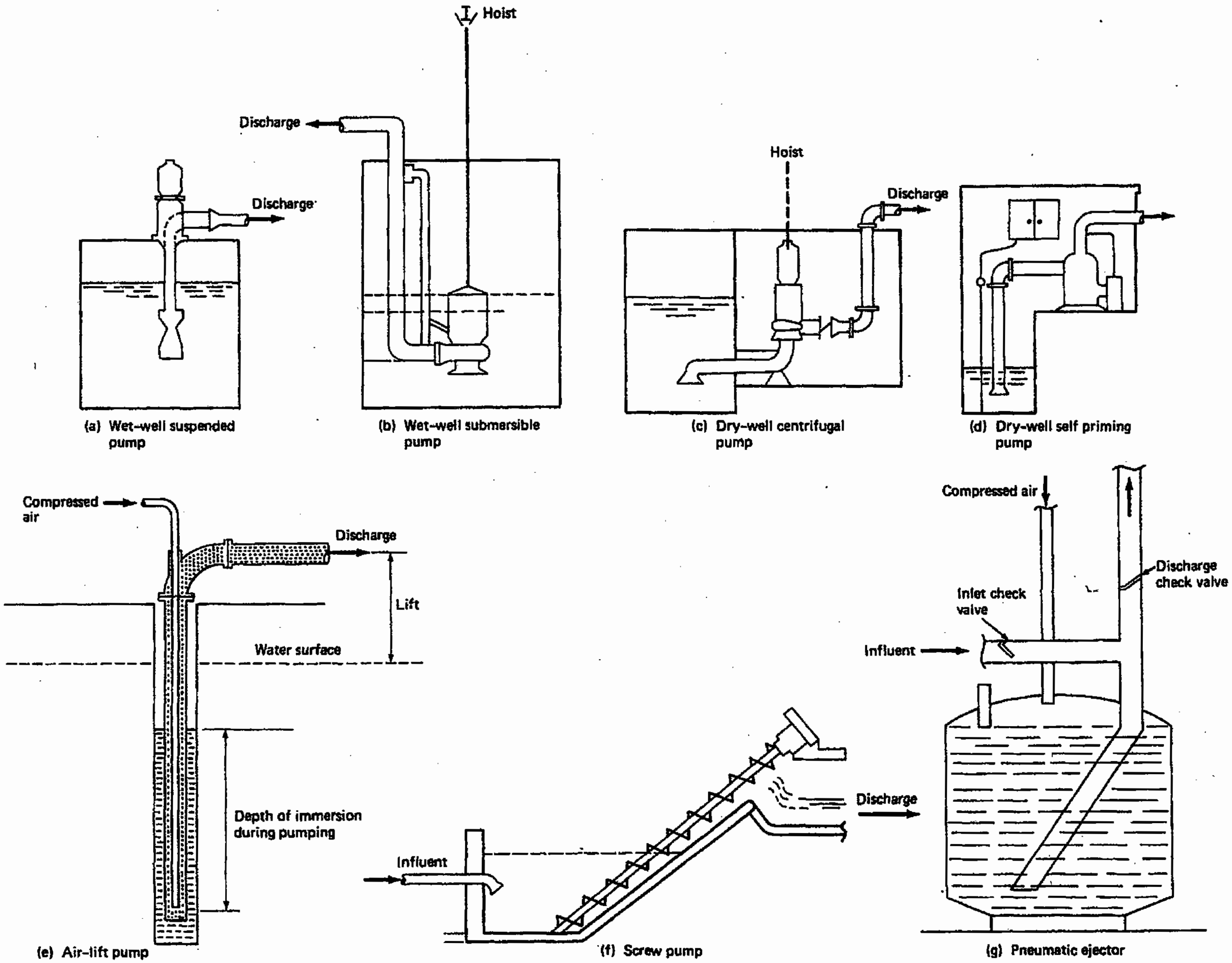


Figure 9-2 Types of Pumps and Pumping Stations.

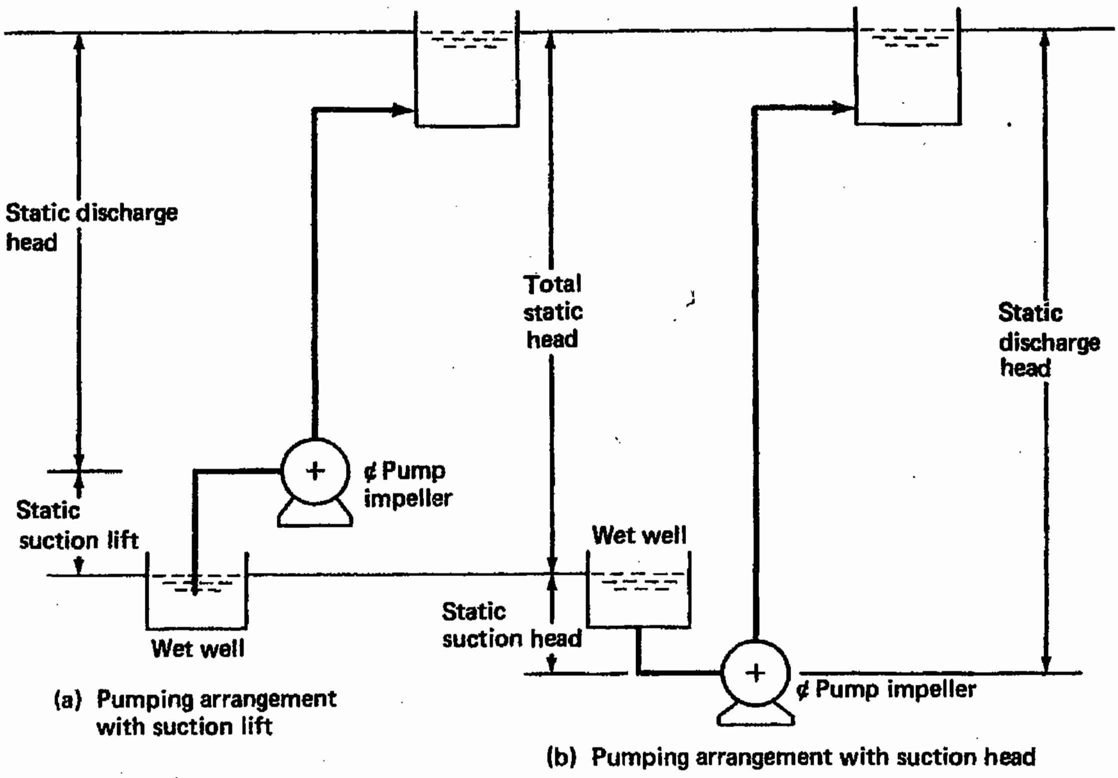


Figure 9-3 Head Terms Used in Pumping.

other (1 m of water = 9.81 kPa; or 1 ft of water = 0.433 psi). Many head terms in pumping include static suction lift (or static suction head), static discharge head, and total static head. These terms are illustrated in Figure 9-3.

The total dynamic head (TDH) of a pump is the sum of total static head, the friction head, and the minor losses. The friction head consists of loss of head in piping (suction and discharge). It is calculated from Darcy-Weisbach or Hazen-Williams equations. The minor losses are produced because of fittings, valves, bends, entrance, exit, etc., and are normally calculated as a function of velocity head. Some of these terms are defined in Eqs. (9-1)–(9-6). The kinetic energy or velocity head is expressed by Eq. (9-5) ($K = 1.0$) and is often added with the minor loss calculations.

$$\text{TDH} = H_{\text{stat}} + h_f + h_m \tag{9-1}$$

$$h_f = \frac{fLV^2}{2gD} \text{ (Darcy-Weisbach)} \tag{9-2}$$

$$h_f = 6.82\left(\frac{V}{C}\right)^{1.85} \times \frac{L}{D^{1.167}} \text{ (Hazen-Williams, SI units)} \tag{9-3}$$

$$h_m = K\frac{V^2}{2g} + h_v \tag{9-4}$$

$$h_v = \frac{V^2}{2g} \tag{9-5}$$

$$h_L = h_f + h_m \tag{9-6}$$

The Hazen-Williams equation is most commonly used for force mains. This equation in various forms is given below:

$$V = 0.355\, C\, D^{0.63} \left(\frac{h_f}{L}\right)^{0.54} \quad \text{(SI units)} \tag{9-7}$$

$$V = 0.550\, C\, D^{0.63} \left(\frac{h_f}{L}\right)^{0.54} \quad \text{(U.S. customary units)} \tag{9-8}$$

$$Q = 0.278\, C\, D^{2.63} \left(\frac{h_f}{L}\right)^{0.54} \quad \text{(SI units)} \tag{9-9}$$

$$Q = 0.432\, C\, D^{2.63} \left(\frac{h_f}{L}\right)^{0.54} \quad \text{(U.S. customary units)} \tag{9-10}$$

where

TDH = total dynamic head (or total head, TH), m (ft)
H_{stat} = total static head, m (ft) (see Figure 9-3)
h_f = total friction head loss in suction and discharge pipes, m (ft)
h_L = total head loss, m (ft)
h_m = minor losses, m (ft)
h_v = velocity head, m (ft)
V = velocity in the pipe, m/s (ft/s)
Q = flow rate, m^3/s (ft^3/s)
f = coefficient of friction (The value of f depends on the Reynolds number and the relative roughness and diameter of the pipe. It may range from 0.01 to 0.10.)
C = coefficient of roughness [The value of C depends on the pipe material and age of the pipe and characteristics of liquid being pumped. It may range from 60 to 140 (see Secs. 9-5-12 and 16-4 for sludge pumping).]
D = equivalent diameter of the pipe, m (ft)
g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)
K = head loss coefficient (see Appendix B)
L = length of pipe, m (ft)

9-4-2 Capacity (Discharge or Flow Rate)

The capacity, discharge, or flow rate of a pump is the volume of liquid pumped per unit of time. It is expressed as cubic meters per second (m^3/s), liters per second (L/s), gallons per minute (gpm), cubic feet per second (ft^3/s), etc.

9-4-3 Work Power and Efficiency

The work done by a pump is proportional to the product of the specific weight of the fluid being discharged and the total head against which the flow is moved. The pump efficiency is the ratio of the useful pump output power to the input power. Pump output power and efficiency are expressed by Eqs. (9-11)–(9-13):

$$P_w = K'Q(\text{TDH})\gamma \tag{9-11}$$

$$E_p = \frac{P_w}{P_p} \tag{9-12}$$

$$E_e = \frac{P_p}{P_m} \tag{9-13}$$

where

P_w = power output of the pump (water power), kW [hp (horse power)]
P_p = power input to the pump (brake power), kW (hp)
P_m = power input to the motor (electrical energy or wire power), kW (hp)
Q = capacity, discharge, or flow rate, m^3/s (ft^3/s)
TDH = total dynamic head, m (ft)
γ = specific weight of the liquid pumped, kN/m^3 (lb/ft^3)
E_p = pump efficiency, usually 70–90 percent
E_e = motor efficiency, usually 90–98 percent
K' = constant depending on the units of expression
(TDH = m, $Q = m^3/s$, $\gamma = 9.81\ kN/m^3$, P_w = kW, $K' = 1\ kW/kN \cdot m/s$)
(TDH = ft, $Q = ft^3/s$, $\gamma = 62.4\ lb/ft^3$, P_w = hp, $K' = 1/550\ hp/ft \cdot lb/s$)

9-5 CENTRIFUGAL PUMPS

Centrifugal pumps are most commonly used for water and wastewater pumping. Because of their extensive use in water supply and wastewater treatment, the remainder of the chapter is devoted to the principles of centrifugal pumps, their design, specifications, and pumping station operation.

9-5-1 Action of a Centrifugal Pump

In centrifugal pumps the head is developed principally by centrifugal force. The inlet to the pump is axial and the outlet is tangential. The flow is accelerated by the rotating impeller, which imparts to it both radial and tangential velocity; the ratio of the two depends on the design of the impeller. The increase in cross section of the volute (casing) produces the change from velocity head to pressure head.

9-5-2 Effect of Varying Speed

The pressure and discharge of a pump varies with pump speed. The specific speed of a pump is defined as the speed at which a pump will discharge a unit flow under a unit head at maximum efficiency. It is expressed by Eq. (9-14)

$$N_s = \frac{NQ^{1/2}}{H^{3/4}} \tag{9-14}$$

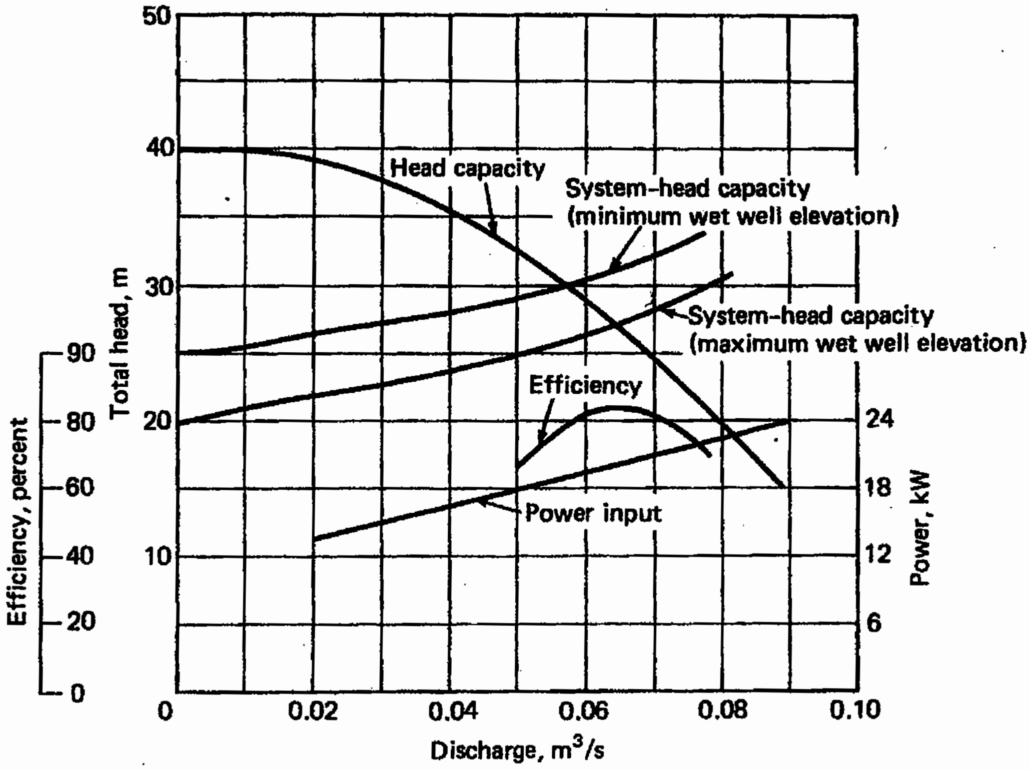


Figure 9-4 Characteristic Curves of a Centrifugal Pump and System Head Capacity Curves.

where

N_s = pump specific speed[a]
N = pump rotative speed, rev/min (rpm)
Q = flow at optimum efficiency, m^3/s (gpm)
H = total head, m (ft)

The specific speed is an index number descriptive of suction characteristics of a given impeller. As a general rule, pumps with high specific speeds are more efficient than those with low. Also, high specific speed is associated with high discharge at low head. Use of specific speed in pump selection is given in Sec. 9-5-10.

9-5-3 Pump Characteristic Curves

The discharge of a centrifugal pump is a function of not only the specific speed but also of the pressure conditions under which the pump operates. The pump manufacturers conduct tests where pump capacity is changed by throttling a valve in the discharge piping and measuring the flow and pressures at the suction and discharge lines. The power inputs are also measured. The results of the tests are represented by a series of curves representing head, power, and efficiency versus flow at constant speed. These curves are shown in Figure 9-4. The shape of head capacity curves is important in selecting pumps

[a] N_s in U.S. customary units = 51.6 × N_s in metric units.

for specific applications. These shapes of head capacity curve may be rising head, flat, steep, or drooping head. Such curves are a function of specific speed and pump design.

Manufacturers frequently present pump data in the form shown in Figure 9-5. A pump casing can accommodate impellers of several sizes; therefore, a particular pump casing may be used for different head and flow applications. This feature is helpful because the existing pump can be expanded by replacement of the impeller and motor. Use of the pump curves in pump selection is discussed in the Design Example. For such modifications, if all other variables are kept equal, Q, TDH, and P can be varied with the choice of pump speed and impeller diameter in accordance with the following relations:

Pump speed N variable

1. Q varies as N
2. TDH varies as N^2
3. P_w varies as N^3

Pump impeller diameter D variable

1. Q varies as D^3
2. TDH varies as D^4
3. P_w varies as D^5

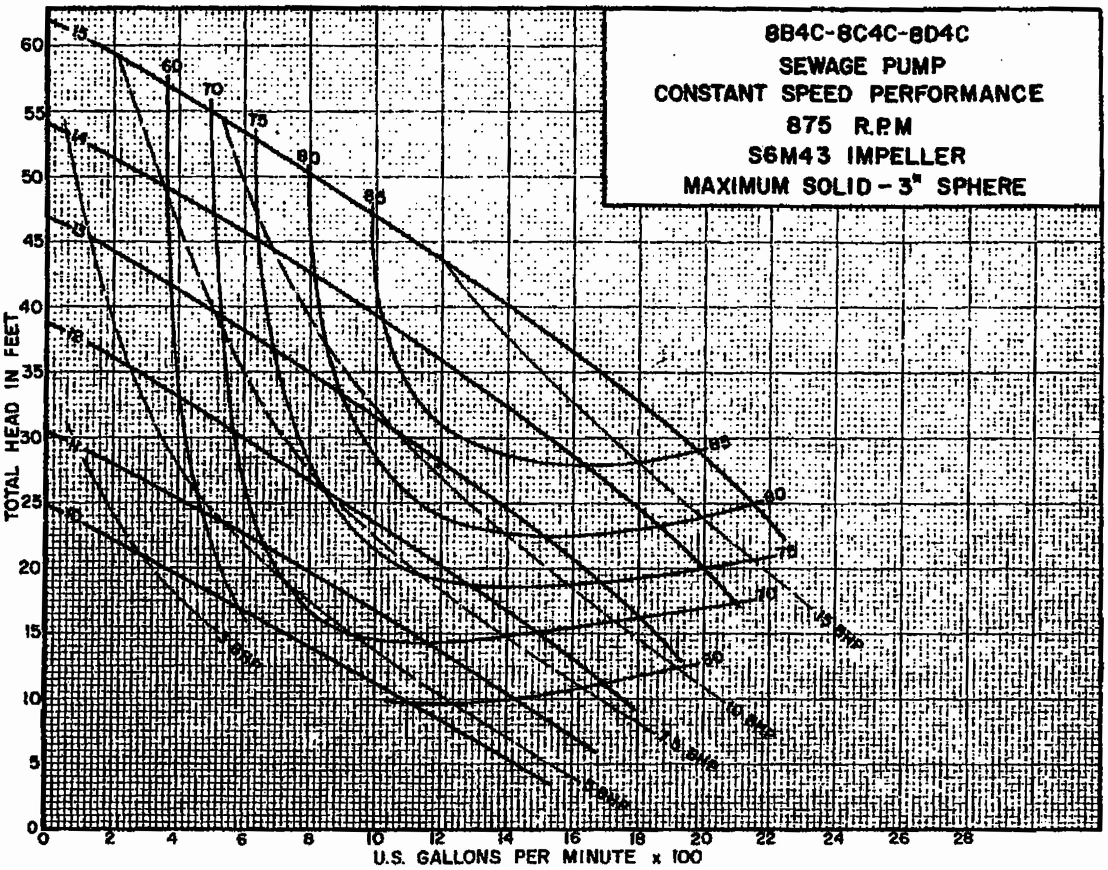
9-5-4 System Head Capacity

A system head capacity curve is prepared by plotting total dynamic head (TDH) at various flow conditions. This curve is prepared by the designer for suction and discharge piping arrangements. The intersection point of pump head capacity and system head capacity curves is important because this point represents the operating head and capacity of the pump for the selected piping system. For good pump selection this point (pump-operating point) should be as close as possible to the peak efficiency. If the wet well levels fluctuate considerably, it is usual to prepare two system curves using the minimum and maximum static heads based on wet well water surface elevations. The system head capacity curve is shown in Figure 9-4.

9-5-5 Net Positive Suction Head

For a centrifugal pump to operate, the liquid must enter the center of the impeller under pressure, usually atmospheric pressure. This pressure is referred to as net positive suction head (NPSH). There are two values of NPSH: the available and the required. The available NPSH depends on the location and design of the intake system and is calculated by the engineer. It is the minimum suction head required at the inlet of the impeller to prevent boiling of the liquid under the reduced pressure condition created at the impeller and smooth operation of the impeller without cavitation. Eq. (9-15) is used to calculate the available NPSH. The required NPSH is determined by the pump manufacturer

(a)



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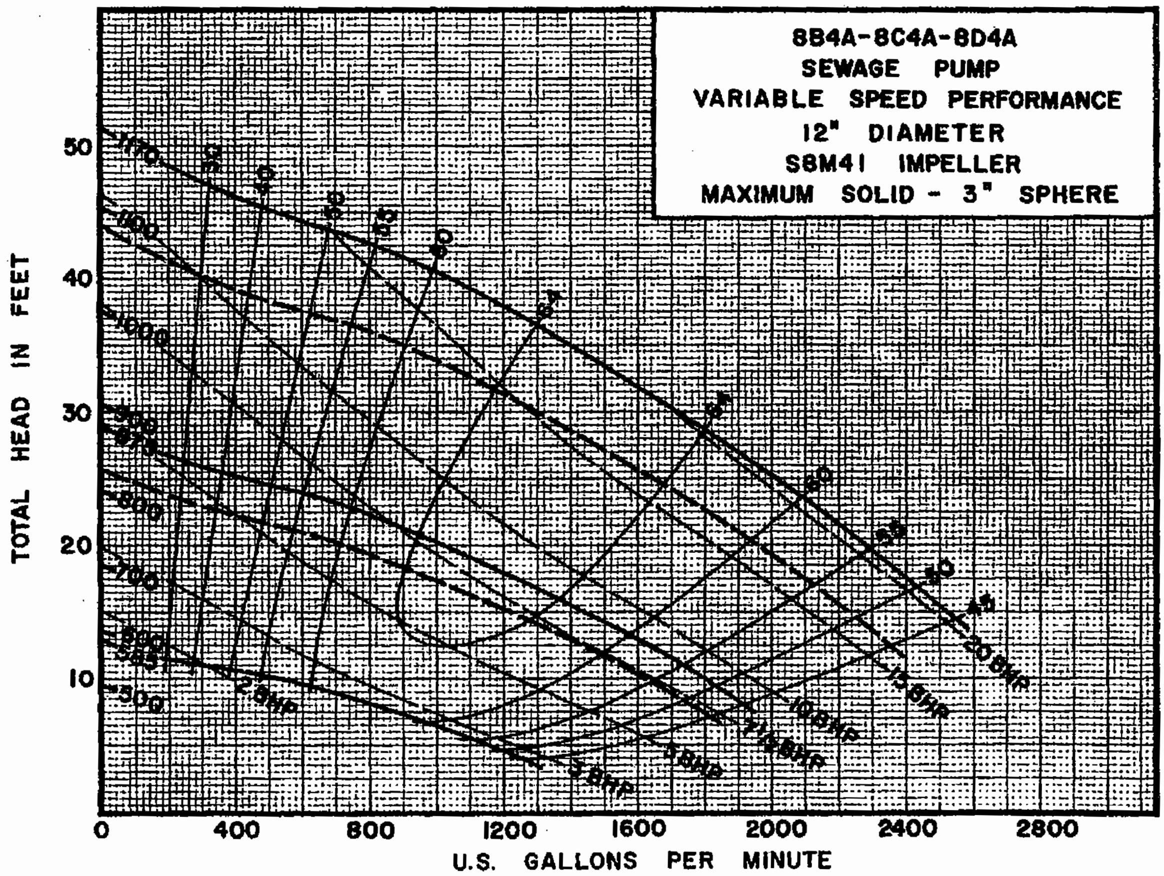


Figure 9-5 Manufacturer's Characteristic Curves of Centrifuge Pumps (courtesy Lakeside Equipment Corp.): (a) variable impeller size 25.4–38.1 cm (10–15 in.) and 875 rpm and (b) variable speed 585–1170 rpm and impeller diameter 30.5 cm (12 in.).

based on extensive testing programs. The available NPSH should exceed the required NPSH with a margin of safety (1 m or more).[5]

$$\mathrm{NPSH_{av}} = H_{\mathrm{abso}} + H_s - h_L - H_{vp} \tag{9-15}$$

where

$\mathrm{NPSH_{av}}$ = available net positive suction head, m
H_{abso} = absolute pressure on the surface of the liquid in the suction well, m (ft) This is the barometric pressure at the given altitudes (see Appendix A).
H_s = the static head of the liquid above the center line of the pump, m (ft). If the liquid level is below the centerline, H_s is negative.
h_L = total head loss due to friction, entrance, valves, fittings and specials in the suction piping, m (ft)
H_{vp} = absolute vapor pressure of the liquid at the pump temperature, m (H_{vp} at various temperatures are given in Appendix A)

The calculation procedure for available NPSH is illustrated in the Design Example.

9-5-6 Cavitation

Cavitation is the phenomenon of cavity formation or collapse of cavities. Cavities develop when the absolute pressure in a liquid reaches the vapor pressure related to the liquid temperature. This may happen under the following design and operating conditions:[5]

1. When impeller under high speed (revolutions per minute, rpm) travels faster than the liquid can enter or move
2. When suction is restricted
3. When required NPSH is equal to or greater than available NPSH
4. When specific speed is too high for optimum design parameters
5. When the temperature of the liquid is too high for suction conditions
6. When the pump operates at extreme capacities below or above the best efficiency point (BEP)

Cavitation causes reduction in flow, and under serious conditions, the pump may lose its prime and cause pitting of the impeller surface, a rattling or pinging noise, and eventual breakdown of the pumping equipment.

9-5-7 Pump Combinations

Pumps may be connected in series or in parallel. In parallel operation, for a given head the total discharge is added up for all the pumps. In series operation, the total head for all pumps is added up for a given discharge. Parallel and series combinations are shown in Figures 9-6(a) and (b).

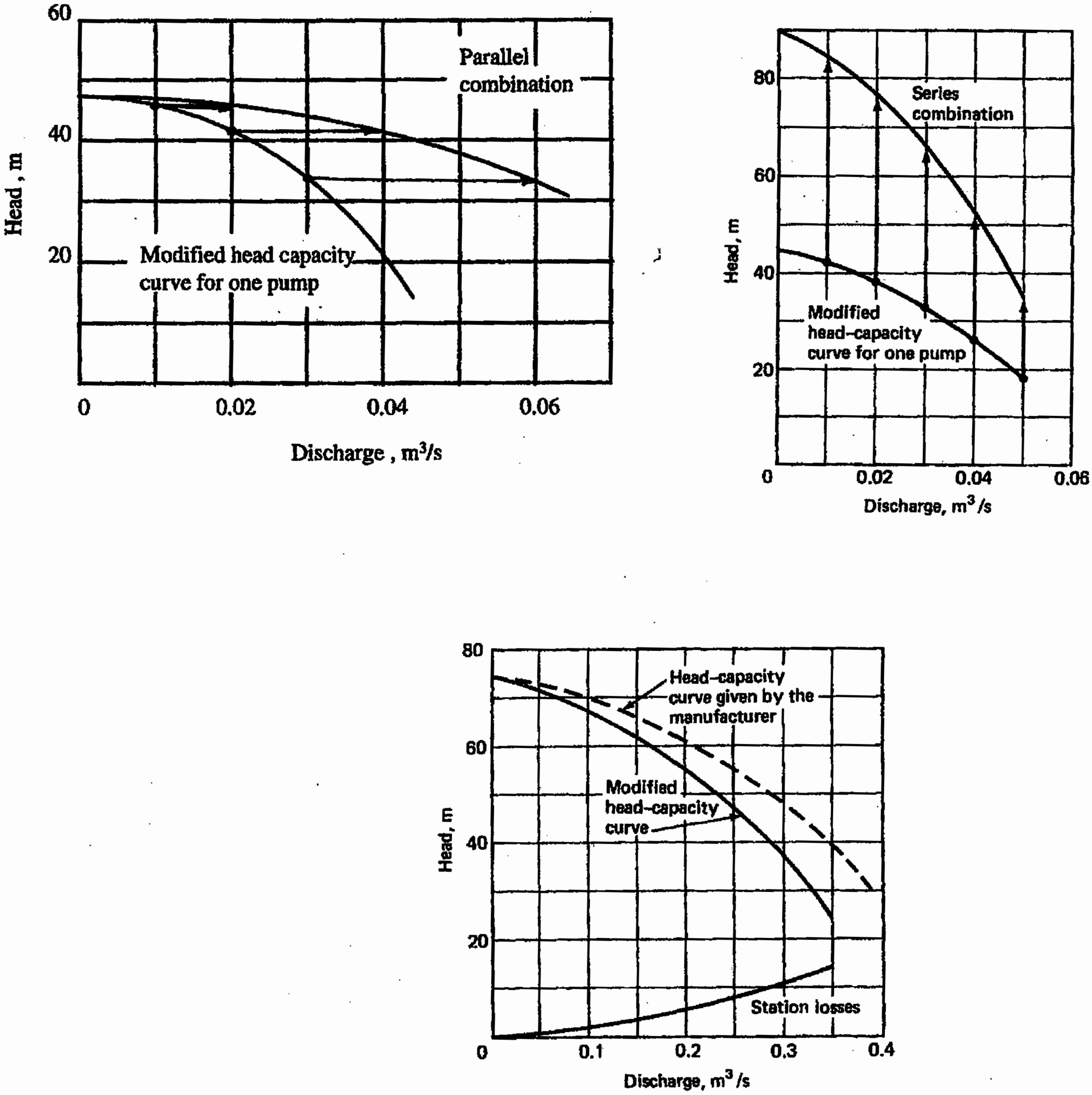


Figure 9-6 Pump Combinations in Parallel and Series and Modified Pump Curve.

9-5-8 Modified Pump Head Capacity Curve

When two or more pumps are arranged in parallel, these pumps will discharge into a common header or force main. The procedure for determining the pump-operating point is based on development of modified pump head capacity curve. This curve is prepared by subtracting from the pump head capacity curve the head losses in the suction and discharge piping of each pump. The combined head capacity curve is prepared using the modified curves of each pump. The point of intersection of the combined curve with the system head capacity curve gives the total capacity of the pump combination and modified head for each pump. Note that only the losses in that part of the system common to all pumps are included in the system head capacity curve. The modified head capacity curve is shown in Figure 9-6(c). Procedure for preparation of modified curve for pump combination is presented in the Design Example (see Sec. 9-9-2, Step B, 1). Detailed discussion may be found in Ref. 2.

9-5-9 Pump Classification

Centrifugal pumps are classified in many ways. These classifications or types are generally based on the manner in which the liquid enters and leaves the casing and the manner in which the impeller or vanes impart the energy to the water to transform the velocity into the head. Based on the impeller configuration and specific speed, pumps are traditionally divided into three major classes: radial flow, mixed flow, and axial flow. There are, however, many other variations as presented in Table 9-2. Basic characteristics of the radial flow, mixed flow, and axial flow pumps are given below. Major differences in pump classes are presented in Table 9-3.

Radial Flow Pumps

1. The pressure head is developed basically by the action of centrifugal force.
2. The liquid normally enters the impeller at the hub and flows radially to the periphery.
3. The impellers used may be single- or double-suction.
4. The impeller may have vanes that may be straight or double curvature, such as Francis-Vane.
5. Pump shaft may be horizontal or vertical.
6. The pumps have specific speeds below 80 (4200 in U.S. customary units) with single impeller and 115 (6000 in U.S. customary units) with double-suction impeller.
7. For raw wastewater pumping, the nonclog type is used (Sec. 9-5-11).

Mixed Flow Pumps

1. Head is developed partly by centrifugal force and partly by the lift of the vanes on the liquid.
2. The liquid enters axially and discharges in an axial and radial direction.
3. The casing may be either the volute or the diffusion type. This gives two variations of mixed flow pumps, namely, mixed flow volute and mixed flow propeller pumps.
4. The pump may have single- or double-suction and may be single- or multistage.
5. The specific speed of the pump is in the range of 80–120 (4200–6250 U.S. customary units).
6. As specific speed increases, mixed flow pumps become more like axial flow pumps.
7. The mixed flow pumps are applicable for medium-head application (7–16 m) and for medium to large capacities.
8. The pumps require positive submergence, although with proper selection of rotative speeds they may be used for limited suction lift applications.
9. Common application is for pumping of raw wastewater and storm water flow. For raw wastewater pumping, a nonclog-type pump must be used (Sec. 9-5-11).

Axial Flow Pump

1. The pumps are also called propeller pumps. They develop most of the head by a propelling or lifting action of the vanes on the liquid.

TABLE 9-2 Classification of Centrifugal Pumps

Types or Variations	Description
Suction	
Single-suction	A pump equipped with one or more single-suction impellers
Double-suction	A pump equipped with one or more double-suction impellers
Number or stages	
Single-stage	A pump in which total head is developed by one impeller
Multistage	A pump having two or more impellers acting in series in one casing
Position of the shaft	
Horizontal	A pump with the shaft normally in horizontal position
Vertical pump	
Dry-pit type	A pump with vertical shaft located in a dry well
Submerged type	A pump with vertical shaft located in a wet well
Casing	
Volute	A pump having a casing made in the form of a spiral or volute. The velocity is transformed into head by gradually increasing area of the water passage. Dry-well pumps generally have volute case.
Circular	A pump having a casing of constant cross section, concentric with the impeller. Curved vanes are used to transform velocity into head. Wet-well pumps have circular casing.
Diffusion (turbine)	A pump that transforms velocity into head by diffusion vanes. Diffusion casing is used with wet well pumps.
Impeller	
Enclosed (front and back shroud)	A pump that has enclosed impeller with shroud; commonly used for pumping sanitary sewage
Open or semi-enclosed	A pump that is constructed with the vanes attached to an upper or inbound shroud. Lower shroud is not used. The pump is best suited for pumping large volumes with intermittent service (storm water).

2. They have a single inlet impeller with the flow entering axially and discharging axially.
3. Specific speed is usually above 200 (10,000 in U.S. customary units).
4. These pumps are customarily used for large flows (over 600 L/s) and low head (below 10 m) installations. Best applications include irrigation work and storm water pumping. They should not be used for pumping raw wastewater, sludge, or unscreened storm water.

TABLE 9-3 Major Differences in Three Classes of Centrifugal Pumps.

Category	Radial Flow	Mixed Flow	Axial Flow
Specific speed	Less than 80 for single-suction (4200 U.S. customary units) and 115 for double-suction (6000 U.S. customary units)	80–120 (4200–6250 U.S. customary units)	More than 200 (10,000 customary)
Discharge direction	Radial to the axis of rotation	Partially radial and partially parallel to the axis of rotation	Parallel to the axis of rotation
Pumping action	Centrifugal force and lifting action	Centrifugal	Lifting action
Head ranges	High to medium (more than 16 m)	Medium to low (7–16 m)	Low (less than 10 m)
Capacity	Low to medium	Medium to high	High

9-5-10 Comparison of Specific Speed and Pump Class

Pump comparison based on impeller configuration and specific speed is best illustrated in Figure 9-7 and by use of an example. Suppose a designer needs to select a pump type for pumping of raw wastewater for a total head of 30 m and the maximum anticipated capacity of 0.25 m^3/s at an operating drive of 1200 rpm. The specific speed is calculated from Eq. (9-14):

$$N_s = \frac{1200 \times (0.25)^{1/2}}{(30)^{3/4}} = 46.81$$

Referring to Figure 9-7, this application falls within the radial flow, Francis-Vane area, with an anticipated pump efficiency of 88 percent.

9-5-11 Nonclog Pump

The term *nonclog pump* has been used for radial flow and mixed flow pumps. Nonclog pumps differ from conventional units in arrangement, size, smoothness, and contour of channels and impellers to permit passage of clogging material. These pumps have open passages (often slightly less than that of the discharge pipe) and a minimum number of vanes (two in smaller and not more than four in large-size pumps). The peripheral (torque flow or vortex) pumps also offer unique nonclog features as the blades are recessed entirely into the curved shroud; thus, the flow does not pass through the impeller. Experience has shown that, in pumping of raw wastewater, such materials as rope, string, rags, sticks, cans, rubber goods, and grease present the greatest problem. For pumps

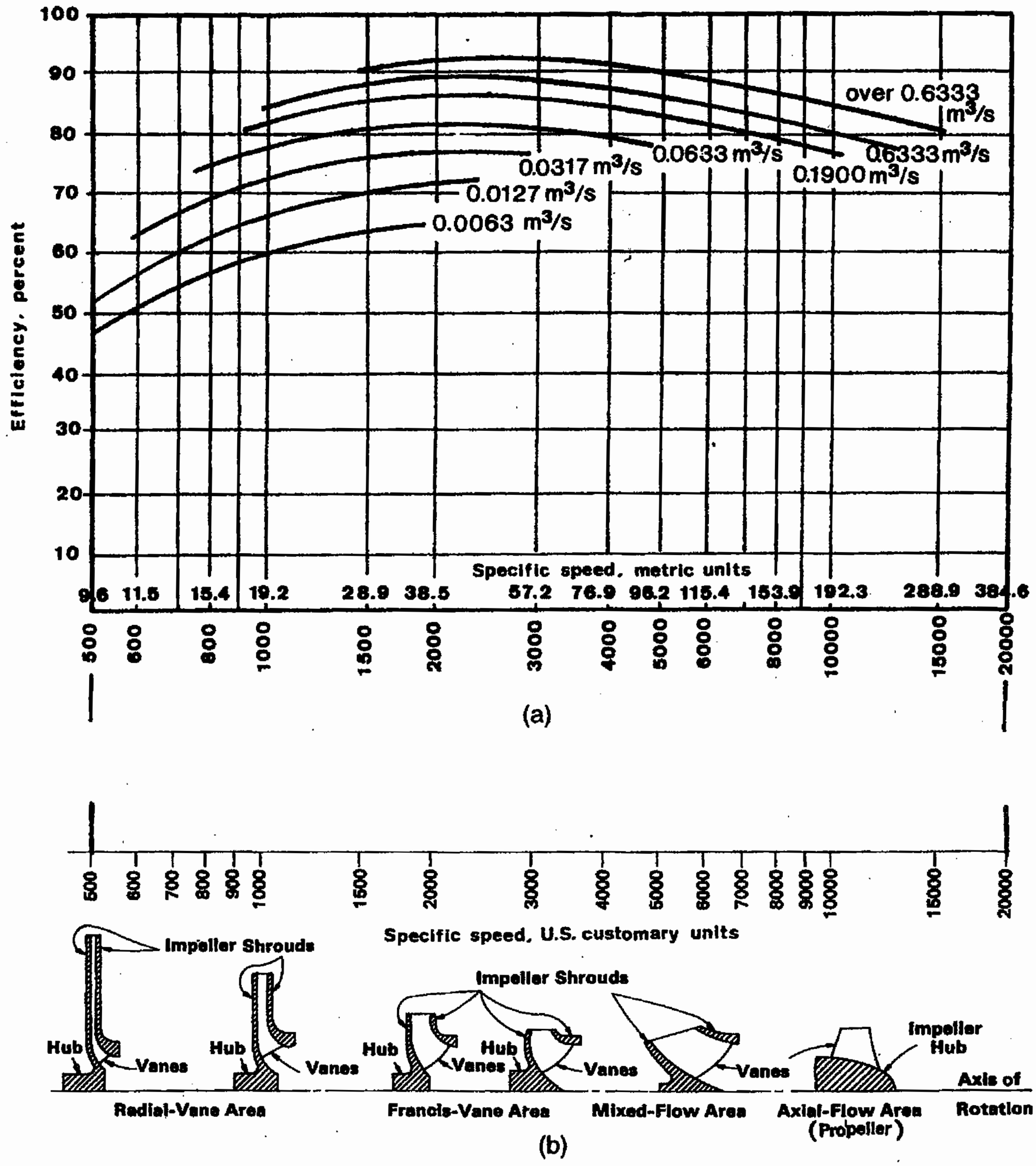


Figure 9-7 Comparison of Pump Capacity, Efficiency, and Impeller Designs with Specific Speed: (a) pump efficiency versus specific speed (courtesy Worthington Pump Corporation) and (b) profile of several pump impeller designs placed according to where each design fits on the specific-speed scale (courtesy of the Hydraulic Institute).

smaller than 25 cm in size that handle sanitary sewage, nonclog units are used exclusively. Pumps smaller than 10 cm in size are not recommended for such applications.

9-5-12 Sludge Pumping

Pumps commonly used for sludge transport are nonclog centrifugal, peripheral (torque-flow or vortex), screw, and plunger pumps. The head loss in the pipelines transporting sludge is generally calculated from the Hazen-Williams equation [Eq. (9-3)]. The Hazen-Williams coefficient C is decreased because sludge is more difficult to pump than wastewater. The suggested apparent values of Hazen-Williams coefficient for different types of sludges are summarized below (see Sec. 16-4 for sludge pumping):

1. Waste activated sludge (total solids up to 1 percent), $C = 90$
2. Primary sludge (total solids up to 4.5 percent), $C = 55$
3. Thickened or digested sludge (total solids up to 8 percent), $C = 35$

It is a good design practice to provide sludge piping larger than 15 cm (6 in.) in diameter. Smaller piping may be used for well-homogenized sludge or if the piping is glass lined. For small sludge flows, the pumps are operated on a time cycle to achieve larger flows and velocities during pumping cycle. A liberal number of cleanouts and hose gates should be provided in the piping for cleaning the stoppages.

9-5-13 Constant- and Variable-Speed Pumps

Constant-speed pumps are generally used when continual flow is not essential. These pumps generally operate on level controls. These pumps are simplest, most reliable, and low in cost. However, if a continuous flow is desired from the pumping station, a variable-speed drive may be required to adjust the pumping speed to the influent flow rate. The control system varies the head capacity curve of the pump in relation to the station head curve. Therefore, while the flow is continuous from the station, the flow rate will vary with the water surface elevation of the wet well.

Pumping stations having large variations in the flow rate would require more pumping units under the constant-speed system than those under the variable-speed system. On the other hand, the constant-drive units generally have better efficiency than the variable-speed units because the variable-speed units reach the best efficiencies when operating at maximum speed.

9-5-14 Pump Drives

Pump drives convert electrical or thermal energy into the rotating kinetic energy required to drive a pump. They include electric motors (constant- and variable-speed drives), internal combustion engines, and turbines. The selection of a pump drive must be based on (1) evaluation of available types and reliability, (2) cost of the energy sources, (3) cost of the drive, characteristics of the load demand, and power output of the drive. Centrifugal pumps generally have a relatively low starting torque requirement and smooth running torque. Reciprocating pumps often have a higher starting torque and a more cyclic or pulsating running torque.

Electric Motors. Electric motors are the most common pump drive in use today.[10] Their common use as pump drives are because of their high efficiency in energy conversion, their simplicity and reliability, and the widespread availability of reliable electric power. The typical electrical motor efficiencies are summarized in Table 9-4. Electric motors used as pump drive are primarily alternating current (a-c) motors; however, some

TABLE 9-4 Typical Electric Motor Efficiencies.

Motor Power Rating (kW)	Typical Efficiency (percent)
1–5	70–80
5–7.5	80–85
7.5–20	85–88
20 and above	88–92

direct current (d-c) motors are also used. Two general types of a-c motors in common use are induction and synchronous. Electric motors are also constant-speed and variable-speed.

Induction Motors. The most common electric a-c motor is the induction motor. This type of motor is commonly used for lower power applications [less than 750 kW (1000 hp)] or higher speed applications (more than 600 rpm). The primary disadvantage of induction motors is their *power factor.* In simple terms, the voltage applied to the motor leads the current, resulting in a power factor of less than 1.0. Since some electric utilities assess additional charges for low power factor, it may be desirable to increase the power factor for large motors.

Synchronous Motors. One way to increase the power factor is to use synchronous motors. These motors can be operated over a range of power factors from less than 1.0 to greater than 1.0. In simpler terms, synchronous motors can be operated with either leading or lagging current. Therefore, these motors tend to correct the deficiency of induction motors by increasing the power factor.[11] Synchronous motors are more complex than induction motors and typically are cost-effective for large-power applications (above 750 kW) or low-speed applications (less than 500 rpm).

Constant- and Variable-Speed Motors. Electric motors are either constant-speed or variable-speed. Constant-speed motors may be either the squirrel cage, synchronous, or wound rotor induction type. For constant-speed applications, electric motors are best suited. If variable-speed operation is desired, an auxiliary variable-speed drive unit is required. The two most common variable-speed units for pumps driven with a-c electric motors are *eddy current* drives and *variable frequency* controllers. Eddy current drives are magnetic devices. An electromagnetic input rotor is rotated around an electromagnetic output rotor. The speed of the output is adjusted by increasing and decreasing the current applied to the magnets.

Variable-frequency controllers adjust the speed of an a-c motor by varying the frequency of the a-c current applied to the motor. Variable-speed electric drives are used with variable-speed pumps to match the pump output with the influent conditions. They have several benefits: (1) reduction in wet well size, and (2) the water hammer is controlled by slowing the pump gradually before shutdown. For wastewater pumping applications the motor speed is generally in the range of 500–1700 rpm. Detailed discussion on pump drive units may be found in Refs. 2–4 and 7.

Internal Combustion Engines. Internal combustion engines operate on natural gas, gasoline, or diesel fuel. The engines are similar to common automobile or truck engines but are designed for continuous high-power output service. Internal combustion engines are most often used where (1) wastewater gas is available for fuel, (2) electrical power may not be available, and (3) a backup system for electric power outages may be necessary.

Turbines. Turbine engines are either gas-driven or steam-driven. Both types are briefly described below.

Gas Turbine. Gas turbine engines are similar to jet engines used on aircraft. They operate at very high speeds and require gear reducers to drive pumps. They are very seldom used as pump drivers.

Steam Turbines. At the turn of the century, steam was a common power source for pumps, but steam-driven pumps require boilers and other auxiliary equipment to generate steam. Unless steam is available from other sources (such as waste-heat generated from solid waste incineration), steam turbines are usually not feasible alternatives for pump drivers.

9-6 DESIGN OF PUMP STATIONS

Pump stations are the structures that house pumps, piping, equipment, and chemicals. They may be large architecturally designed facilities or may be simple and small structures. In either case the design must ensure that the pump station meets the functional requirements of the pumping facility. In addition, the structure of the pump station houses and protects pumps and auxiliary facilities from weather, vandals, and other undesirable elements. The design of the pumping station involves (1) selection of site, (2) selection of pumping station, (3) selection of pumps and controls, and (4) design consideration of pump station. Each of these is briefly discussed below.

9-6-1 Selection of Site

Proper site selection of a pumping station is essential for good design and operation of the facility. Important site considerations include power, access roads, flood protection, and environmental considerations. These are briefly discussed below.

Power. Pump stations require a significant amount of power to operate pumps. For operation during emergencies, a dual-power source for continuous operation in the event of primary power failure should be provided. The second source of power may be either a totally independent electrical circuit, motor generators using natural gas or diesel, or secondary engine drive for the pumps. Pumps with secondary engine drive have both electric motors and an internal combustion engine connected to the pump drive shaft. A clutch mechanism is installed on each drive so that either can be used to drive the pump.

Access. Pump stations require a great deal of maintenance and attention. Therefore, all weather access roads and parking to the structures is vital.

Flood Protection. The equipment contained in a pump station can be severely damaged by flood waters. Therefore, pump stations must be located above 100-year flood stage or must be protected from floods.

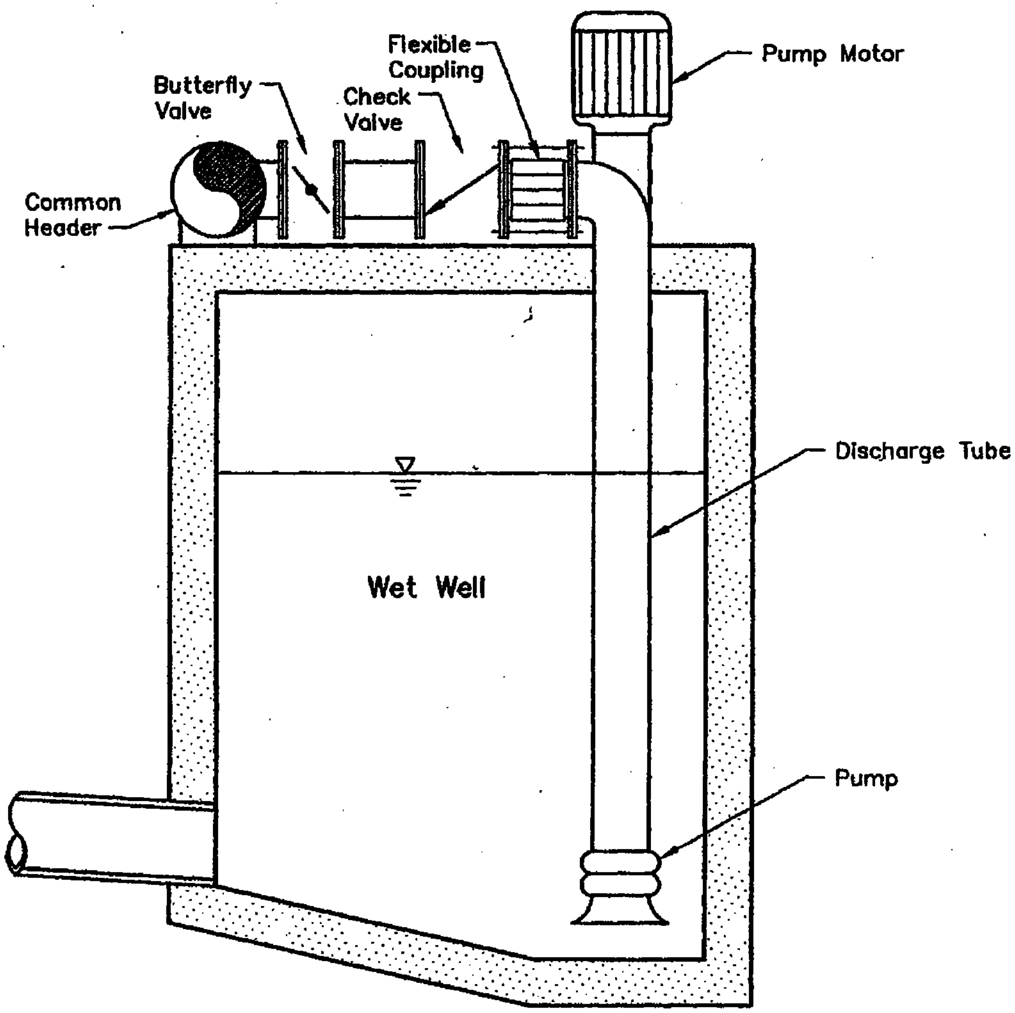
Environmental Consideration. The pumping stations generally are associated with large volumes of wastewater contained within the facility. Odors are of great concern. Location relative to population areas is important. Provision for odor control (such as aeration, chlorination, or hydrogen peroxide treatment) and an air scrubbing system from the pumping building should be made.

9-6-2 Selection of Pumping Station

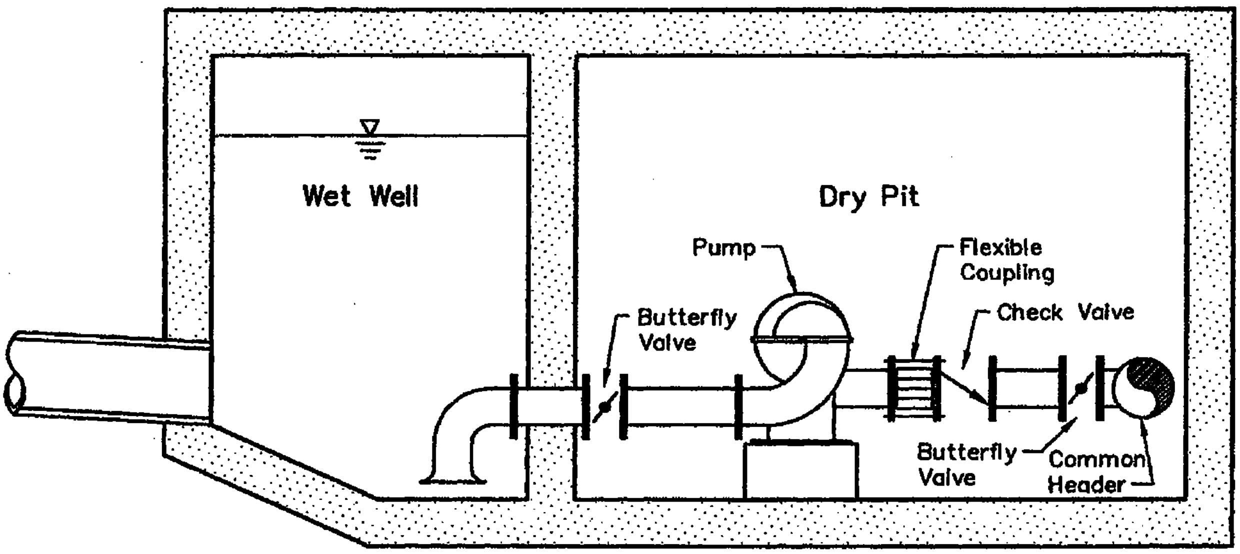
In general, pumping stations can be classified as *wet pit* or *dry pit*. These classifications are based on the location of the pumps relative to the wet well or dry pit. In wet pit pumping stations, the pumps are located in the wet well as illustrated in Figure 9-8(a). In dry pit pumping stations, the pumps are located in a dry enclosure separated from the wet well as illustrated in Figure 9-8(b). Selection of a pumping station is usually based on the pumping equipment selected. For example, submersible and vertical pumps require a wet pit structure, while horizontal pumps usually require a dry pit structure.

Wet well and dry well pumping stations were briefly discussed in Section 9-3. Basic design considerations for wet and dry well pumping stations are given below:

1. The wet and dry wells should be separated by a water- and gastight wall with separate entrances provided to each well.
2. Stairways should have nonslip steps and should be provided in dry wells and in wet well where bar screens or other equipment requiring routine inspection and maintenance may be located. Manhole steps with an entrance hatch should be avoided. Circular or spiral stairways with a walk-in door are preferred.
3. As much as possible, all equipment requiring routine inspection should be installed in the dry well.
4. Ventilation should be provided for all dry wells below ground surface and in all wet wells housing equipment. As a general rule, ventilation equipment should provide at least six air changes per hour or the minimum standard of the applicable building code.
5. Mechanical lifting facilities should be provided where pump, motor, valve, meters, piping, and the like weigh more than 90 kg (198 lb). Hoisting equipment, such as a crane, should be permanently installed if any single item is expected to weigh more than 230 kg (510 lb).



(a)



(b)

Figure 9-8 Common Pump Station Design (courtesy Chiang, Patel & Yerby, Inc.): (a) wet pit pump station and (b) dry pit pump station.

9-6-3 Selection of Pump and Control

Pumping requirements can be determined from knowledge of head and flow. Selection of a specific unit requires examination of manufacturers' rating curves and comparison of these with the system-head capacity curve. If pump selection is left to the manufacturer, the engineer must check its suitability. The following considerations may be given to pump selection:

1. All raw wastewater pumps should have nonclog impellers and should be able to pass 6.5-cm minimum-size objects. The suction and discharge lines should not be less than 10 cm. Inspection and cleanout plates on the bowl of the pump or a head hole at the first fitting connected to the suction line should be provided.
2. The capacity of the pumping station should be such that the expected peak flow can be pumped with the largest pumping unit out of service.
3. The size and number of pumping units for larger stations should be selected so that the range of inflow can be met without starting and stopping pumps too frequently and without requiring excessive wet well storage capacity. Pumping stations discharging directly to the treatment plant should not cause a surging effect. This can be avoided by providing automatically controlled variable-capacity pumps matching the inflow rate. Occasionally, the capacity and depth of the wet well can be coordinated with the constant-speed pumping units so that the rise and fall of the water level in the wet well results in a variable-pumping capacity approximating the inflow rate. This is possible because all centrifugal pumps have the inherent characteristic that, as the head decreases, the capacity increases.
4. No single pumping unit should have a capacity greater than the peak design capacity of the wastewater treatment plant.
5. It is considered good practice to provide space for additional pumps for future needs.
6. For an effective control system, a minimum of 0.6-m differential water head between low and high water level in the wet well should be provided. The air bubbler system is the most reliable system for control. The bubbler system should be equipped with two air compressors or with a standby cylinder of inert gas (CO_2, N_2, etc.) in case the air compressor fails. The bubbler system should also be designed with an automatic purge system to prevent clogging of the bubbler line. Float control systems may be fouled because of grease and floating objects. Considerations should be given to those systems that can help in removal of scum and floating matter.
7. Only self-priming pumps or pumps with an acceptable priming system must be used where suction head is negative. A self-priming pump should include means for venting the air back to the wet well when the pump is priming. Self-priming pumps have caused serious problems; therefore, their use in raw wastewater pumping is not recommended.

9-6-4 Design Considerations of Pumping Station

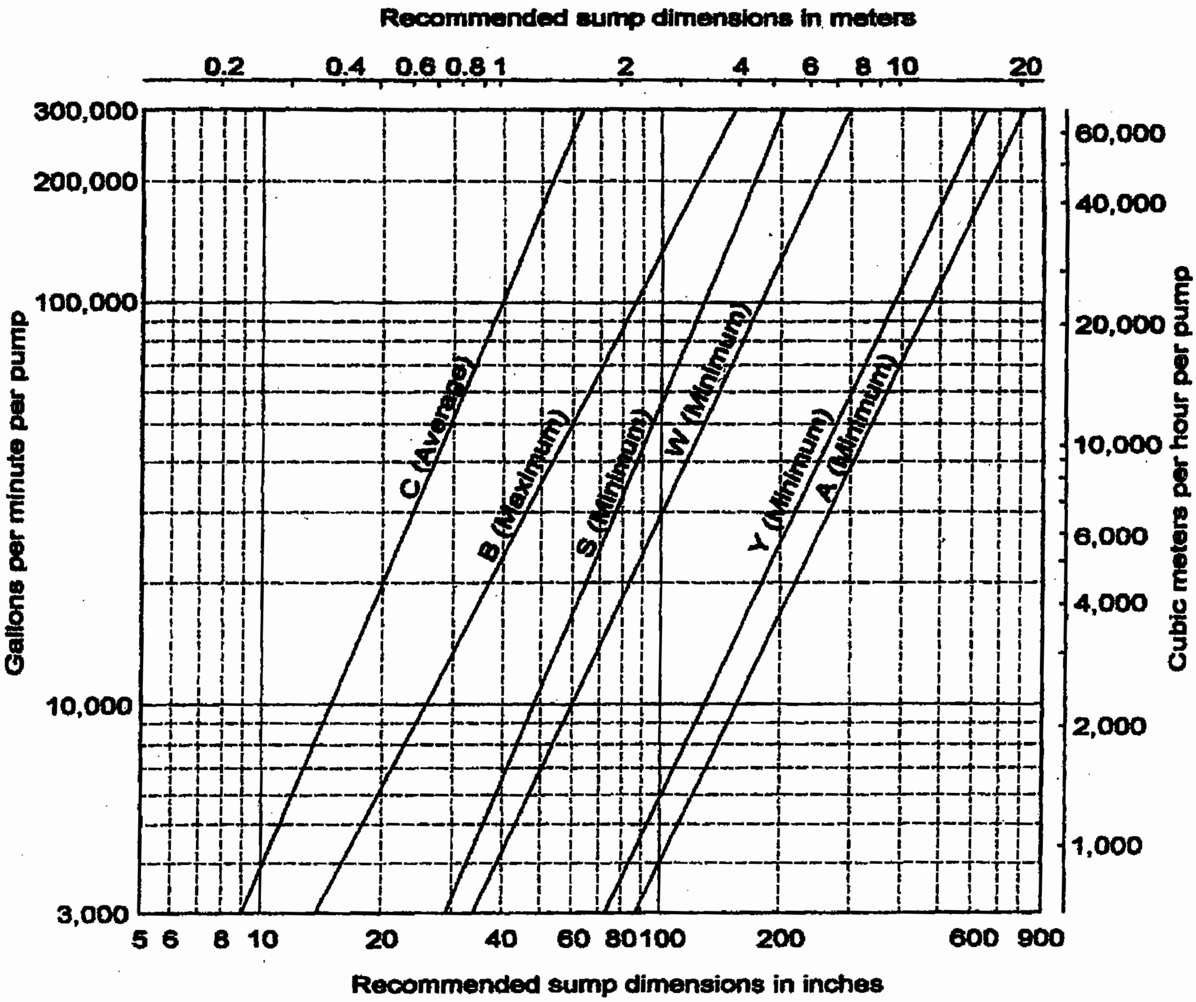
Design of pumping stations involve the layout of the structure and equipment for efficient operation and maintenance. Necessary facilities are required to house pumps and pump controls, spare parts and accessories, and basic facilities for operation and maintenance personnel.

Wet Well and Suction and Discharge Piping. One of the most critical aspects of pump station design is the pump wet well and suction piping. The important consideration in design of a suction well is to provide a smooth transition because turbulence in the pump suction can adversely affect pump performance.

Wet Well. Proper design of suction wells includes considerations for placement of the pump suction relative to the water surface, suction well walls, and other pumps. Velocities and direction of flow in the suction well are other important considerations. The basic design considerations of a wet well are given below:

1. Because of flammable sewer gas, all equipment and electrical work in the wet well must be of explosion-proof and spark-proof construction.
2. Wet well should be divided into two or more sections by isolation gates. This will facilitate cleanup and maintenance.
3. Space for a future pump should be provided near the inlet where the turbulence is at a maximum. Generally, space left at the ends create dead spots where solids can settle, causing odors and other problems.
4. The wet well floor should be horizontal from the wall to a point 0.3 to 0.4 m (12 to 18 in) beyond the outermost edges of the suction bell. The remaining section should be sloped upward to the opposite wall at a slope of 1 : 1 or steeper.
5. The minimum center to center spacing between the pumps and distance from the walls to the center of the suction bell and the height from the floor are given by the Hydraulic Institute. These dimensions may be found in Figure 9-9. Antivortex baffling should be considered for the pump suction in large pumping stations.
6. Bell mouth inlets are better than straight inlets because they eliminate sharp edges on which solids can accumulate and minimize head loss, and there is less possibility of a vortex forming in the wet well when turned-down bellmouth inlets are used. The bell should not be more than $D/2$ and not less than $D/3$ above the floor (where D is the diameter of the suction bell). In order to obtain scouring inlet velocities and still obtain nearly optimum hydraulic entrance conditions, velocities at the entrance should be kept less than 3 m/s.
7. Consideration should be given to the submergence of the pump suction inlet to prevent vortexing and smooth transition of flow. Hydraulic Institute recommendations are given in Figure 9-9.
8. Wet well volume must be established with respect to pump type and operation (constant-speed or variable-speed drive). Constant-speed pumps require a larger wet well volume to prevent short cycling of the pump (frequent starting and stopping). For variable pumps, sufficient volume, i.e., time, should be provided for the change in capacity when a pump is started and brought to full speed before the start or stop point of the next pump is reached. Many regulatory agencies limit maximum retention time at average design flow. This criteria is to minimize septic condition and odor problems. Volume calculations for wet wells are given Refs. 2, 5, and 9.

(a)



(b)

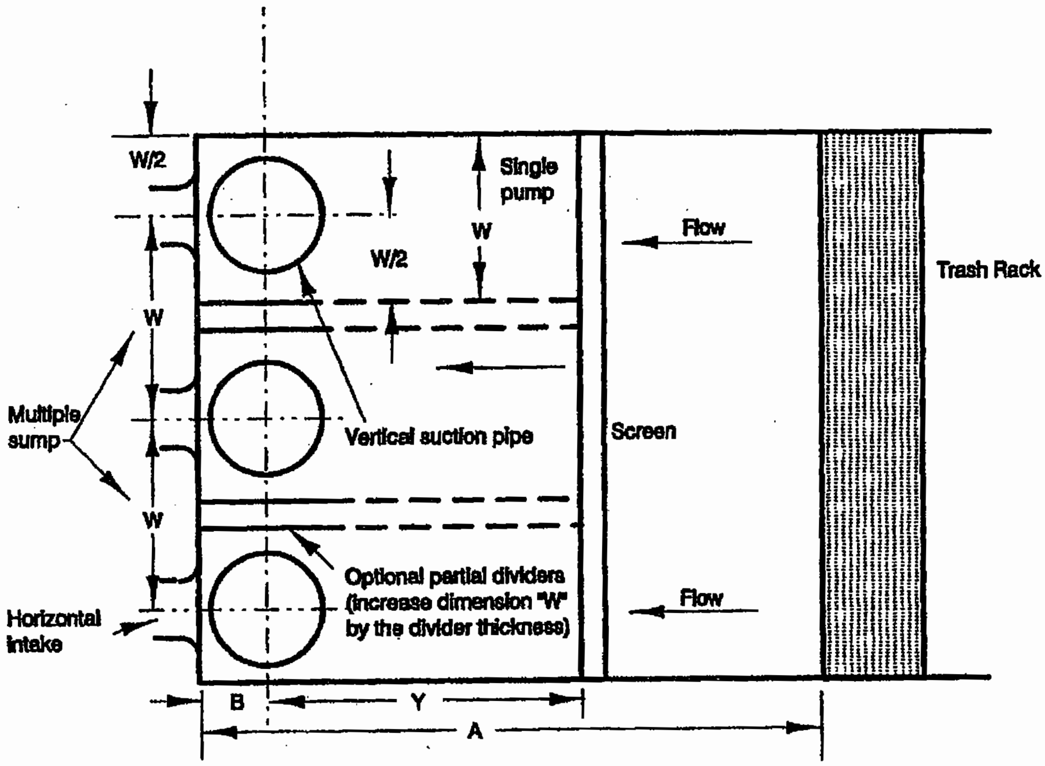


Figure 9-9 Suction Well Dimensions (from Ref. 1; courtesy of the Hydraulic Institute): (a) sump dimensions for different flows; (b) sump dimensions—plan view.

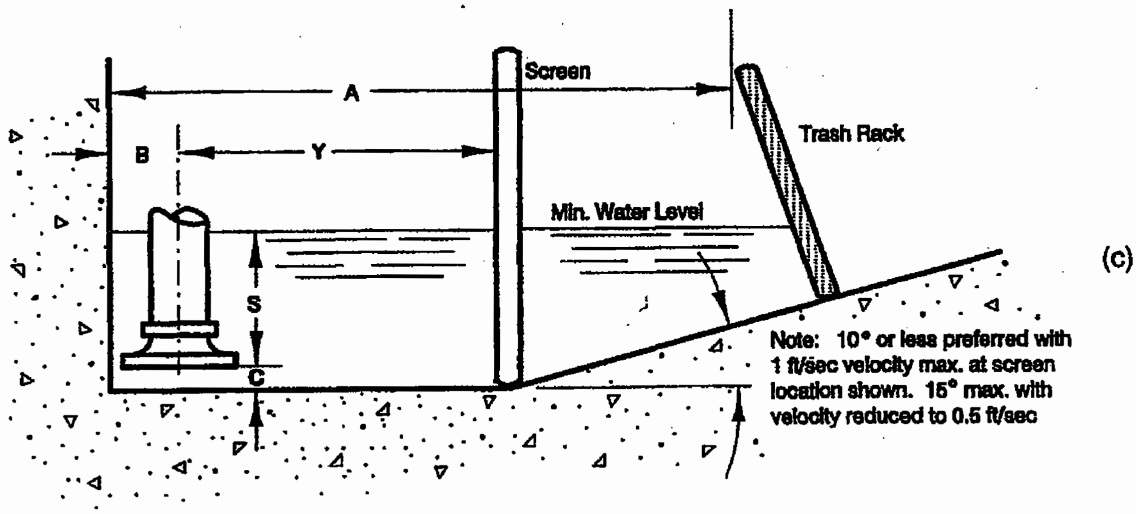


Figure 9-9—cont'd (c) sump dimensions elevation view using wet well-type pump.

Suction Piping. Important design considerations for design of pump suction piping include prevention of air pockets and reduction of turbulence in the suction piping. To prevent air pockets, suction piping should be leveled or sloped slightly upward toward the pump. If reducers are required, they should be eccentric-type for horizontal piping and concentric for vertical piping. To prevent turbulence, suction piping should be straight and no smaller than the pump suction nozzle. If bends are required, they should have a long radius. Many recommended designs of suction piping and undesired conditions are illustrated in Figure 9-10.

Flexible couplings should be installed on the pump suction, as well as discharge piping, to reduce vibration and prevent temperature effects on piping from stressing the pump casing. When flexible couplings are provided, the pump foundation and base plate must be designed to resist the thrust forces created by the flexible coupling.

Pump suction piping should be oversized to facilitate future expansion of the pumping station. Desirable velocities in pump suction piping are 1.0 to 1.8 m/s. A gate valve should be installed in the suction piping. With the gate valve closed, the pump should be opened for repairs without flooding the pump room.

If site-specific limitations dictate variances from these recommendations, suction model tests should be conducted. These tests will verify the operational features of the proposed design of the suction well. Design modifications based on such tests will provide considerable cost savings and trouble-free operation.

Discharge Piping. The velocities in the discharge piping at maximum pump discharge should range from 1.8 to 2.4 m/s. A concentric increaser, check valve, gate valve, pressure gauge, other fittings, and a tee to connect to the pump common header are generally provided.

Pump Station Ventilation and Cooling. Pumping equipment generates a great deal of heat during operation. This heat is typically generated from lost efficiencies of the pump driver. If this heat is not removed from the pump station, temperature will become ex-

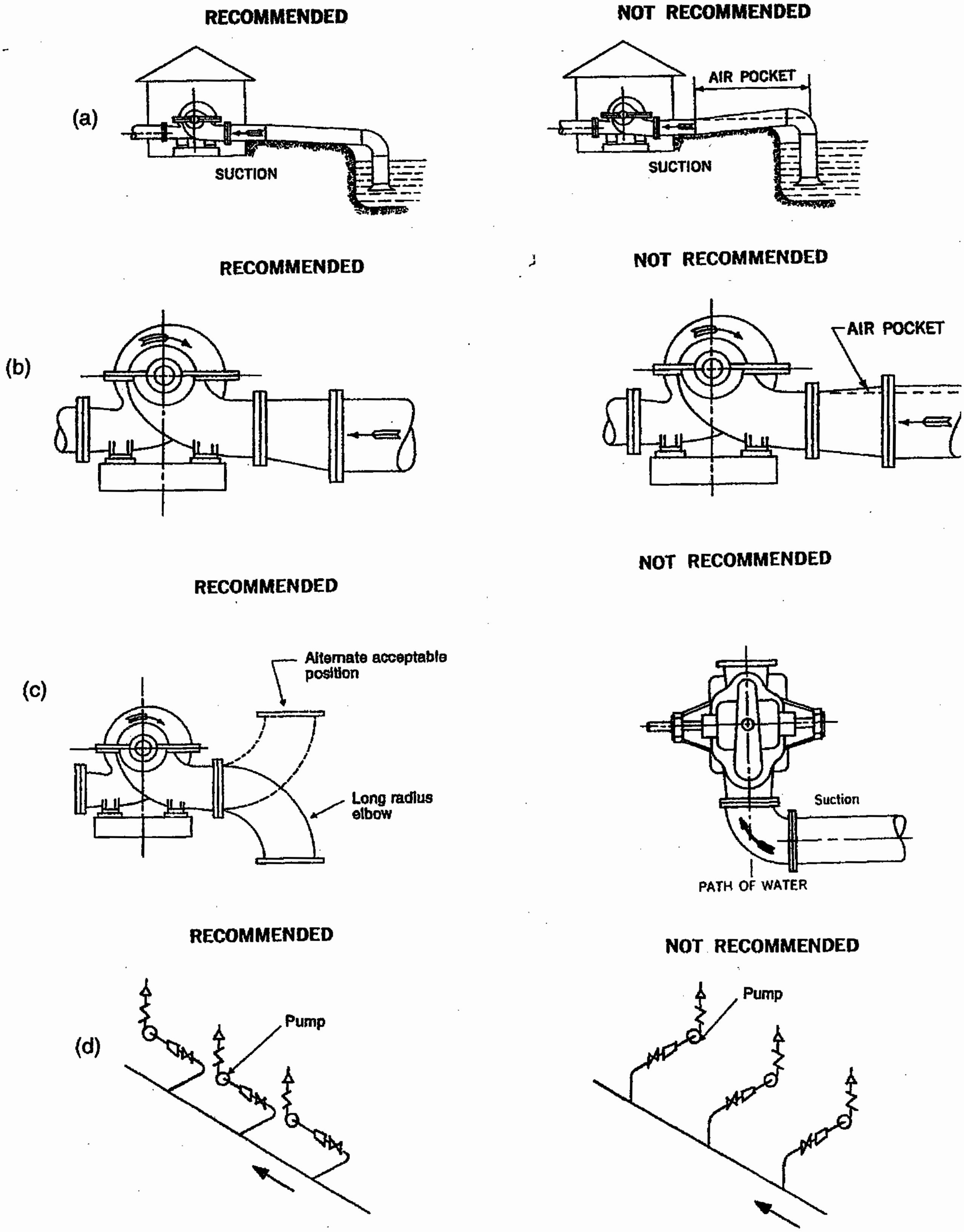


Figure 9-10 Recommended Suction Piping Layouts and Undesired Design Condition: (from Ref. 1; courtesy of the Hydraulic Institute): (a) suction piping should be level or have a slope upward towards the pump to prevent air traps; (b) suction piping should use ec· centric reducers to prevent air traps; (c) avoid suction piping configurations that cause ar imbalance in water distribution to the pump suction; and (d) piping arrangement for three pumps arranged for parallel operation.

cessive and may damage the equipment. The heat load generated by electric motors is calculated from Eq. (9-16).[11,12]

$$H = PUL\left(\frac{1-E}{E}\right) \tag{9-16}$$

where

E = motor efficiency in decimal fraction
H = heat load, kW
L = motor load factor, 1.0 or decimal fraction. The motor load factor is the fraction of the load being delivered under the estimated cooling conditions.
P = motor power rating, kW. The motor power rating is the power output of the motor.
U = motor use factor, 1.0 or decimal fraction. The motor use factor may be applied when motor use is intermittent with significant nonuse during all hours of operation. For conventional application its value could be 1.0.

The pump stations are typically cooled by forced air ventilation. This method requires forcing a sufficient quality of outside air through the pump station to remove the heat generated from the pump operation. Eq. (9-17) is typically used to estimate heat removed by air flow.[11]

$$Q = \frac{H}{C_P\,\rho\,(T_i - T_o)} \tag{9-17}$$

where

Q = air flow rate required to remove heat, m^3/s (ft^3/hr)
H = quality of heat removed, kW (Btu/hr)
C_P = specific heat of air; the specific heat of air at 77°C is 1.02 kW-s/kg-°C or 1.02 kJ/kg · K (0.245 Btu/lb-°F)
ρ = density of air, 1.2 kg/m^3 (0.075 lb/ft^3)
T_i = temperature of indoor air, °C (°F)
T_o = maximum outdoor temperature, °C (°F)

During some periods of the year, the wastewater temperature will be less than the due point of the ambient air. When this occurs, condensation will form over pumps, piping, and other metallic surfaces in contact with water. This can be prevented by (1) installing air driers to reduce the due point of the ambient air or (2) insulating all metallic surfaces in contact with the water. Usually both these options may not be economically justified. Therefore, engineers must make provisions in pump room floors to collect and convey condensation or any other liquid drips.

Equipment Access. The layout of pump stations should provide access to all pumps, valves, piping, and miscellaneous equipment for maintenance. Lifting equipment, such as overhead bridge cranes are often installed in larger pump stations to facilitate repair and removal of equipment.

Miscellaneous Equipment and Facilities

Electrical Switchgear. Sufficient room for electrical switchgear cabinets should be provided in the pump station. The space required will depend on the power rating of the motors, voltages, and complexity of the control system. The reader should seek assistance from experienced electrical engineers for information on switchgear space requirements for specific installations.

Control Facilities. Space should be provided for instrumentation and control equipment. These facilities may range from a simple relay cabinet to computer control facilities. In larger installations, a dedicated room for control equipment is often provided. An instrumentation and control engineer should be consulted for information on space requirements for control facilities.

Potable Water and Sanitary Facilities. Pump stations should provide potable water and sanitary services for operation and maintenance personnel. A clean water supply is often required for pump seal and bearing lubrication. Pump stations should be provided with toilet, shower, and locker facilities for operators' comfort. Additionally, all floor drains should be discharged into sanitary sewers.

9-7 MANUFACTURERS OF PUMPING EQUIPMENT

A list of the manufacturers of pumping equipment is provided in Appendix D. The responsibilities of the design engineer for equipment selection are given in Sec. 2-10.

9-8 INFORMATION CHECKLIST FOR DESIGN OF PUMPING STATION

Before designing a pumping station, the design engineer must develop design data and make many decisions that are necessary for good design. Some of the important items are listed below:

1. Characteristics of the liquid that must be pumped (suspended solids, floating solids, maximum size of the objects, density, temperature, pressure, etc.)
2. Expected flow range (minimum, average, and peak design flows)
3. Site plan, piping schematic, and hydraulic profile from wet well to the receiving facility
4. Minimum and maximum water surface elevations in the wet well and elevation of water surface at receiving facility: These values are established after evaluating the liquid elevation in influent sewer, plant layout, and hydraulic considerations through

the plant, flood protection, site plan considerations, and operating and maintenance requirements. A great deal of engineering judgment and experience is required in establishing these elevations.

5. Type of pumping station (wet well or dry well) and suction conditions [including limits of submergence, suction head, suction lift, and available *net positive suction head* (NPSH)]
6. System head capacity curves
7. Initial pumping unit selection to include types and number of pumps, constant- or variable-speed drive, specific speed, etc.
8. Drive unit (electrical motor or engine) and expected power requirements
9. Design criteria for pumping station prepared by the concerned regulatory agencies
10. Equipment manufacturers and equipment selection guide (catalog)

9-9 DESIGN EXAMPLE

9-9-1 Design Criteria Used

1. Design the pumping equipment for peak, average, and minimum design flows of 1.321, 0.440, and 0.220 m^3/s (see Table 6-9).
2. The site plan, piping schematic, and hydraulic profile developed for the Design Example are shown in Figures 9-11 and 9-12.[b]
3. Based on the preliminary examination of the plant layout, piping, and hydraulic considerations, the following elevations have been assigned:
   - floor level elevation of wet and dry well are assumed = El. 0.00 m
   - minimum water surface elevation in the wet well = El. 2.44 m
   - maximum water surface elevation in the wet well = El. 3.66 m
   - normal water level elevation in the grit chamber = El. 13.57 m
4. As an initial selection, provide a total of five pumping units of equal size, including one unit as a standby. Select dry well centrifugal pumps with variable-speed drive.

9-9-2 Design Calculations

Step A: Preparation of System Curves. System losses from the common pump header (Figures 9-11 and 9-12) to the point of application at the grit removal facility should be calculated by making an inventory of all system valves, fittings, and specials. Also friction losses through the force main must also be calculated. The system losses and friction losses should be calculated for different flow conditions covering the entire anticipated operating flow ranges. These calculations are given below:

1. Compute system losses for valves, fittings, and specials.

   The system losses are calculated using Eq. (9-4). The values of head loss coefficients

[b]In this design a single header force main is provided from the pumping station to the grit chamber. Many designers prefer two separate lines or a loop around the flow measurement device. Such arrangement provides added safety in case the flow measurement device needs repairs. A retention basin is provided in this design (see Chap. 6) to bypass and store the flow should such an emergency develop.

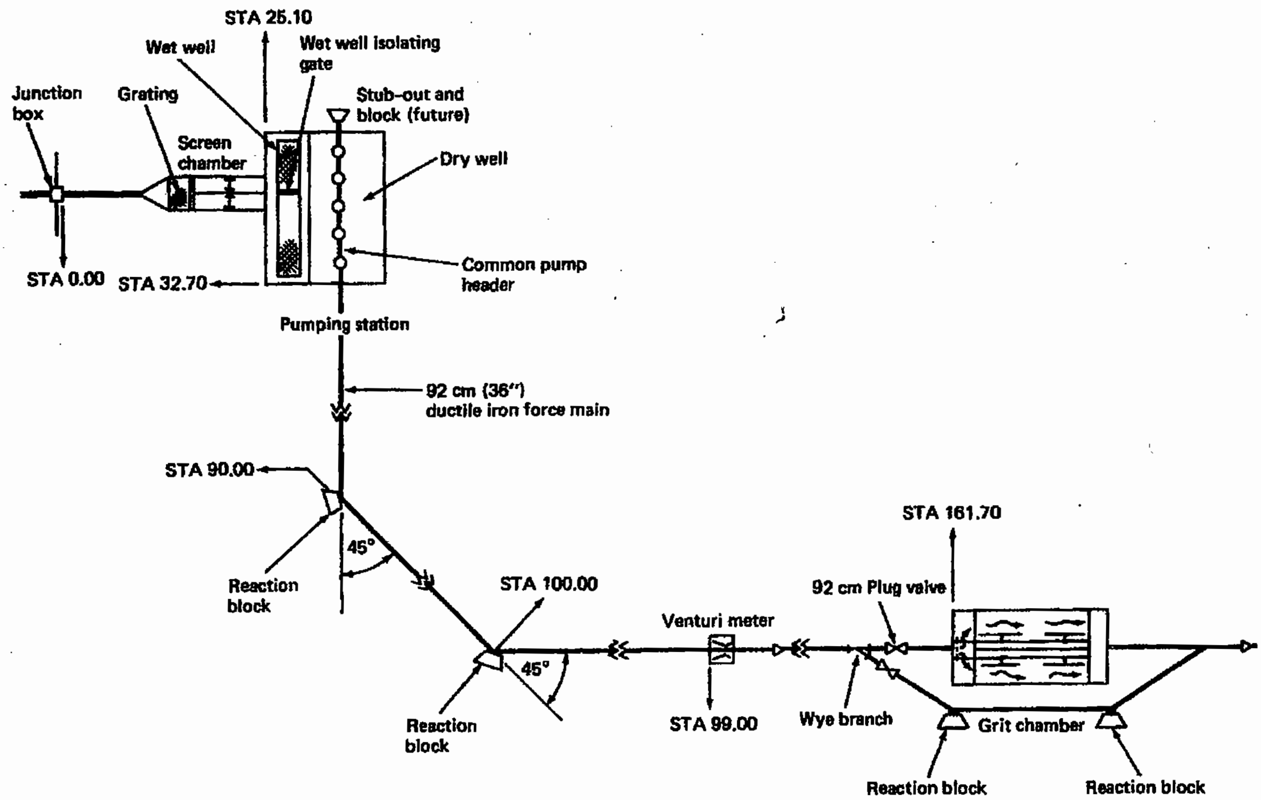


Figure 9-11 Piping Schematic and Layout of Bar Screen, Pumping Station, Flow Meter, and Grit Chamber. Reference Station STA 0.00 Is at the Junction Box. See Figure 20-4 for Details of Yard Pipings.

for different fittings and calculated values of head losses for six different flow conditions are summarized in Table 9-5. The common header (force main) is a ductile iron pipe 0.92 m (36 in.) in diameter, and therefore all fittings are compatible with this pipe.

2. Compute head loss in the force main.
   Friction losses through the force mains are difficult to predict because the observed Hazen-Williams C value may vary substantially with different materials and service lines. It has been customary to use a C value of 140 for new pipes and 100 for old pipes. These considerations are important because they will affect the pump selection and operating conditions.
   The head losses through the proposed force main for two extreme conditions of Hazen-Williams C of 140 and 100 are calculated for several assumed flows. These values are given in Table 9-5.
3. Compute total head losses.
   Total head losses at different flow conditions at C of 140 and 100 are obtained by adding the friction head losses in the force main, fittings, valves and specials, and velocity head. These values are also given in Table 9-5.
4. Develop system head curve.
   The total dynamic heads (TDH) at maximum and minimum static heads (due to fluctuations in the wet well elevations) are obtained by adding total head losses to the respective static heads. A summary of these calculations is given in Table 9-6. The total

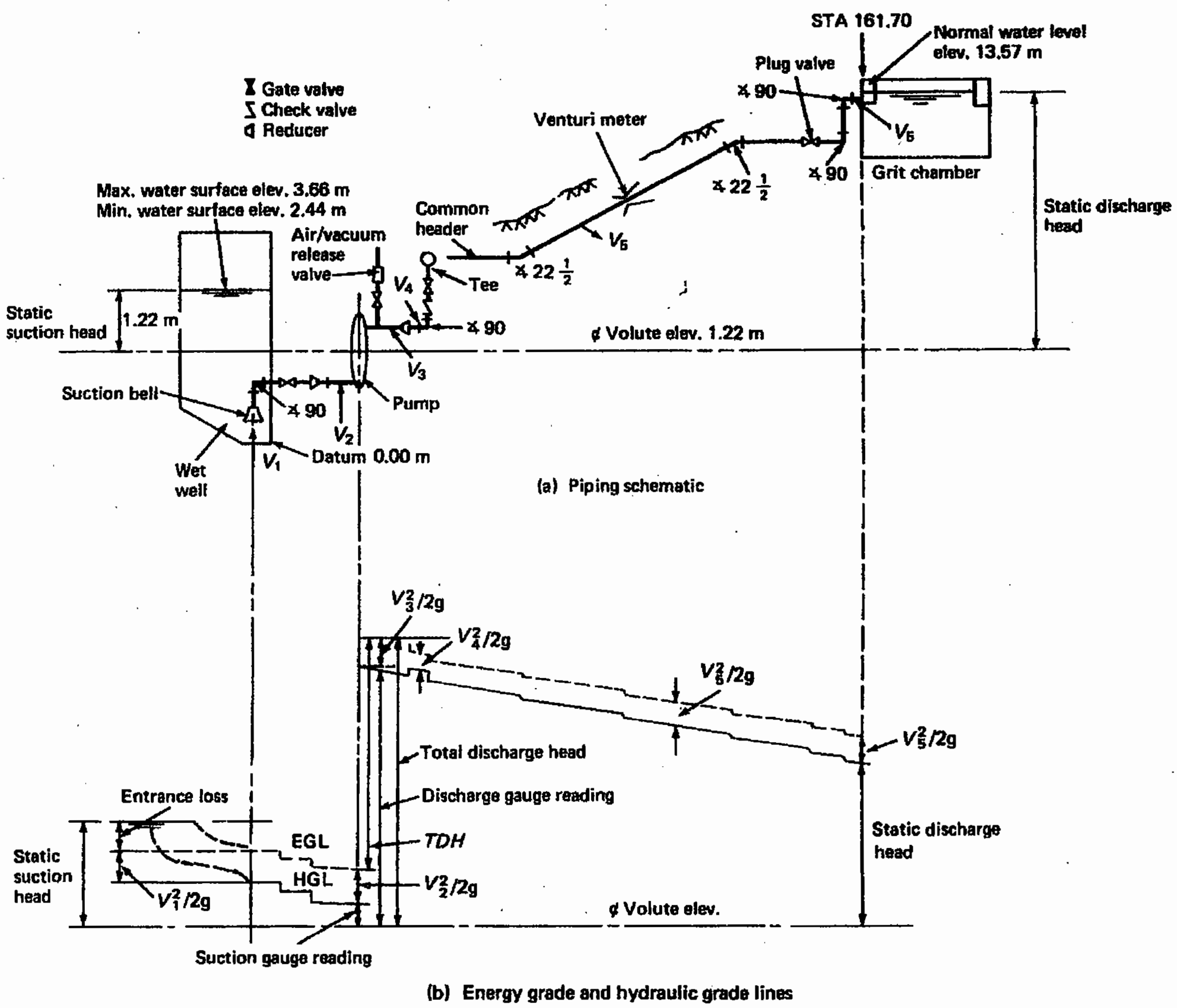


Figure 9-12 Piping Schematic Profile of Energy Grade and Hydraulic Grade Lines. All Elevations Are with Respect to Assumed Datum at the Floor Level of the Wet Well. See Figure 9-8 for STA Location.

dynamic head losses given in Table 9-6 are plotted against assumed pumping rates in Figure 9-13. It should be noted that system head curves are plotted for minimum and maximum wet well water levels and at Hazen-Williams C values of 140 and 100. It should also be noted that only the head losses in the common header (that part of pipe system common to all pumps) are included in the system head curve computations.

Step B: Preparation of Pump Characteristic Curves. Once the system head curves are plotted, the individual pump characteristic curves are superimposed on the system head curves (Figure 9-13). The pump's characteristic performance varies with the type of pumps and the manufacturers. It is customary to evaluate the characteristic performance of several different pumps supplied by various manufacturers. The pump that exhibits the best efficiencies under the system curves must be selected. The pump that exhibited the best performance and efficiency characteristics under the given system curve for the Design Example is shown in Figure 9-14. This pump has a 54-cm-diameter (21¼-in.) im-

TABLE 9-5 System Head Loss Calculations[a]

Item No.	Valves, Fittings, and Specials in 92-cm-Diameter Force Main	Numbers Provided	K Value	Assumed Flow 0.253 m³/s (4000 gpm) Velocity (m/s)	Head Loss (m)	Assumed Flow 0.507 m³/s (8000 gpm) Velocity (m/s)	Head Loss (m)	Assumed Flow 0.761 m³/s (12,000 gpm) Velocity (m/s)	Head Loss (m)	Assumed Flow 1.013 m³/s (16,000 gpm) Velocity (m/s)	Head Loss (m)	Assumed Flow 1.267 m³/s (20,000 gpm) Velocity (m/s)	Head Loss (m)	Assumed Flow 1.521 m³/s (24,000 gpm) Velocity (m/s)	Head Loss (m)
1	90° Elbow	2[b]	0.30	0.38	0.004	0.76	0.018	1.14	0.040	1.52	0.071	1.91	0.112	2.29	0.16
2	45° Elbow	2[c]	0.20	0.38	0.003	0.76	0.012	1.14	0.026	1.52	0.047	1.91	0.074	2.29	0.107
3	22½° Elbow	2[b]	0.15	0.38	0.002	0.76	0.009	1.14	0.020	1.52	0.035	1.91	0.056	2.29	0.080
4	Venturimeter (see Chapter 10)	1[d]	0.14	1.52	0.017	3.05	0.066	4.58	0.150	6.10	0.266	7.62	0.414	9.15	0.597
5	Wye branch	1[c]	1.00	0.38	0.007	0.76	0.030	1.14	0.066	1.52	0.118	1.91	0.186	2.29	0.267
6	Plug valve	1[d]	1.00	0.38	0.007	0.76	0.030	1.14	0.066	1.52	0.118	1.91	0.186	2.29	0.267
7	Outlet (velocity head)	1[d]	1.00	0.38	0.007	0.76	0.030	1.14	0.066	1.52	0.118	1.91	0.186	2.29	0.267
	Subtotal	—	—	—	0.047	—	0.195	—	0.434	—	0.773	—	1.214	—	1.745
				Assumed Flow 0.253 m³/s Head Loss $C = 100$	Head Loss $C = 140$	Assumed Flow 0.507 m³/s Head Loss $C = 100$	Head Loss $C = 140$	Assumed Flow 0.761 m³/s Head Loss $C = 100$	Head Loss $C = 140$	Assumed Flow 1.013 m³/s Head Loss $C = 100$	Head Loss $C = 140$	Assumed Flow 1.267 m³/s Head Loss $C = 100$	Head Loss $C = 140$	Assumed Flow 1.521 m³/s Head Loss $C = 100$	Head Loss $C = 140$
8	Head losses in 92 cm force main, station 32.70 m to station 161.70 m (length = 129.00 m)[e]			0.032	0.017	0.117	0.063	0.247	0.132	0.420	0.225	0.641	0.344	0.896	0.481
Total				0.079	0.064	0.312	0.258	0.681	0.566	1.193	0.998	1.855	1.558	2.641	2.226

[a]All calculations are for the common header from pump station to grit camber.
[b]Shown in Figure 9-12.
[c]Shown in Figure 9-11.
[d]Shown in Figures 9-11 and 9-12.
[e]Head loss is calculated using Hazen-Williams Eq. (9-3).

TABLE 9-6 Summary Computations of Total Dynamic Head (System Curve)

Assumed Flow (m^3/s)	Friction Loss[a] (m)		Total Discharge Head (m)		Total Dynamic Head (TDH) (m)			
					Min. Wet Well		Max. Wet Well	
	$C = 100$	$C = 140$	Min. Wet Well	Max. Wet Well	$C = 100$	$C = 140$	$C = 100$	$C = 140$
0.253	0.08	0.06	11.13	9.91	11.21	11.19	9.99	9.97
0.507	0.31	0.26	11.13	9.91	11.44	11.39	10.22	10.17
0.761	0.68	0.57	11.13	9.91	11.81	11.70	10.59	10.48
1.013	1.19	1.00	11.13	9.91	12.32	12.13	11.10	10.91
1.267	1.86	1.56	11.13	9.91	12.99	12.69	11.77	11.47
1.521	2.64	2.23	11.13	9.91	13.77	13.36	12.55	12.14

[a]Total head losses from Table 9-5 (rounded to second place of decimal).

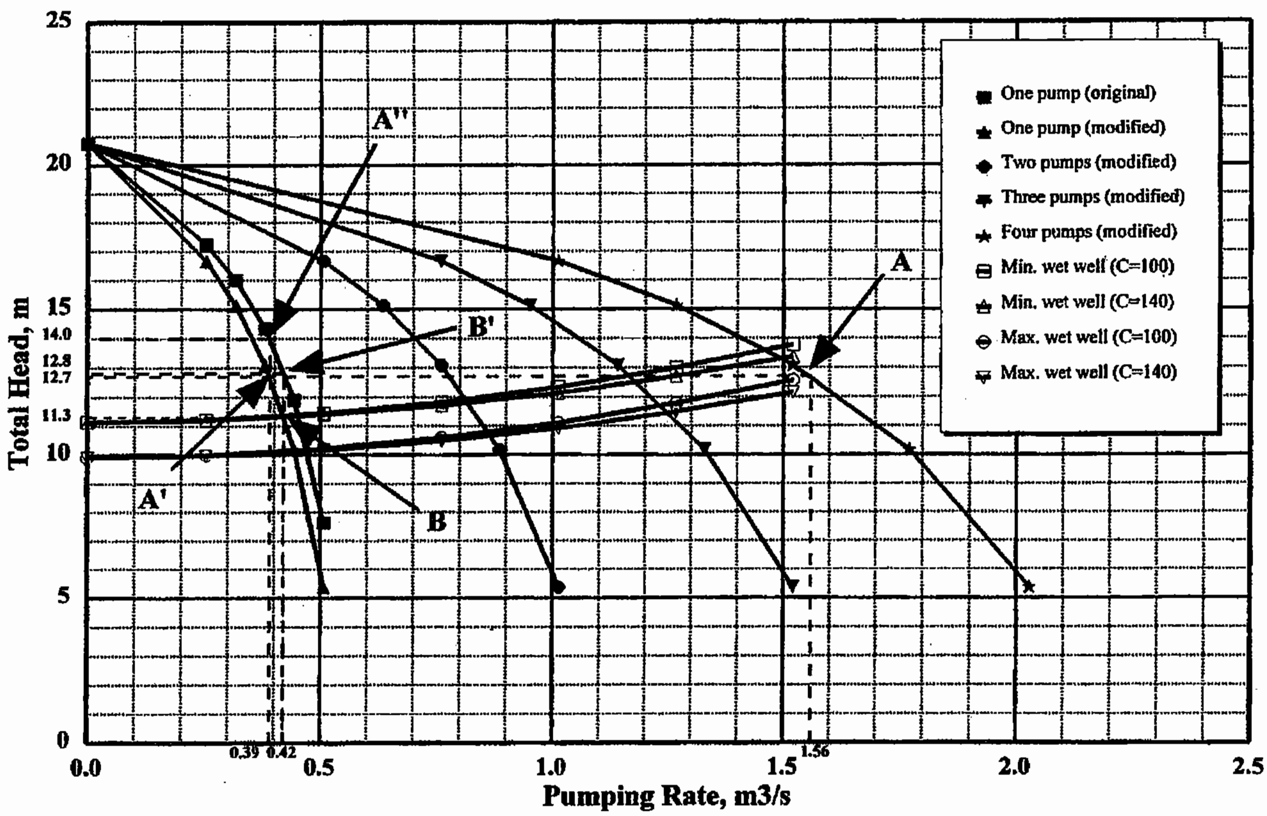


Figure 9-13 System Head Capacity, Pump-Modified Head Capacity, and Pumps Parallel Combination Curves Showing Operating Heads and Capacities (courtesy Chiang, Patel & Yerby, Inc.)

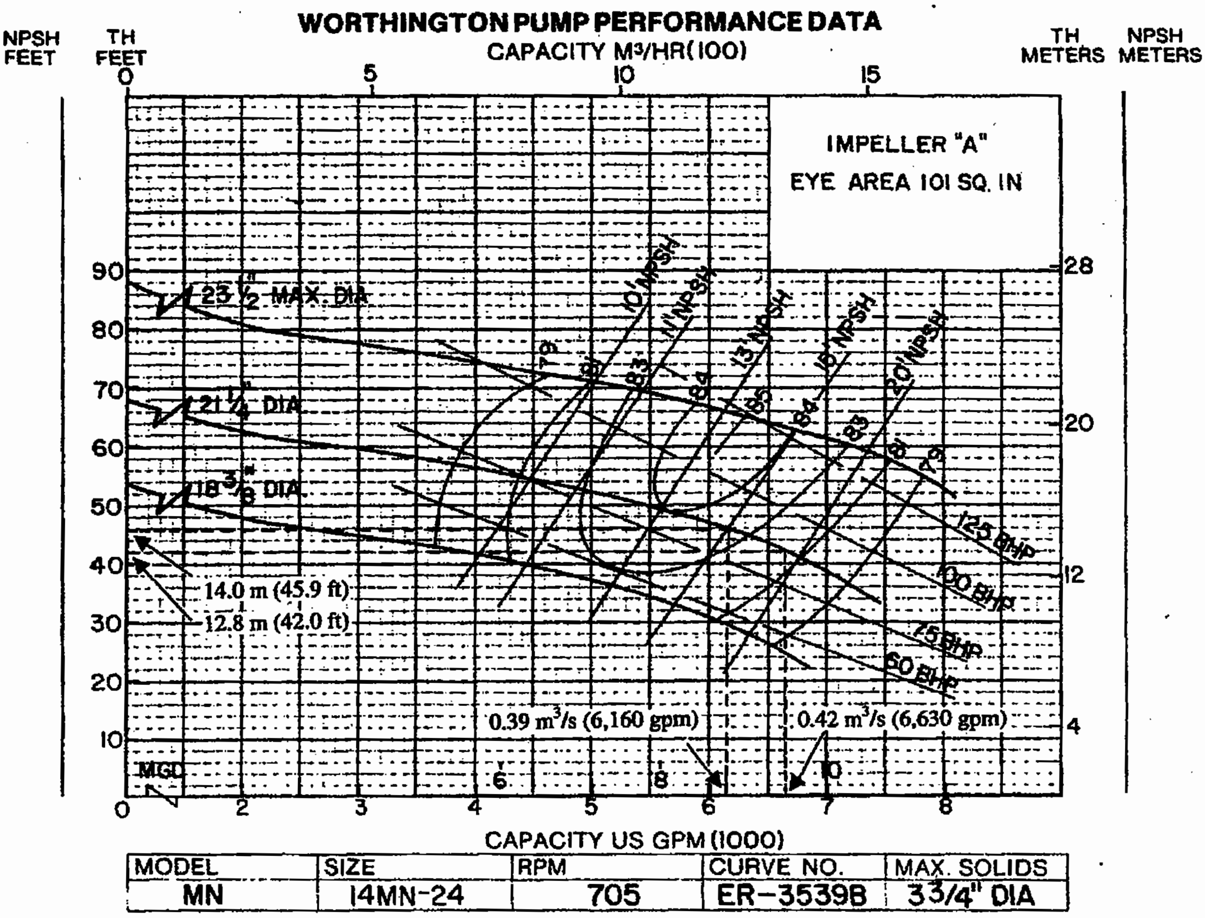


Figure 9-14 Centrifugal Pump Selected in the Design Example (courtesy Worthington Pump Corporation).

peller and variable-speed drive. The procedure and steps involved in selection of this pump are given below:

1. Prepare modified pump curve.
   The manufacturer's head capacity curve is developed from the gauge reading at suction and discharge lines. It is necessary to account for the losses in the individual pump pipings. These losses are computed from the suction inlet to the station header for varying pumping rates through the individual pump. Calculations are presented in Table 9-7. These individual pump losses are subtracted from the manufacturer's pump curve as shown in Table 9-8. Modified pump curves for the selected pump is shown in Figure 9-13.
2. Prepare pump combination curves.
   Four identical pumps are combined in parallel to provide the peak design flow. Therefore, combined modified pump curves with two, three, and four pumps in parallel are developed using data from Table 9-6. These plots represent combined modified pump curves showing multiple pump operations in parallel.
3. Determine pump's operating head and capacity and efficiency.
   The pumping head and capacity of the station are determined from the intersection of system head curves and the combined modified pump curve (Figure 9-13). These val-

TABLE 9-7 *Determination of Individual Pump Losses for Preparation of Modified Pump Curve*

				Assumed Flow 0.253 m^3/s (4000 gpm)		Assumed Flow 0.317 m^3/s (5000 gpm)		Assumed Flow 0.380 m^3/s (6000 gpm)		Assumed Flow 0.443 m^3/s (7000 gpm)		Assumed Flow 0.507 m^3/s (8000 gpm)	
Item No.	**Valves, Fittings, and Specials**	**No. Provided**	***K* Value**	**Velocity (m/s)**	**Head Loss (m)**	**Velocity (m/s)**	**Head Loss (m)**	**Velocity (m/s)**	**Head Loss (m)**	**Velocity (m/s)**	**Head Loss (m)**	**Velocity (m/s)**	**Head Loss (m)**
1	Entrance, suction bell (32 in.) 81 cm	1[a]	0.04	0.49	0.001	0.615	0.001	0.737	0.001	0.860	0.002	0.984	0.002
2	90° Elbow (24 in.) 61 cm	1[a]	0.30	0.87	0.012	1.085	0.018	1.300	0.026	1.516	0.035	1.735	0.046
3	Gate valve (24 in.) 61 cm	1[a]	0.19	0.87	0.007	1.085	0.011	1.300	0.016	1.516	0.022	1.735	0.029
4	Reducer (14 in.) 35.5 cm	2[a]	0.25	2.56	0.167	3.203	0.262	3.839	0.376	4.476	0.511	5.122	0.669
5	Check valve (20 in.) 51 cm	1[a]	2.50	1.24	0.196	1.552	0.307	1.860	0.441	2.168	0.599	2.482	0.785
6	90° Elbow (20 in.), 51 cm	1[a]	0.30	1.24	0.024	1.552	0.037	1.860	0.053	2.168	0.072	2.482	0.094
7	Gate valve (20 in.) 51 cm	1[a]	0.19	1.24	0.015	1.552	0.023	1.860	0.034	2.168	0.046	2.482	0.060
8	Tee (20 in. × 20 in.) 51 cm × 51 cm	1[b]	1.80	1.24	0.141	1.552	0.221	1.860	0.317	2.168	0.431	2.482	0.565
	Total	—	—	—	0.563	—	0.880	—	1.264	—	1.718	—	2.250

[a]Shown in Figure 9-12.

[b]The tee connects the pump discharge pipe to the common station header. Location of the tee is shown in Figure 9-12.

Note: The friction loss in the connecting piping is small and therefore is not included in the preparation of modified pump curves.

TABLE 9-8 Calculations for Preparation of Modified Pump and Parallel Pump Combination Curves

Pump Characteristic Head-Capacity Curve		Individual Pump Head Loss[b] (m)	Pump-Modified Head (m) (One Pump)	Pump Capacity, Parallel Combination		
Capacity (m^3/s)	Total Head[a] (m)			2 Pumps (m^3/s)	3 Pumps (m^3/s)	4 Pumps (m^3/s)
0.253	17.22	0.56	16.66	0.506[c]	0.759[d]	1.012[e]
0.317	16.00	0.88	15.12	0.634	0.951	1.268
0.380	14.33	1.26	13.07	0.760	1.140	1.520
0.443	11.89	1.72	10.17	0.886	1.329	1.772
0.507	7.62	2.25	5.37	1.014	1.521	2.028

[a]Values taken from Figure 9-14 for 21¼-in.-diam. (54-cm) impeller corresponding to assumed pump capacity.
[b]Values taken from Table 9-7.
[c]$2 \times 0.253 = 0.506$.
[d]$3 \times 0.253 = 0.759$.
[e]$4 \times 0.253 = 1.012$.
Note: It is customary not to extend the manufacturer's curve. Points beyond the manufacturer's curve may be associated with excessive power demand and cavitation. A point corresponding to pump capacity of 0.507 m^3/s (8000 gpm) is outside the manufacturer's curve for 21¼-in.-diameter impeller (Figure 9-14).

ues vary with the water level in the wet well. Many station-operating conditions are summarized in Table 9-9. Important operating conditions are discussed as follows: (a) Point A is important to describe the operating conditions of the pumps when the wet well is near maximum level (minimum static head) and all four pumps are operating. Under this condition each pump will have operating head and capacity corresponding to point A′, or point A″ with reference to manufacturer's characteristic curve. On the other hand, when the wet well has a minimum level (maximum static head), only one pump will be pumping, and the operating head and capacity are indicated by point B. This corresponds to point B′ on manufacturer's curves. These are two extreme conditions under normal operation of the pump station and should be used to determine the range of pump efficiency.[c] The pump efficiency for these two points is marked in Figure 9-14. (b) The capacity of pumping station is 1.56 m^3/s (point A) when the wet well is at a maximum level.[d] This capacity is considerably higher than the peak design flow of 1.321 m^3/s. Furthermore, the minimum station capacity of 1.45 m^3/s is also kept slightly higher than the peak design flow. This point corresponds to the intersection of the head-capacity curve of four pumps, and the system heads-capacity curve at a minimum wet well level. Because of uncertainties in forecasting friction losses through the pipings and uncertainties in predicting flow rate, higher station ca-

[c]It is important that the motor power selected be at least 10–15 percent greater than the maximum power input required to the pump. This will prevent motor overloading over the entire range of pump curve and enhance motor life. Additionally, the motor will operate at its most efficient point, which is usually around 80 percent of its full load. The 82-kW (110 hp) motor is the next standard available motor. This will give adequate reserve power (see Table 9-10).

[d]Only one pump operates at low wet well level, and all four pumps operate at maximum wet well elevation.

TABLE 9-9 Summary of Pump Operating Heads and Capacities

Condition and Description	Reference Point in Figure 9-10	Operating Head		Operating Capacity		Remarks
		m	ft	m^3/s	gpm	
At minimum static head (maximum wet well level), $C = 100$, four pumps in parallel combination	A	12.7	41.7	1.56	24,630	Maximum station capacity with four pumps in parallel operation
Project horizontally from point A to the individual modified pump curve	A′	12.7	41.7	0.39	6,160	Operating condition of each pump when four pumps are arranged in parallel
Project vertically from point A′ to the individual pump characteristic curve	A″	14.0	45.9	0.39	6,160	Line A′A″ represents station losses (1.3 m)[a]
At maximum static head (minimum wet well level), $C = 100$, one pump in operation (modified pump curve)	B	11.3	37.1	0.42	6,630	Operating condition of one pump
Project vertically from point B to pump characteristic curve	B′	12.8	42.0	0.42	6,630	Line BB′ represents station losses (1.5 m)[a]

[a]Station losses are the head losses in the suction and discharge piping of each pump (see Sec. 9-5-8 and Table 9-5).

TABLE 9-10 Summary Calculations of Pump Efficiency, Power Output of the Pump, Power Input to the Pump, and Motor Power

Operating Conditions for a Single Pump	Maximum Wet Well Level (Minimum Static Head, Four Pumps Operating)	Minimum Wet Well Level (Maximum Static Head, One Pump Operating)
Pump operating head	14.0 m (45.9 ft) (point A″)	12.8 m (42.0 ft) (point B′)
Pump operating discharge	0.39 m^3/s (6160 gpm) (point A″)	0.42 m^3/s (6630 gpm) (point B′)
Pump operating efficiency	83.4 percent (Figure 9-14)	81.7 percent (Figure 9-14)
Power output of the pump, water power, Eq. (9-11)	53.7 kW (71.5 hp)[a]	52.7 kW (70.4 hp)
Power input to the pump, brake power, Eq. (9-12)	64.4 kW (85.7 hp)[b]	64.5 kW (86.2 hp)
Motor power, Eq. (9-13)	73.2 kW (97.4 hp)[c]	73.3 kW (97.9 hp)

[a] $14.0\text{ m} \times 0.39\text{ m}^3/\text{s} \times 9.81\text{ kN/m}^3 = 53.7\text{ kW}\left(\dfrac{45.9\text{ ft} \times 6160\text{ gpm} \times 62.4\text{ lb/ft}^3}{7.48\text{ gal/ft}^3 \times 60\text{ s/min} \times 550\text{ ft·lb/hp·s}}\right) = 71.5\text{ hp}.$

[b] 53.7 kW/0.834 = 64.4 kW (71.5 hp/0.834 = 85.4 hp).

[c] 64.4 kW/0.88 = 73.2 kW (85.7 hp/0.88 = 97.4 hp). Provide 82-kW standard motor (110 hp).

pacity at maximum static head is preferred. In actual practice, the variable-drive units are adjusted to match the actual flow conditions.

Step C: Determination of Pump Efficiency and Power Input. The pump efficiencies corresponding to minimum and maximum wet well levels are shown in Figure 9-14. The pump efficiencies, power output of the pump, power input to the pump, and motor power are summarized in Table 9-10. Maximum motor power required is 73.3 kW. Provide a 82-kW (110-hp) motor having specified full-load motor efficiency of 88 percent (see specifications).

Step D: Determination of Available NPSH. The NPSH is calculated from Eq. (9-15). Calculation steps are given below:

$$\text{NPSH}_{\text{av}} = H_{\text{abso}} + H_s - h_L - H_{vp}$$

Assume H_{abso} at an altitude of 500 m above sea level is approximately 9.75 m (Appendix A, Table A-3). The barometric pressure is further reduced by 0.4 m of water due to storm activity. Therefore $H_{\text{abso}} = 9.35$ m of water.

$$H_s = \text{minimum suction head} = 1.22\text{ m (Figure 9-12)}$$

H_f = suction pipe losses to suction gauge at 0.42 m^3/s are calculated as follows[e] (see Tables 9-7 and 9-9 for additional information):

$$\text{Entrance (81 cm)} = \frac{0.04V^2}{2g} = \frac{0.04\,(0.81\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = 0.001\text{ m}$$

$$\text{One 90° elbow (61 cm)} = \frac{0.3V^2}{2g} = \frac{0.3\,(1.430\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = 0.031\text{ m}$$

$$\text{One gate valve (61 cm)} = \frac{0.19V^2}{2g} = \frac{0.19\,(1.430\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = 0.020\text{ m}$$

$$\text{One reducer (35.5 cm)} = \frac{0.25V^2}{2g} = \frac{0.25\,(4.223\text{ m/s})^2}{2 \times 9.81\text{ m/s}} = 0.227\text{ m}$$

$$\text{Velocity head (35.5 cm)} = \frac{V^2}{2g} = \frac{1\,(4.223\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = 0.909\text{ m}$$

$$\text{Total} = 1.188\text{ m}$$

H_v at operating temperature of 40°C = 55.324 mm of mercury (mm Hg) (Appendix A)

$$= \frac{55.324\text{ mm Hg}}{1000\text{ mm/m}} \times 13.58^{\text{f}}$$

$$= 0.75\text{ m of water}$$

$$\text{NPSH}_{av} = 9.35\text{ m} + 1.22\text{ m} - 1.19\text{ m} - 0.75\text{ m} = 8.63\text{ m}$$

NPSH required in normal pumping range from the manufacturer's data (Figure 9-14) = 5.5 m (18 ft). The available NPSH_{av} should exceed the required NPSH by at least 1 m.

Step E: Design Details of Pumping Station. Important design considerations for wet well, dry well, and pump arrangement were presented in Secs. 9-6-2 and 9-6-3. Layout of pumping station and piping arrangements are shown in Figures 9-11 and 9-12. Design details of wet well are provided below.

1. Select the pumping station layout.
   The pumping station in the Design Example has a wet well and a dry pit with a common wall. The minimum dimensions of different components of the wet well are based on the recommendations of the Hydraulic Institute.[1] The screened wastewater

[e]At minimum suction head the wet well elevation will be at a minimum, and only the lead pump will be operating at an average flow of 0.42 m^3/s. The velocities in different pipe sections are calculated from the pipe size. For example at suction bell, $V = \dfrac{0.42\text{ m}^3/\text{s}}{\pi/4 \times (0.81\text{ cm})^2} = 0.81\text{ m/s}$

[f]Specific gravity of mercury is 13.58.

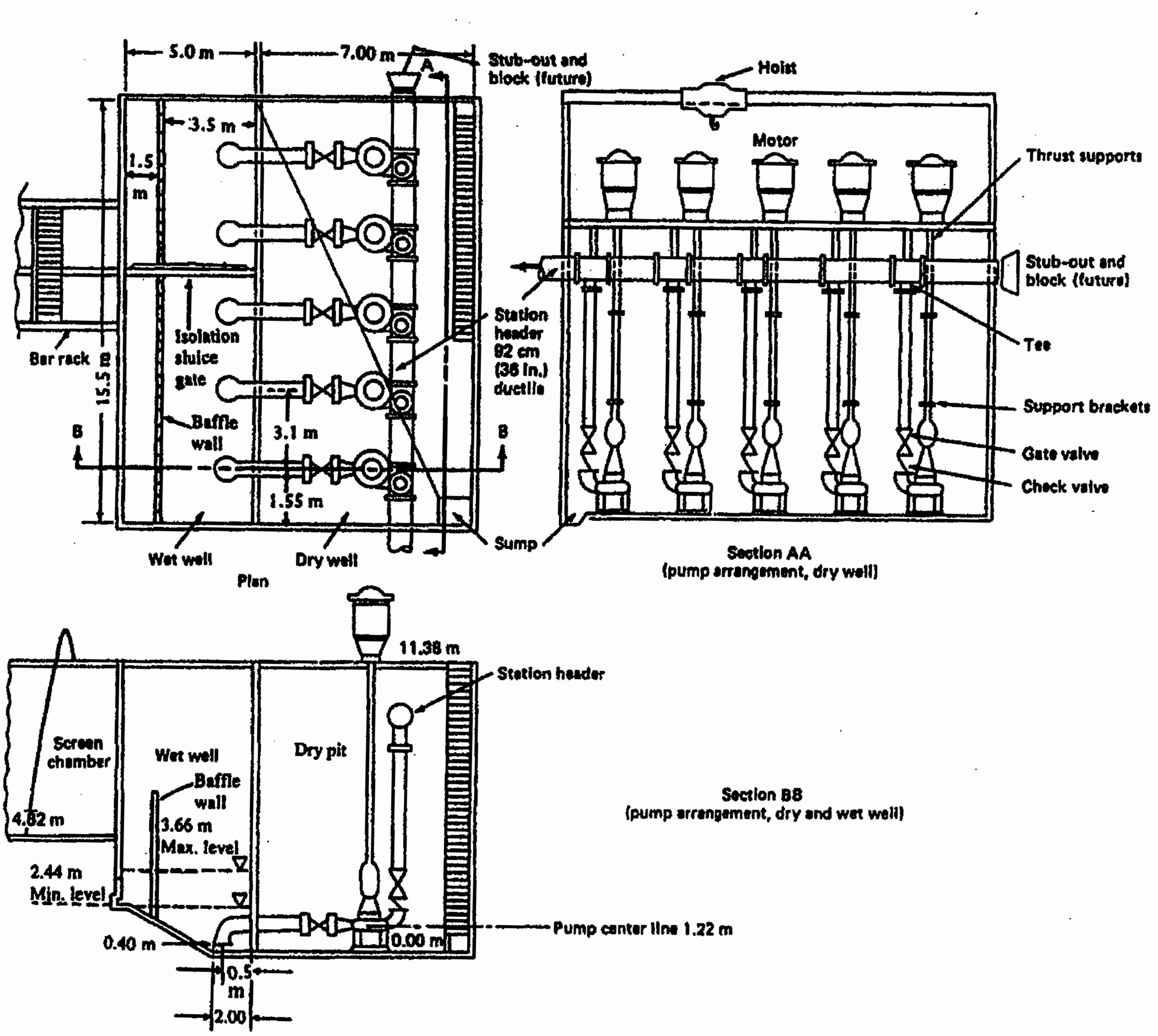


Figure 9-15 Layout of the Pumps and Piping Arrangements. All Elevations Are with Respect to Assumed Datum at Floor of the Dry Well or Wet Well.

drops into the wet well that is divided into two sections by an isolation gate. One section houses two suction bells, while the other section houses three suction bells. The purpose of the isolation gate is to remove one section out of service for cleaning and maintenance.

The wet well has a baffle wall 1.50 m from the free fall of the screen chamber. The baffle wall has numerous orifice openings, and it directs the flow towards the pumps and reduces the eddy currents in the suction well. The suction well floor is level from the wall common to the dry pit to a point 0.3 to 0.4 m (12 to 18 in.) beyond the outermost edge of the suction bell. The remainder section of the well is sloped upward to the opposite wall at a slope of 1 : 1. The pump station layout is shown in Figure 9-15.

2. Determine the suction well dimensions.

   A total of five identical pumps are arranged in parallel. One pump serves as a standby unit. The discharge of each pump at maximum station capacity when all four pumps are in operation is given in Table 9-9. This value is 0.39 m^3/s or 23.4 m^3/min (6160 gpm). For this discharge the minimum dimensions of components A, B, C, S, W, and

TABLE 9-11 Minimum and Actual Dimensions of Different Components of Suction Well

Components of Suction Well	Minimum Dimensions Obtained from Figure 9-9 (cm)	Actual Dimensions Provided in the Design (Figure 9-15) (m)
A[a]	305	3.50
B	50	0.50
C	37	0.40
S	100	2.04
W	126	3.10
W/2	63	1.55
Y	257	2.30

[a]A dimension is from the trash rack to the common wall between the wet well and dry pit. In this design the trash rack is not in the wet well, and wastewater flow is towards the suction well. Therefore, it is assumed that the dimension "A" is from the common wall to the baffle wall or $A = B + Y$ [see definition sketch in Figure 9-9(c)].

Y are obtained from Figure 9-9(a). These minimum dimensions and actual wet-well dimensions are summarized in Table 9-11. Actual dimensions are shown in Figure 9-15.

3. Design the baffle wall.

The most important design consideration for a baffle wall is the size of the orifice openings that will create a head loss at least equal to the upstream kinetic energy. The thickness of the baffle wall should be at least equal to the diameter of the orifice to fully develop a horizontal jet. Also, a large number of orifice openings should be provided.

The height of the baffle wall with openings is from the maximum level in the wet well to the bottom of the baffle wall that is supported over a 1:1 slope.

The maximum horizontal velocity in the wet well towards the pumps

$$= \frac{1.321\ \text{m}^3/\text{s}}{7.60\ \text{m (length of wet well)} \times 2.16\ \text{m}} \quad \text{(see Figure 9-16)}$$

$$= 0.08\ \text{m/s}$$

$$\text{The kinetic energy} = \frac{v^2}{2g}$$

$$= \frac{(0.08\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2}$$

$$= 0.0003\text{m}$$

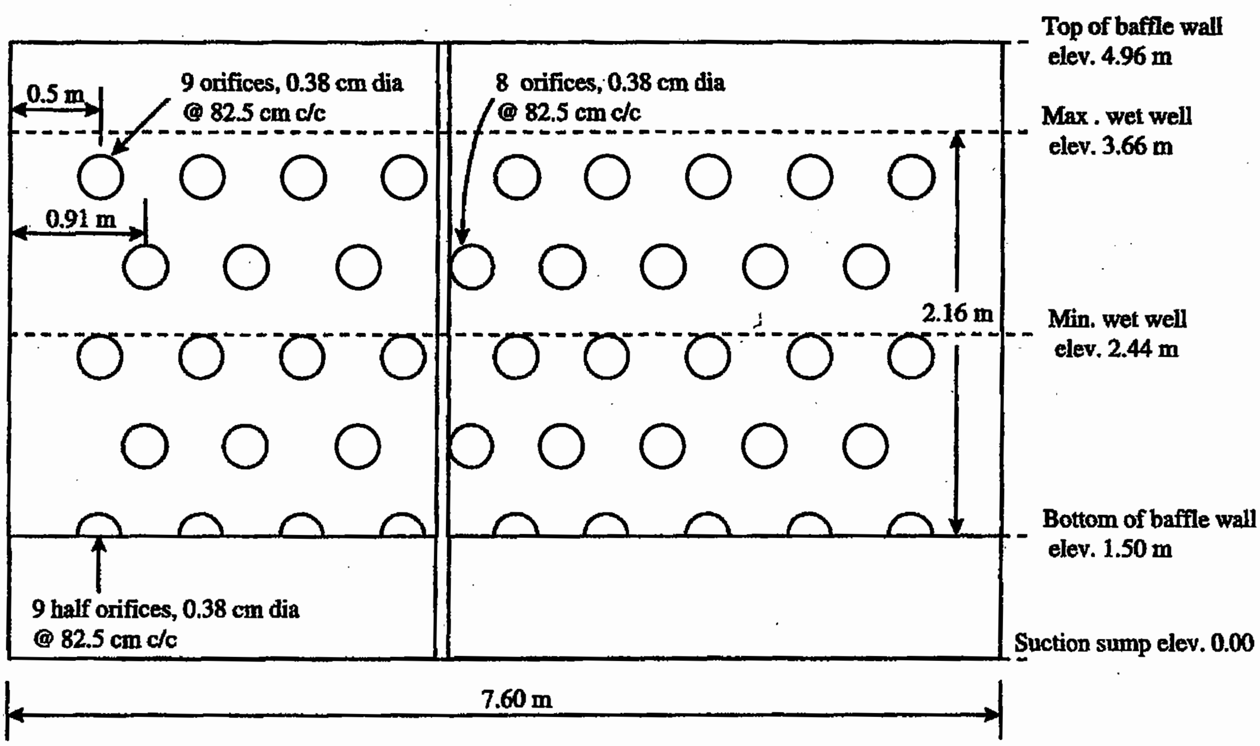


Figure 9-16 Orifice Arrangement in the Baffle Wall.

For this design use a velocity head of 0.015 m across the baffle wall. Higher head loss across the baffle wall will ensure proper flow distribution.

Using orifice equation [Eq. (8-3)] and assuming $C_d = 0.6$,

$$\text{Velocity through orifice openings } v = C_d\,(h_L \times 2g)^{1/2}$$
$$= 0.6\,(0.015\text{ m} \times 2 \times 9.81\text{ m/s}^2)^{1/2}$$
$$= 0.33\text{ m/s}$$

Area of the orifice openings in the baffle wall

$$A = \frac{1.321\text{ m}^3/\text{s}}{0.33\text{ m/s}}$$
$$= 4.00\text{ m}^2$$

Provide 38-cm (15-in.) diameter openings in the baffle wall

$$\text{Number of openings} = \frac{4.00\text{ m}^2}{\pi/4 \times (0.38)^2} = 35.3$$

Provide 34 circular and 9 half-circle orifice openings in the baffle wall. This gives a total of 38.5 openings. The design details are shown in Figure 9-16.

4. Check pump cycle time.
The wet well volume should be large enough to provide required storage to allow sufficient time between the successive start or stop cycles. The time between starts is a

function of the pumping rate and the flow entering the station. Eq. (9-18) is used to calculate the minimum time of one pumping cycle:

$$t = \frac{4V}{Q} \tag{9-18}$$

where

t = pumping cycle time, min
V = volume between maximum and minimum wet well level elevation, m^3
Q = pumping rate of one pump, m^3/min

From the dimensions of the wet well and discharge of each pump at maximum station capacity, the pumping cycle time is calculated.

$$\text{Wet-well volume (Figure 9-15)} = 15.5 \text{m} \times 5.0 \text{ m} \times 1.22 \text{ m}$$
$$= 94.55 \text{ m}^3$$
$$\text{Pumping rate (Table 9-9)} = 0.39 \text{ m}^3/\text{s} = 23.4 \text{ m}^3/\text{min}$$
$$t = \frac{4 \times 94.55 \text{ m}^3}{23.4 \text{ m}^3/\text{min}} = 16.2 \text{ min}$$

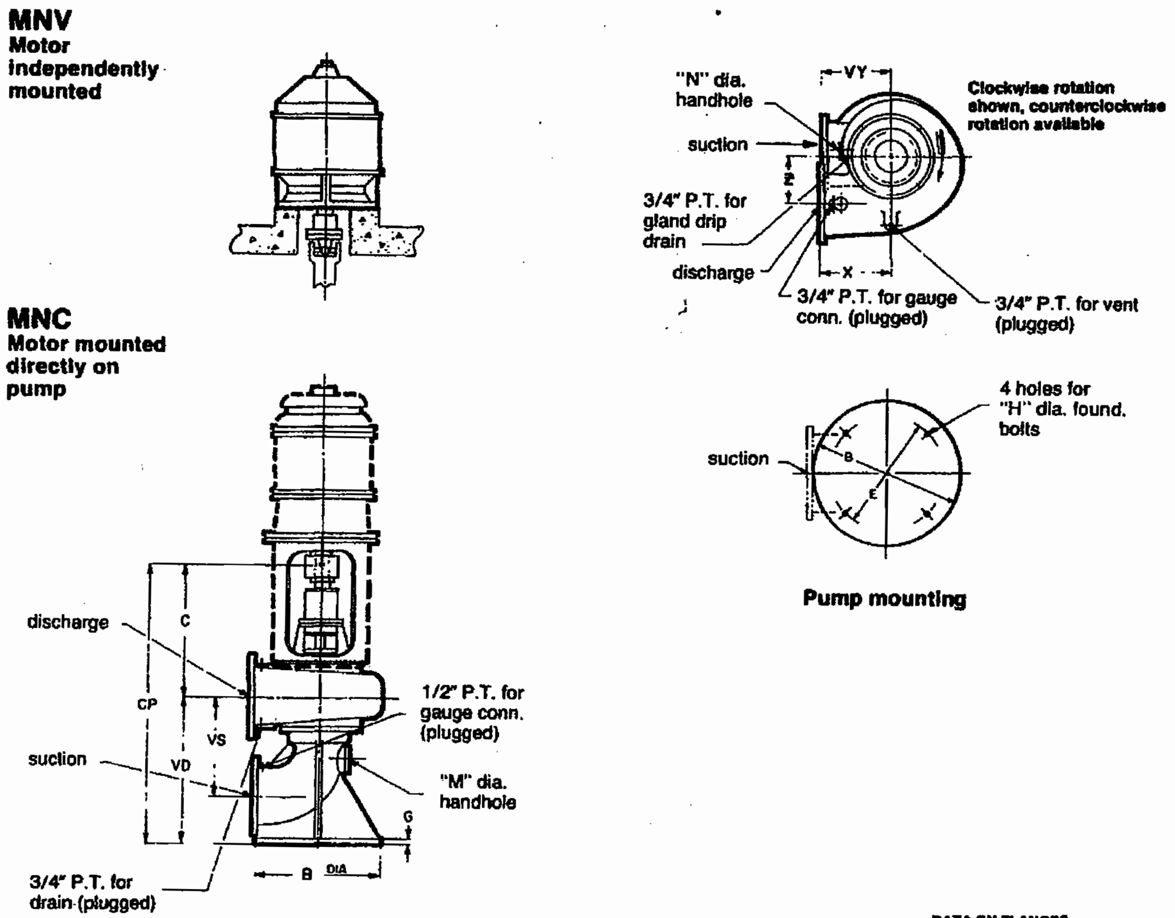
A pumping cycle time of 16.2 min gives approximately four cycles per hour. Many design standards specify a maximum pumping cycle per hour of a pumping station. If this condition is not met, it is recommended that the designer should consider providing deeper wet well. In this design a 1.22-m difference in the maximum and minimum wet-well elevations is not sufficient. A differential elevation of 2.5 to 4 m is considered more desirable for the following reasons: (a) a longer cycle time will result in deeper wet well, which would reduce the quick pump start and stop cycle, and (b) longer differential operating elevations will be available between the control signals from the bubbler tube (Table 9-12).

5. Calculate the air circulation rate.
The air circulation is necessary to dissipate the heat generated. The air circulation rate is calculated from Eqs. (9-16) and (9-17). Motor power factor $U = 0.75$ (see motor specifications, Sec. 9-11-2) $H = 82 \text{ kW} \times 1.0 \times 0.75 \left(\frac{1 - 0.88}{0.88}\right) = 8.4 \text{ kW}$.
Assume maximum indoor and outdoor temperature of 48°C (118.4°F) and 37.8°C (100°F), respectively.

$$Q = \frac{8.4 \text{ kW}}{1.02 \text{ kW} \cdot \text{s/kg °C} \times 1.2 \text{ kg/m}^3 (48 - 37.8) \text{ °C}}$$
$$= 0.67 \text{ m}^3/\text{s per pump}$$

The maximum number of pumps that can operate at any time is four. It is expected that a four-pump operation is only for a very short time. Therefore, design an intermittent air circulation system for a maximum air exhaust rate of 2.7 m^3/s.

6. Provide floor drains.
Floor drains, sump, and pump must be provided to remove all condensates and drips



PUMP SIZE	DISCH DIA.	SUCT DIA.	B	C	CP	E	G	H	M	N	VD	VS	VY	X	Z	DATA ON FLANGES: DIA. OF FLANGE	FLANGE THICKN	SIZE OF BOLT	BOLT CIRCLE	NO. OF BOLTS
14MNC-24 14MNV-24	14	14	30.00	38.62	76.12	27.50	1.50	1.00	5.00	6.00	37.50	25.00	16.00	21.00	17.50	21.00	1.38	1.00	18.75	12

Figure 9-17 Installation Details and Dimensions of the Selected Pump in the Design Example (courtesy Worthington Pump Corporation).

from the pump and piping. A maximum water level alarm in the sump must be provided in the dry pit.

7. Develop pump installation details.
   Manufacturer's details and dimensions of a selected pump are shown in Figure 9-17. Pump installation details are shown in Figure 9-18. Details of pumps, motor, and air bubbler control system are provided in the specifications.

9-10 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT PUMPING STATION

The designed pumping station has a standby pump to maintain continuous service when one unit is temporarily out of service. Also, variable-speed pumps have been provided to match the incoming flows and reduce the plant surges. Explosion-proof equipment, ventilation of equipment in dry well, separation of wet well, isolation gate in wet well, bar screen ahead of wet well for protection of pumps, and manual and automatic control systems for the pumps have been provided to achieve trouble-free operation and main-

(a)

(b)

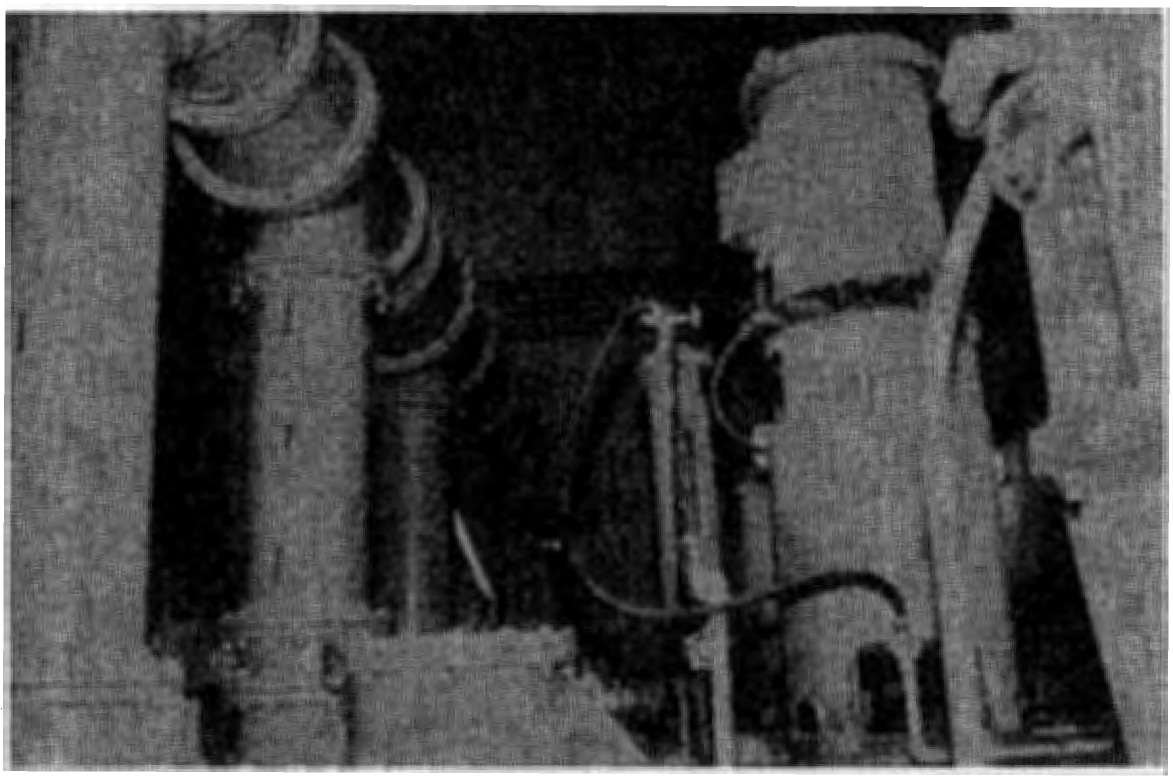


Figure 9-18 Typical Pump Installation Details: (a) vertical elbow suction pumps with motor directly mounted on the top and (b) vertical pumps discharging into a common header. The Motor Is Directly Mounted on the Top.

tenance. Common operation and maintenance problems that may develop at pumping stations and procedures to correct them are summarized below.[10,13]

9-10-1 Equipment Maintenance

The following routine maintenance steps should be taken to avoid major breakdowns:

1. Daily observations of pump operations should be conducted and a log should be kep of these inspections.
2. Inspect stuffing boxes for free movement of glands and examine for leaks on weekl basis.
3. Check temperature of the pump casing by placing a hand on the unit. If it is hot, checl stuffing box and bearings. Investigate further for possible problems.
4. Lubricate the motors at least once each week, taking care to avoid overlubrication o the top bearings (overflow onto the commutator or armature windings may be damaging).
5. Lubricate the thrust bearings of each pump every three to six months, depending o frequency of operation.
6. Lubricate yokes and slip joints of flexible shafting at least once a month in accordanc with the manufacturer's recommendations.
7. Listen and smell for unusual noises and odors.
8. Completely overhaul pumps in accordance with manufacturer's instructions as determined by pump operation conditions.
9. Standby unit shall be exercised on regular basis for uniform wear.

9-10-2 Common Operational Problems and Suggested Solutions

There are a variety of common operating problems that may occur periodically at pumping stations. Some of the common problems and suggested corrective measures are liste below:

1. Surging of the plant influent is indicated by flooded weirs, flow meter records showing intermittent high and low peak flows, and drop in treatment plant efficienc Surging during dry weather flow may be due to malfunctioning pump controls, insufficient hydraulic capacity of the plant, or illegal connections to the system. Th solution to the problem is to check and adjust the pump controls, install a surge tank and remove illegal connections. Surging during wet weather is an indication of excessive infiltration and inflow. This problem may be overcome by checking and repairing manhole seals and covers, broken sewer lines, and illegal roof connection
2. Improper liquid levels in the wet well indicate coating on liquid level probe hangups in the level detectors, leaks in the floats, and fouling of bubbler control. Th problem can be overcome by cleaning and repairing probes, level detectors, float and bubbler.
3. Accumulation of solids or scum in the wet well may be due to scum blanket in th wet well and improper operation of level-sensing equipment. The extent of the prob

lem can be determined by sounding the wet well with a pole to determine the solids level and measurement of wet well draw-down during pumping cycle. The problem may be corrected by starting the pump manually and lowering the liquid level in the wet well to the lowest possible level without breaking the suction. The scum should then be broken up with a high-pressure water hose.

4. Odor problems in the wet well may arise from long storage in the wet well or flat grade in the collection system. The severity of the problem is indicated by hydrogen sulfide emission, corrosion of iron work and concrete, and black color observed in liquid or solids. The problem may be corrected by proper operation of lift station, addition of hydrogen peroxide or chlorine solution in the wet well or collection lines, installation of air diffusers in the wet well, or installation of blower and gas scrubber for oxidation of gases exhausting to the atmosphere.
5. Pump may not start due to blown fuses, defective control, or defective motor. Check and correct the following conditions: (a) fuses and their ratings, (b) corroded or shorted contact switches, (c) loose or broken terminal connections, (d) automatic control mechanism, (e) switches that are not set for operation, (f) contacts of the controls that may be dirty or arching, (g) wiring that may be short-circuited, and (h) a motor that may be shorted or burnt out.
6. Pump may not run or circuit breakers may not reset because of a clogged pump suction, discharge pipes, or closed valve.
7. Pump may be running but with reduced discharge due to the following reasons: (a) pump not primed or pump that is air-bound, (b) clogged impeller because of grease or other obstructions, (c) speed of motor too low because of improper wiring or defects, (d) discharge head too high, (e) suction lift too high, (f) discharge or suction lines clogged, (g) air leaks in suction line or in packing box, (h) valve partially closed, (i) incorrect impeller adjustment or damaged impeller, (j) worn-out or defective packing, couplings, or wearing rings.
8. A high power bill may have many causes. These include (a) clogged pump, (b) misaligned belt drive, (c) speed of rotation too high, (d) operating head lower than rating for which the pump was designed, (e) check valves open or force main draining back into the wet well, (f) pump shaft bent, (g) rotating elements binding, (h) packing boxes too tight, (i) wearing rings worn or binding, and (j) impeller rubbing.
9. Excessive wear or damage to the pumps may be caused by grit or grease accumulations in the wet well.
10. Noisy pump may be caused by the following: (a) cavitation, (b) pump not completely primed, (c) inlet clogged, (d) inlet not submerged, (e) pump not lubricated properly, (f) worn out bearings or impellers, (g) insecure foundation, and (h) defects in the pumps.

9-11 SPECIFICATIONS

Brief specifications of various components of pumping stations are presented in this section. These specifications are written as an integral part of the design, in order to describe many components that could not be fully covered in the Design Example. Many minor details have been omitted; therefore, these specifications should be used only as a tool to fully understand the design and as a guide in developing the specifications for a real design. The design engineer must work closely with the equipment manufacturers to develop the detailed specifications for the selected equipment.

9-11-1 General

Contractor shall furnish and install five vertical shaft, dry pit, mixed flow, centrifugal pumping units with variable-speed drive as detailed in the drawings. These pumps shall be identical and arranged in parallel to discharge into a 92-cm (36-in.) common header. All pumps shall be installed complete with all appurtenances necessary for proper operation of the equipment.

9-11-2 Pumps

Casing. The pump casing shall be of one-piece volute with integral discharge flange. The casing shall be made of close-grained cast iron of sufficient strength, weight, and thickness to provide accurate alignment and prevent deflection. The casing shall be designed to permit removal of the rotating elements without disturbing the suction or discharge connections and be provided with a cleanout to permit inspection and cleaning of the pump interior. The casing shall be hydrostatically tested to one and one-quarter times the maximum shutoff pressure and provided with 2-cm vent, drain, and gauge connections.

Suction Head. The suction head shall be designed to provide equal flow distribution to the impeller eye. It shall be of the same material as the casing and shall have flanged connection, a handhole with removable cover, and a 2-cm gauge tap connection.

Impeller. The impeller shall be a single-stage, end-suction, mixed-flow, enclosed, nonclog type with a minimum number of vanes, designed to pass 8-cm solids. The impellers shall be made of specified material of given strength, machined and polished, and statically and dynamically balanced prior to assembly. The impeller shall be secured to the shaft with a shear key such that clogging of the impeller will not damage the pump. Also, the pump shall be provided with reverse rotation protection.

Wearing Rings. Removable wearing rings of specified material shall be furnished on the impeller and suction head. They shall be securely fastened to prevent any relative rotation and designed to compensate for a minimum of 7-mm wear.

Pump Shaft, Sleeve, and Stuffing Box. The pump shaft shall be of sufficient size and strength to transmit the full power with a liberal safety factor. The shaft shall be of specified material, machined over the entire length, protected from wear in the stuffing box by shaft sleeve sealed to prevent leakage between the sleeve and the shaft.

Bearings. The bearings shall be of specified material, arranged to eliminate all radial play, and designed for a minimum life of 100,000 h. The bearings shall be sealed to prevent entrance of contaminants, grease lubricated, and provided with tapped openings for addition of lubricant and draining. The bearing frame shall be arranged to provide for the axial adjustment of the wearing rings by the use of jacking screws and removable shims between the bearing frame and the stuffing box head.

Shop Testing. Each pump shall be fully tested on water in the manufacturer's shop in accordance with the standards of the Hydraulic Institute to determine compliance with the rated conditions.[1] Certified curves shall be submitted for approval prior to shipment.

Variable-Speed Drives. The variable-speed drives shall be of an eddy current type consisting of a stationary frame, a constant-speed and an adjustable-speed magnetic member, bearings, bearing brackets, housing, and such other components as are necessary to provide complete operating unit.

Motors. Motors shall be weatherproof, with rodent screens suitable for use outdoors. Motor windings shall be insulated, and motors shall operate continuously at a rated voltage and frequency with a temperature rise not to exceed 40°C above ambient when operating at 115 percent of the rated power. Motors shall be rated for a minimum of 82 kW (110 hp) and have a full-load efficiency of not less than 88 percent. All motors shall have a full-load power factor of not less than 75 percent. The locked rotor torque shall be not less than 100 percent of full-load torque. The breakdown torque shall be not less than 200 percent of full-load torque. All motor bearings shall be of the antifriction type suited for a 10-year minimum life.

Liquid Level Bubbler Control. A liquid level bubbler control designed to operate with the adjustable frequency power supply shall be supplied. The bubbler control shall include compressor and air tank with gauge of specified rated capacity, drain valve, bleed valve, pressure maintenance switch, pressure regulator gauge, air filter with drain, air flow regulator of suitable range, level, gauge, electronic pressure transducer, pressure relief valve, needle valve, power on–off switch, power-on indicating light, and pneumatic piping and fitting to connect to the bubbler tube.

The pressure signal from the bubbler shall be provided on an analog output directly proportional to the wet well level above the bottom of the bubbler tube. Contacts to indicate a high and low wet well level shall be provided. The adjustable speed drive system shall be designed to stop the pumps within the wet well parameters given in Table 9-12. The liquid elevations to start the pumps are generally higher than those to stop the pumps. This may be necessary to maximize wet well storage and to stop and start cycle time. Wet well dimensions shall have a length of 15.0 m, width of 5.0 m, and a total depth of 11.38 m. Influent flow conditions are minimum flow = 0.152 m^3/s and maximum flow = 1.321 m^3/s.

Certified Drawings. Certified prints of the proposed equipment shall be furnished for approval. These shall include a combined electrical drawing showing pumps, motors, driving equipment and coupling, a pump selection drawing with list of materials, performance curves, and liquid level bubbler control system. A written description of sequence of operation of pump drive system under normal automatic mode, normal manual mode, emergency-automatic mode, and manual operation mode shall be provided.

TABLE 9-12 *Operating Conditions of the Pumping Station*

Condition	Elev. above Floor of the Wet Well (m)	3rd Lag Pump			2nd Lag Pump			1st Lag Pump			Lead Pump		
		Max. Speed[a]	Min. Speed[b]	Stop	Max. Speed	Min. Speed	Stop	Max. Speed	Min. Speed	Stop	Max. Speed	Min. Speed	Stop
End of bubbler tube	4.44	X			X			X			X		
High-level alarm	4.16	X			X			X			X		
High level in the wet well	3.66	X			X			X			X		
3rd lag pump minimum speed	3.50		X		X			X			X		
3rd lag pump stopped	3.35			X	X			X			X		
2nd lag pump minimum speed	3.20			X		X		X			X		
2nd lag pump stopped	3.05			X			X	X			X		
1st lag pump minimum speed	2.89			X			X		X		X		
1st lag pump stopped	2.74			X			X			X	X		
Lead pump minimum speed	2.59			X			X			X		X	
Minimum level in the wet	2.44			X			X			X		X	
Low-level alarm and shut down	2.16			X			X			X			X
End of bubbler tube	1.88												

[a]Maximum speed 705 rpm (Figure 9-14).
[b]Variable-speed drive pumps generally have minimum speed at half the maximum speed (approximately 350 rpm).
Note: The operating elevations to start the pumps are different than those to stop the pumps given in this table.

9-11-3 Painting

All ferrous metal surfaces shall receive a protective coating of rust-inhibitive primer and finished coat of approved paint.

9-12 PROBLEMS AND DISCUSSION TOPICS

9-1 A centrifugal pump is operating at a speed of 1200 rpm and discharges 2.5 m^3/min. The total head of 120 kPa is measured on a pressure gauge located in the discharge pipe. The power required is 7.0 kW. Calculate (a) the pump efficiency and (b) the discharge, head, and power if the pump speed is changed to 1800 rpm.

9-2 A pump with a Francis-Vane impeller is selected for operation at a best efficiency of 87 percent. The operating capacity is 0.2 m^3/s against a total head of 16 m. Determine the operating speed and the specific speed.

9-3 A centrifugal pump is operating at an elevation of 1829 m above sea level. The pump requires 30 kPa net positive suction head (NPSH) when delivering the water at 20°C. What is the allowable suction lift of the pump if entrance and friction losses in the suction lines are 15 kPa? Assume the reduction in barometric pressure caused by weather change is 4.1 kPa.

9-4 Water is pumped from reservoir A to B. The water surface elevations in reservoirs A and B are 300 m and 318 m, respectively. The suction line is 9.5 cm, which is short enough that the head losses may be neglected. The force main is 400 m long and 79 cm in diameter. There are three 90° and two 45° bends. Calculate the annual power bill for pumping 0.30 m^3/s. Unit power cost is 5 cents per kW·h. Assume $C = 100$, and combined pump and motor efficiency is 78 percent.

9-5 The data for characteristic curves of a variable-speed pump supplied by a manufacturer are given below. Draw the modified curve and H–Q curve for two pumps operating in parallel. Also draw the system head curves. Use the following data:

Piping, valves, fittings, and specials from wetwell to common header
Pipe diameter = 46 cm, and straight length is small.
2 gate valves 46 cm, K = 0.19
1 elbow 90° 46 cm, K = 0.30
2 reducers 31 cm, K = 0.25
1 check valve 46 cm, K = 2.50
1 tee 46 cm, K = 1.80
1 entrance suction bell 62 cm, K = 0.04

Piping, valves, fittings and specials from common header to the point of discharge are given below:

The venturi meter has throat diameter of 38 cm
Pipe diameter = 76 cm, and straight length is 65 m. $C = 100$
2 elbows 90°, K = 0.3
2 elbows 45°, K = 0.2
1 Venturi meter, K = 0.14
1 plug valve K = 1.00
1 Wye branch K = 0.38

The pump volute elevation is 300.00 m. Maximum and minimum water surface elevations in the wet well are 303.00 and 301.00 m. The water surface elevation in the discharge unit is 315.00 m. The maximum and minimum station discharges are 0.65 and 0.20 m^3/s. Calculate (a) operating head and capacity of the pump station when both pumps are operating, (b) operating head and discharge of each pump, and (c) corresponding head and discharge given on manufacturer's Head-Q curve.

Pump Characteristic Curve Data Supplied by the Manufacturer

Head, m	Discharge, m^3/s
20	0
15	0.30
10	0.45
5	0.50

9-6 A centrifugal pump delivers 0.15 m^3/s at 30 m TDH when operating at a speed of 2000 rpm. Determine the discharge, total dynamic head, and power input if the impeller diameter is reduced from 30 cm to 24 cm.

9-7 Determine the available NPSH of a pump that delivers water at 20°C. The pump is operating at an elevation of 40 m above sea level. The water surface elevation in the wet well is 38 m, and the water surface elevation in the discharge unit is 60 m. The capacity of a single pump is 0.2 m^3/s. The details of piping, valves, fittings, and specials from wet well to the common pump header are

C	= 100
Pipe diameter	= 25 cm
Length of pipe	= 10 m
Two 45° elbows, 25 cm, K	= 0.2
Two 90° elbows, 25 cm, K	= 0.25
One entrance suction bell, 46 cm, K	= 0.04
One gate valve, 25 cm, K	= 0.1

9-8 Different types of pumps are listed below. Write various applications of each type of pump: (a) screw, (b) rotary, (c) centrifugal, (d) diaphragm, (e) air-lift, (f) plunger, and (g) pneumatic ejector.

9-9 Define force main, wet well, dry well, casing, stuffing box, TDH, and specific speed.

9-10 The modified head-capacity curve and system-head capacity curve for minimum wet well level are given below. Using these data, determine the operating heads and capacities at minimum and maximum wet well levels for one pump, two pumps in parallel, and two pumps in series. The difference between maximum and minimum wet-well elevations is 2.0 m.

Head-Capacity Curve		System-Head Capacity Curve	
Head, m	Capacity, m^3/s	Head, m	Capacity, m^3/s
40	0	30	0
39	0.02	32	0.02
36	0.04	34	0.04
29	0.06	37	0.06
20	0.08	40	0.08
8	0.10	44	0.10

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10

Flow Measurement

10-1 INTRODUCTION

Flow measurement at wastewater treatment facilities is essential for plant operation and process control. The average and diurnal variations in flow are needed to determine how much chemicals to add, how much air to supply into the aeration basins, and how much sludge to return into the biological reactors. Some additional reasons for keeping flow measurement records are

1. Most of the regulatory agencies require that the wastewater treatment plants maintain daily flow records.
2. Flow records are invaluable for future reference, particularly when plant expansion is needed.
3. Substantial increase in daily dry weather flow may be caused by population growth, infiltration, or industrial waste discharge into the sewers.
4. Increase in flow during wet weather is a measure of infiltration/inflow.

In this chapter various types of flow-measuring devices are discussed. In addition, step-by-step design procedure, details, and specifications for a Venturi meter are presented in the Design Example. Design procedure for the Parshall flume is covered in Chapter 14.

10-2 LOCATION OF FLOW MEASUREMENT DEVICES

There are many locations at a wastewater treatment facility where a suitable flow-measuring device can be installed. These locations may include (1) within an interceptor or manhole; (2) at the head of the plant; (3) downstream of bar screen, grit channel, or primary sedimentation facility; (4) in the force main of pumping station; or (5) before the outfall. Each location has advantages and disadvantages and will serve some specific need. Often, more than one flow measurement device may be necessary at different locations. In Table 10-1, the operating characteristics of flow-measuring devices are compared when installed at various locations at a wastewater treatment facility.

TABLE 10-1 Comparison of Operational Characteristics of Flow Measurement Devices Installed at Different Locations

Alternative Location of Flow Measurement Device	Sensitive to Fluctuations in Flow	Measurement Represents the Average Flow Treated	Affected by Debris	Affected by Silt or Other Settleable Solids	Measurement Useful for Plant Operations	Measurement Useful for Effluent Receiving Source
Within intercepting sewer or manhole	yes	no	yes	yes	yes	no
At the head of the plant	yes	no	yes	yes	yes	no
Below bar screen	yes	no	no	yes	yes	no
Below grit removal or sedimentation facility	no	no	no	no	yes	no
Before outfall	no	yes	no	no	no	yes

10-3 FLOW MEASUREMENT METHODS AND DEVICES

10-3-1 Types

A wide variety of flow measurement methods and devices are available that can be used for the determination of wastewater discharges. The selection of the proper measuring method or device will depend on such factors as cost, type and accessibility of the conduit, hydraulic head available, and type and characteristics of liquid streams. In general, flow measurement systems fall into two categories: (1) pressure pipes and (2) open channel flow. However, some systems are applicable to both. For wastewater flow measurement, a system in which the rate of discharge is related to one easily measurable variable is preferred. In such systems, the direct discharge values can be obtained from the rating curves developed for the system. Table 10-2 provides a list of many different devices applicable to fluid flow measurement. Devices that are commonly used for flow measurement of municipal and industrial waste streams are indicated in this table. A brief description of devices suitable for wastewater applications is given below. Detailed discussions on these devices may be obtained in several excellent references.[1-4] Several commonly used flow measurement devices are illustrated in Figures 10-1 and 10-2.

Venturi Tube. The Venturi tube flow meter utilizes the principle of differential pressure. A commercially available Venturi meter consists of a converging section (called approach), a throat, and a diverging recovery section. Due to converging section, the velocity at the throat is increased; as a result, the piezometric head is decreased. The difference in piezometric head between the throat and the beginning of the approach is measured. The difference in these two heads is analyzed by electrical or electromechanical instruments [Figure 10-1(a)]. The flow is a function of the difference in head. Venturi meters provide a high degree of accuracy but are subject to plugging. They are relatively high in cost, and the pressure probes require periodic maintenance. In addition to full-body flanged end Venturis, various types of insert Venturis are frequently utilized to reduce costs. Insert Venturis are manufactured utilizing a variety of materials. An insert Venturi is shown in Figure 10-1(b).

Flow Nozzle. The flow nozzle meters are similar to a Venturi meter and work on the same principle. This device is actually a Venturi meter without the recovery section. The nozzle is constructed inside a pipe, where the approach is made much shorter than in a Venturi meter. The pressure is measured upstream and downstream of the nozzle. The flow nozzle provides a much less expensive installation, but it has a higher head loss than the Venturi meter. Flow nozzle meters are also commercially available [Figure 10-1(c)].

Orifice Meter. The orifice meter (also called orifice plate) is commercially available and operates in the same manner as the Venturi meter and flow nozzle meter. The orifice acts as an obstacle placed in the path of flow in a pipe with no approach or recovery. Because of its ease of installation and fabrication, the orifice meter is the least expensive of all the differential pressure devices. Unfortunately, this device is also the least efficient, and it clogs easily [Figure 10-1(d)].

TABLE 10-2 Types of Flow Measurement Devices Available for Determining Liquid Discharges

Flow Measurement Devices	Principle of Flow Measurement
1. For pressure pipes	
a. Venturi meter[a]	The differential pressure is measured.
b. Flow nozzle meter[a]	The differential pressure is measured.
c. Orifice meter[a]	The differential pressure is measured.
d. Pitot tube	The differential pressure is measured.
e. Electromagnetic meter[a]	Magnetic field is induced, and voltage is measured.
f. Rotameter	The rise of float in a tapered tube is measured.
g. Turbine meter[a]	A velocity driven rotational element (turbine, vane, wheel) is used.
h. Ultrasonic velocity[a]	The ultrasonic transducers send and receive ultrasonic pressure pulses.
i. Ultrasonic Doppler[a]	The transducers transmit beams that are reflected to a receiver by suspended solids or gas bubbles.
j. Elbow meter	The differential pressure is measured around a bend.
2. For open channels	
a. Flumes (Parshall, Palmer-Bowlus)[a]	Critical depth is measured at the flume.
b. Weirs[a]	Head is measured over a weir.
c. Current meter	Rotational element is used to measure velocity.
d. Pitot tube	The differential pressure is measured.
e. Depth measurement[a]	Float is used to obtain the depth of flow.
f. Sonic level meter[a]	The transducer emit and receive the beam reflected from the surface.
3. Computing flow from freely discharging pipes.	
a. Pipes flowing full	
i. Nozzles and orifices	The water jet data is recorded.
ii. Vertical open-end flow	The vertical height of water jet is recorded.
b. Pipe partly flowing full	
i. Horizontal sloped open-end pipe	The dimensions of freefalling water jet are obtained.
ii. Open flow nozzle[a] (Kennison nozzle or California pipe method)	The depth of flow at freefalling end is determined.
4. Miscellaneous method	
a. Dilution method	A constant flow of a dye tracer is used.
b. Bucket and stopwatch	A calibrated bucket is used, and time to fill is noted.
c. Measuring level change in tank	Change in level in a given time is obtained.
d. Calculation from water meter readings	Total water meter readings over a given time period give average wastewater flow.
e. Pumping rate	Constant pumping rate and pumping duration

[a]Flow-measuring devices commonly used at wastewater treatment facilities.

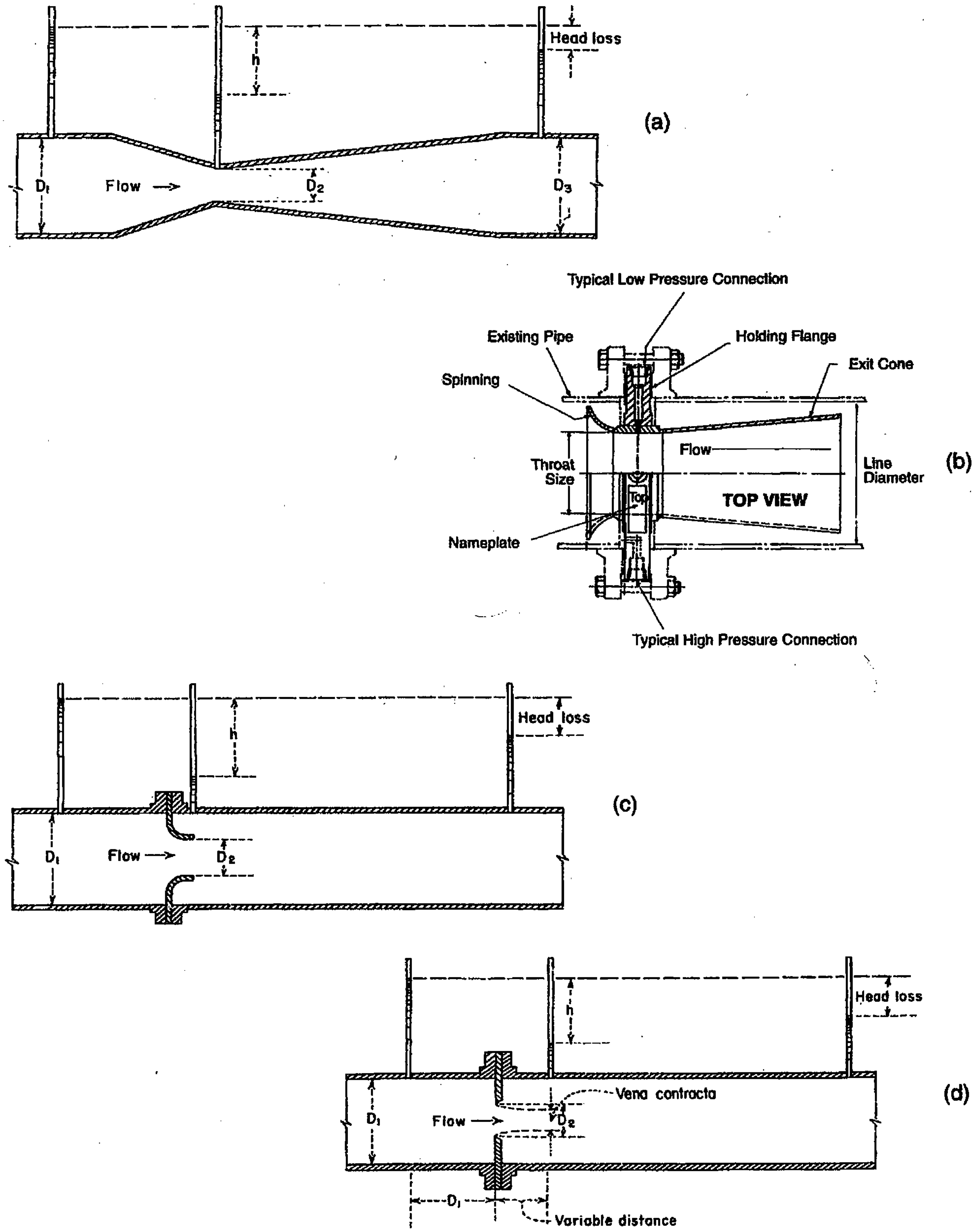
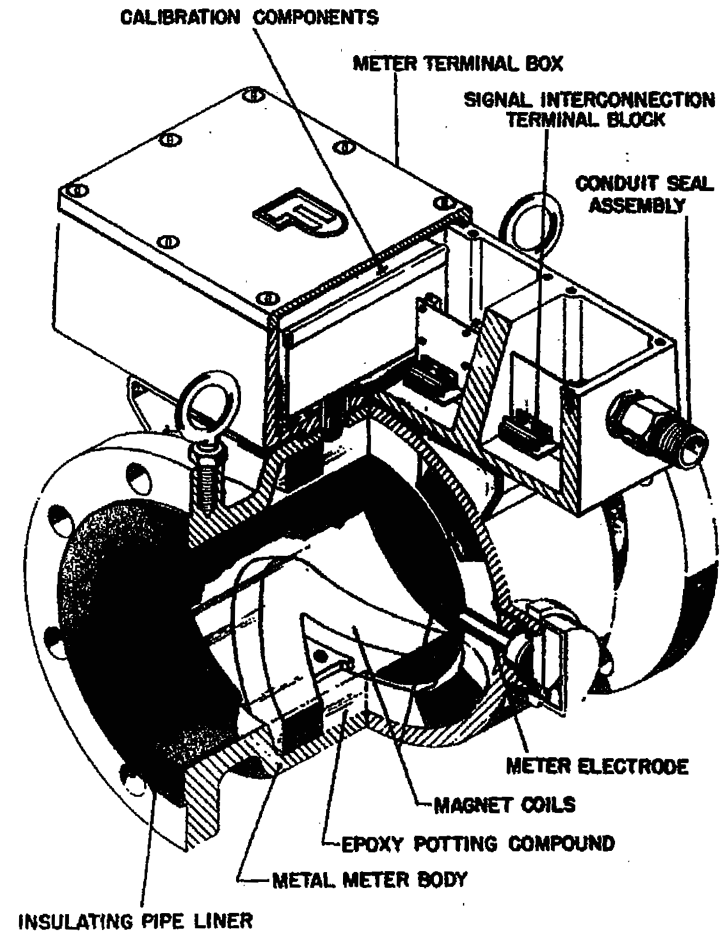


Figure 10-1 Various Flow-Measuring Devices Applicable to Pressure Pipes: (a) Venturi meter (from Refs. 2 and 3), (b) insert Venturi meter (courtesy Badger Meter, Inc.), (c) flow nozzle, and (d) orifice meter (from Refs. 2 and 3).

(a)



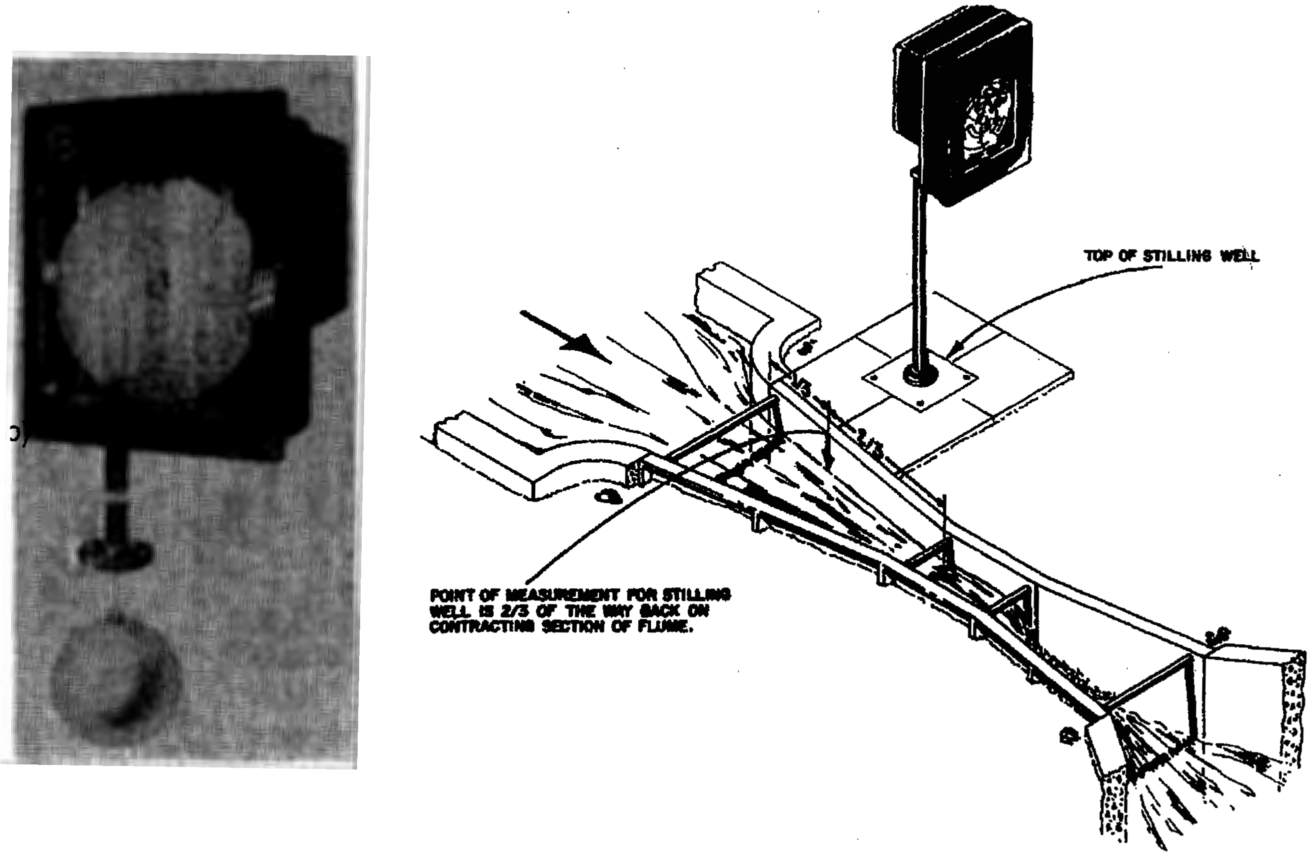


Figure 10-2 Various Flow-Measuring Devices for Pressure Pipes and Open Channels and Typical Installations: (a) magnetic flow meter (courtesy Baily-Fischer & Porter), (b) float actuated liquid level recorder and typical installation with Parshall flume (courtesy Baily-Fischer & Porter).

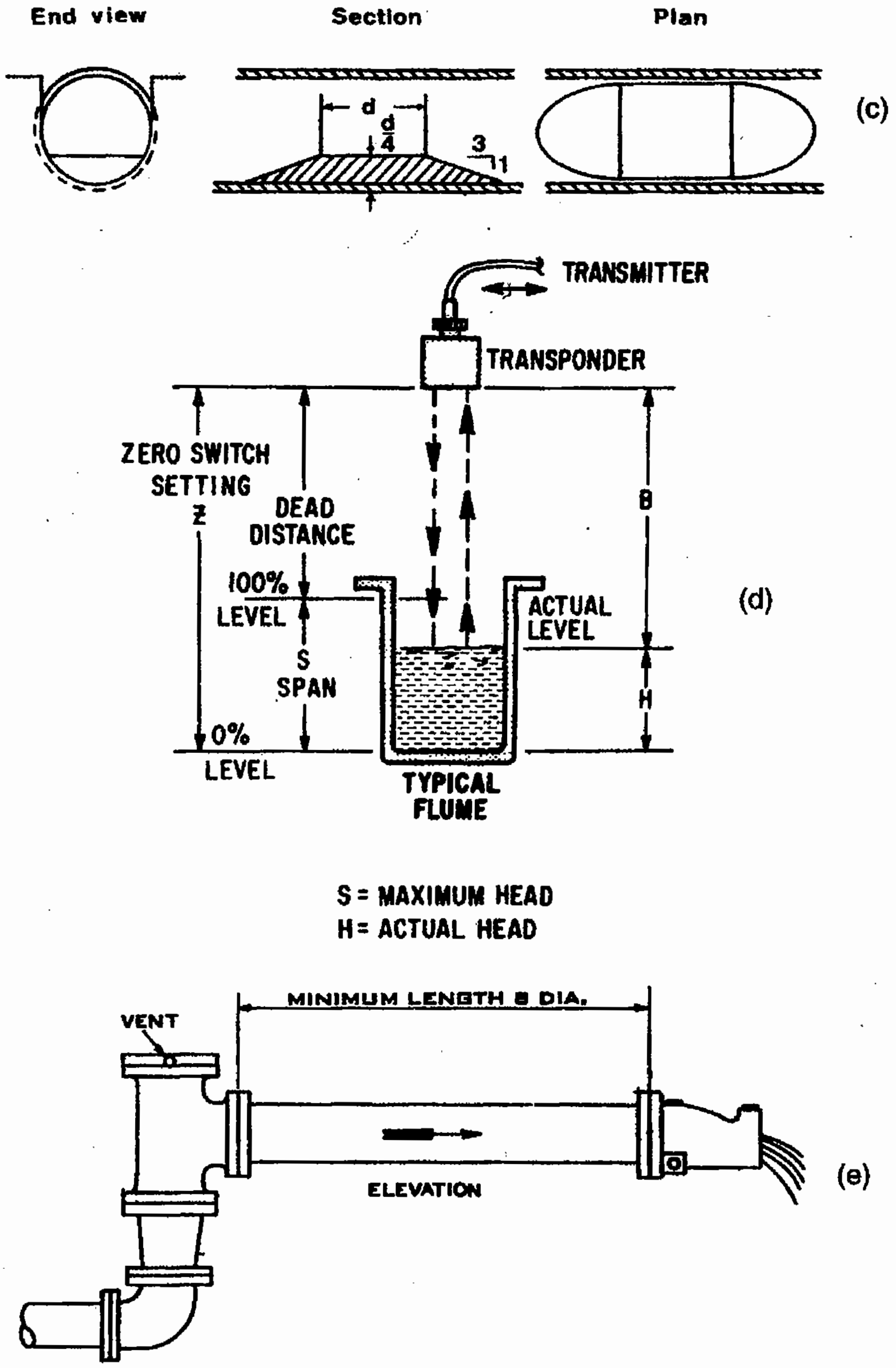


Figure 10-2—cont'd (c) Palmer-Bowlus flume (Refs. 2 and 3), (d) ultrasonic open channel liquid level indicator (courtesy Bailey-Fischer & Porter), and (e) Kennison nozzle.

Electromagnetic Meter. An electromagnetic flow meter utilizes Faraday's law to measure the flow. This law states that, if a conductor is passed through a magnetic field, a voltage will be produced that is proportional to the velocity of the conductor. In electromagnetic meters, the wastewater acts as a conductor, an electromagnetic coil creates the magnetic field, and two electrodes measure the induced voltage. This type of meter offers exceptional accuracy and can measure a large range of flows. Additionally, because it is essentially a straight piece of pipe, it has no additional head loss. The performance is unaffected by temperature, conductivity, viscosity, turbulance, and the presence of suspended solids. The greatest disadvantage of this type of meter is its initial cost and the need for trained personnel to handle routine operation and maintenance [Figure 10-2(a)].

Turbine Meter. The turbine meter utilizes a rotating element (turbine) whose rotational velocity is proportional to the velocity of water. The use of this device is usually limited to pipes running full, under pressure, and liquids low in suspended solids. These meters offer good accuracy and a good range of flows.

Ultrasonic Velocity Meter. The ultrasonic transit time velocity meter is an obstructionless device that is installed in a pipeline carrying the liquid. It consists of two ultrasonic transducers mounted in a diagonally opposed configuration to a precalibrated flow tube. The ultrasonic transducers send and receive ultrasonic pressure pulses. During operation, the pulses are alternately transmitted through the fluid, against and then with the direction of the flow. The pulse transit times, both upstream and downstream, are measured. The average fluid velocity is proportional to the difference between the two transit times. The meter calculates the time difference, and its output is, therefore, proportional to average fluid velocity.[5]

Ultrasonic Doppler Meter. The ultrasonic Doppler meter is similar in configuration to the ultrasonic velocity meter. The operating principle differs as the receiver measures the beam reflected by suspended solids or gas bubbles in the liquid. The frequency of the reflections are Doppler-shifted in proportion to the velocity of the suspended solids or gas bubbles. The transmitter and receiver may be a single ultrasonic transducer, or they may be mounted separately. The transducer(s) may be factory mounted on a flow tube or may be supplied as clamp-on device for use on standard lengths of pipe.[5]

Parshall Flume. The Parshall flume is based on fundamental open channel flow principles. It consists of three parts: a converging section, a throat, and a diverging section. The Parshall flume creates a change in flow pattern by a decrease in width and a simultaneous drop in water surface elevation at the throat. Because the throat width is constant, the free discharge is obtained from a single upstream measurement of depth. However, if the flume is operating under submerged conditions, the downstream depth must also be measured. Calibration curves are prepared for converting the depth readings to discharge. Parshall flumes are self-cleaning and offer small head loss, and calibration can be checked with manual measurement of head. Disadvantages include high cost and less accuracy compared with pressure meters, and a relatively long and straight approach channel is required. The design procedure for a Parshall flume is given in Chapter 14. A typical installation of Parshall flume with float actuated liquid level recorder is shown in Figure 10-2(b).

Palmer-Bowlus Flume. The Palmer-Bowlus flume creates a change in the flow pattern by decreasing the width of the channel without changing its slope. It is installed in a sewer at a manhole. It backs up the water in the channel. By measuring the upstream depth, the discharge is read from the calibrated curve prepared for that unit. It has lower head loss than the Parshall flume, but less accuracy [Figure 10-2(c)].

Weirs. Weirs are rectangular, trapezoidal (Cipolletti), or triangular (V-notch). They are used to measure the flow in open channels primarily because of their high degree of accuracy. They are easy to install in partly filled pipes, channels, or streams, and they act as a dam or obstruction. The water depth over the weir crest measured at a given distance upstream from the weir is proportional to the flow.

Commercially available weirs are cut in a vertical plate with a sharp crest. Proper discharge equations are used to determine the flow. The head over the weir must be measured accurately by a float, hook gauge, or level sensor. Equations of rectangular weirs and V-notches are given in Chapters 11–14. The weirs are accurate and relatively inexpensive. Their main disadvantages include large amounts of head loss, they must periodically be cleaned of any solids that may settle upstream, and the accuracy may be affected by excessive approach velocities and debris.

Depth Measurement. The method requires field measurement of the depth of flow and slope of the sewer. Several types of floats for recording the depth in sewers or manholes are commercially available. Using the Manning equation, the flow can be estimated from depth of flow and slope of the line between two manholes. This method gives approximate flow data; however, it is frequently used for making flow estimations in interceptors. The most desirable feature is the portable float, which can be easily installed in the manhole and then removed for use at other locations.

Sonic Level Meter. The principle of operation of sonic level meter is based upon the relationship between distance and time. The transducer emits a continuous series of sonic pulses that are reflected from the target surface. The round trip time of each pulse is directly proportional to the distance. Thus, the water surface elevation is clearly detected [Figure 10-2(d)].

Open Flow Nozzle. Open flow nozzles are crude devices used to measure flow at the end of freely discharging pipes. The major limitation of this device is that it must have a section of pipe that has a length of at least six times the diameter with a flat slope preceding the discharge. The most common examples of flow nozzles are the Kennison nozzle and the California pipe. Open flow nozzles create large head losses because of free discharge [Figure 10-2(e)].

10-3-2 Selection of Proper Flow Measurement Systems

In the design of wastewater treatment systems, the designer must be careful to select a proper flow measurement device to suit the particular need. Unfortunately, there is no device that is perfect for all situations. In fact, there is no device that is perfect for any situation. The design engineer has to evaluate and weigh the advantages and disadvantages of the available devices and choose the one that offers the fewest number of disadvantages.

In large pressure conduits (force mains), the installation of solids bearing Venturi, nozzle, or orifice meters should be considered. The Venturi meter is accurate, offers little head loss, and is free from solids accumulation but is relatively expensive. An orifice meter is inexpensive and exhibits greater flexibility in covering the different flow ranges but has the disadvantages of high pressure loss and possible accumulation of settled solids. The characteristics of the flow nozzle meter fall between those of the Venturi and orifice meters. An electromagnetic flow measurement device is a promising development for monitoring wastewater with high suspended solids.

For sewers and open channels, flumes and weirs are commonly used. Flumes can handle wastewater with high suspended solids and offer small head loss. Weirs are relatively inexpensive but require more maintenance and have large head losses. Flow calculations using the friction formulas may give large errors because of inaccurate determination of slope and coefficient of roughness. Thus, the most important criteria that must be considered in the selection of flow measurement devices include type of application, proper sizing, fluid composition, accuracy, head loss, installation requirements, operating environment, and ease of maintenance. Many questions must be asked by the designer in regards to the selection criteria of the flow measurement devices. Some of these questions concerning the selection criteria are summarized in Table 10-3.[1,4,5]

The accuracy and repeatability and range of applicability are extremely important criteria for selection of a flow measurement device. Instrument accuracy is usually expressed as a plus or minus (±) percentage of the maximum or actual flow rate. The repeatability is expressed as (±) percentage of full-scale meter reading. The range of application is the ratio of maximum and minimum flow range. The application, range of flow, accuracy, repeatability, and many selection criteria are summarized in Table 10-4.[1,4,5]

Venturi meters, Parshall flumes, and Palmer-Bowlus flumes have received wide application for flow measurement at wastewater treatment facilities. Details and design procedure for the Venturi meter are presented in Sec. 10-7. Design of the Parshall flume is covered in Chapter 14. The procedure for developing rating curves for a Palmer-Bowlus flume may be obtained in several other references.[4-8]

10-4 FLOW SENSORS AND RECORDERS

Automatic flow recording is normally required at medium- and large-sized plants. Normally, the flow-sensing signals from the point of measurement are transmitted to a central panel where they are recorded on a chart. The subject of automatic flow recording is complex, requiring an engineer's background in design, analysis, and application. There is a wide variety of systems commercially available that utilize principles of hydraulics, pneumatics, and electronics. In all cases a sensor signals the differential pressure, liquid level, or change in voltage, etc., to a detector that may convert the reading to the flow units for display or recording. Most modern flow measurement devices have an indicator, a recorder, an on-line integrator (totalizer) mechanism, and/or computer that enables the daily volume to be read off directly, thus eliminating the tedious job of manually integrating the data on the recorder chart. Discussion on instrumentation and controls may be found in Chapter 22.

TABLE 10-3 Selection Criteria for Flow Measurement Devices

Selection Criteria	Considerations
Flow category and application	Is the measurement device suitable for pressure pipe or open conduit flow? Has the equipment manufacturer and equipment selection guide (catalog) been consulted to determine the selection of the metering device for the particular application?
Sizing	Is the expected flow range for the device within the range of flow that needs to be monitored? Will proper operating velocities be maintained?
Flow sensor or recorder	What type of secondary element (level sensor, pressure sensor, transmitters, or recorders) will be needed, and will it be compatible with the primary device?
Fluid characteristics	Is the device compatible with the fluid (reactivity, suspended solids, density, temperature, and pressure) being monitored?
Accuracy and compatibility	Is the accuracy and repeatability of the device consistent with the application? Is the stated accuracy of the component consistent with the accuracy of other plant systems? Is it compatible with other flow-measuring devices already in operation at the facility? What effect will the environmental factors have on the stated accuracy of the primary device?
Operating environment	If used for an explosive gas application, is the equipment appropriately rated and designed to prevent explosion hazard? Is the equipment resistent to moisture and corrosive gases? What provisions are needed to ensure proper operation of the device within the expected temperature and humidity range?
Head loss	Is the head loss caused by the device within the constraints of the head loss constraints of the treatment process?
Installation and service	Is there sufficient straight length of pipe or channel available for installation of the device? What space limitations, if any, are there on the size of the device? Is the device properly located with respect to other valves, pumps, and other devices? Are the flow meter devices accessible for service? Could the meter be damaged during repairs or maintenance of nearby equipment? Are quick disconnect couplings, a method to drain the line, and bypass piping provided, as required? Have all regulatory constraints been addressed? Are provisions made for flushing or rodding the meter and tap lines?

Source: Adapted in parts from Refs. 1, 4, and 5.

TABLE 10-4 *Evaluation of Various Types of Devices Commonly Used for Wastewater Flow Measurement*

	Application		Flow Range and Accuracy											
	Pressure Flow	Open Channel	Range	Accuracy, Max. Flow (%)	Repeatability Percent of Full Scale	Effect of Solids in Wastewater	Head Loss	Power Requirement	Simplicity and Reliability	Maintenance Requirement	Ease of Calibration	Cost	Portability	Application
Venturi meter	Y	N	10:1	±0.5	±0.5	H[a]	L	L	G	M	G	H	N	Force main, wastewater
Flow nozzle meter	Y	N	4:1	±0.3	±0.5	H	M	L	G	L	G	M	N	Force main, wastewater
Orifice meter	Y	N	4:1	±1	±1	H	H	L	G	H	G	L	Y	Force main, wastewater
Electromagnetic meter	Y	N	10:1	±1-2[b]	±0.5	S	L	M	F	M	G	H	N	Force main, wastewater, sludge
Turbine meter[c]	Y	N	15:1	±0.25	±0.05	H	M	L	F	H	G	H	N	Force main, wastewater
Ultrasonic velocity	Y	N	10:1	±1-2	±1	M	L	M	F	M	G	H	N	Force main, wastewater
Ultrasonic Doppler	Y	N	10:1	±1-2	±1	M	L	M	F	M	G	H	N	Force main, wastewater
Parshall flume	N	Y	20:1	±5	±0.5	S	L	L	G	L	G	M	Y	Channel, wastewater, sludge
Palmer-Bowlus flume	N	Y	20:1	±10	±0.5	S	L	L	G	L	G	L	Y	Interceptor, manhole, channel, wastewater, sludge
Weirs	N	Y	20:1	±0.5	±0.5[d]	H	H	L	G	M	G	L	Y	Manhole, treatment unit, wastewater
Depth measurement	N	Y	10:1	±50		M	L	L	G	L	P	L	Y	Interceptor, wastewater, sludge
Open flow nozzle	N	Y	20:1	±1	±0.5[d]	S	H	L	G	M	F	L	Y	Outfall, discharge point, wastewater, sludge

[a]Effect of solids is substantially smaller if solids bearing or continuous flushing-type Venturi meter is used.
[b]Of full scale.
[c]Positive displacement type.
[d]Estimate.
F = fair, G = good, H = high, L = low, M = medium, N = no, P = poor, S = slight, Y = yes.
Source: Adapted in part from Refs. 1, 4, and 5.

10-5 EQUIPMENT MANUFACTURERS OF FLOW-MEASURING AND FLOW-SENSING DEVICES AND RECORDERS

A list of the manufacturers of flow-measuring devices, including flow sensors and recorders, is provided in Appendix D. The design engineer should work closely with the local representatives of the manufacturers in selection of equipment for specific needs. Shelley and Kirkpatrick conducted a state-of-the-art assessment of flow-measuring devices.[1] Their publication also contains a detailed manufacturers' survey of flow-metering devices, flow sensor probes, recording devices, specifications, and costs. Responsibilities of the design engineer and important considerations for equipment selection are also covered in Sec. 2-10.

10-6 INFORMATION CHECKLIST FOR DESIGN OF FLOW-MEASURING DEVICE

The following information is necessary prior to design and selection of a suitable flow measurement system:

1. The characteristics of the fluid for which flow measurement device is needed (suspended solids, density, temperature, pressure, etc.)
2. Expected flow range (maximum and minimum)
3. Accuracy desired (how much error in flow measurement can be tolerated)
4. Any constraints imposed by the regulatory agency
5. Location of flow measurement device and piping system (force main, sewer, manhole, channel, or treatment units)
6. Atmosphere of installation (indoors, outdoors, corrosive, hot, cold, wet, dry, etc.)
7. Head loss constraints
8. Type of secondary elements (level sensors, pressure sensors, transmitters, and recorders)
9. Space limitations and size of device
10. Compatibility with other flow measurement devices if already in operation at an existing portion of the treatment facility
11. Equipment manufacturers and equipment selection guide (catalog)

10-7 DESIGN EXAMPLE

10-7-1 Design Criteria Used

The following design criteria shall be used for the Design Example:

1. A Venturi meter will be provided in the force main.[a] The force main is 92 cm (36 in.) in diameter.
2. The tube beta ratio (diameter of throat/diameter of the force main) shall be equal to 0.5.
3. Maximum and minimum flow ranges are 1.321 and 0.152 m^3/s, respectively.

[a]Some designers may prefer a bypass line around the Venturi meter for maintenance purposes. In this design a bypass line is not provided since the flow can be diverted to a storage lagoon.

4. The flow measurement error shall be less than ±0.75 percent at all flows.
5. The head loss shall not exceed 15 percent of the meter readings at all flows.
6. The selected Venturi meter shall be capable of measuring flows of solids bearing liquids equivalent to that of municipal wastewater. Provision must be made for manual cleaning, hydraulic vent cleaners, or a continuous water purge system.

10-7-2 Design Calculations

Step A: Design Equations. All differential pressure meters utilize Bernoulli's principle to measure flows. The Bernoulli energy equation for two sections of a pipe is given below:

$$\frac{P_1}{\gamma} + Z_1 + \frac{V_1^2}{2g} = \frac{P_2}{\gamma} + Z_2 + \frac{V_2^2}{2g} + h_L \tag{10-1}$$

where

P_1, P_2 = internal pressures in the pipe, kN/m^2
γ = specific weight of the fluid, kN/m^3
Z_1, Z_2 = elevations of the centerline of the pipe, m
V_1, V_2 = velocity of the fluid, m/s
h_L = head loss, m

Since the differential pressure in the Venturi meter is measured between small lengths of pipes (above approach section and middle of the throat), the head loss (h_L) is negligible. Also, for a horizontal pipe, $Z_1 = Z_2$. Therefore, the equation can be simplified and rearranged as follows:

$$\frac{P_1}{\gamma} - \frac{P_2}{\gamma} = \frac{V_2^2}{2g} - \frac{V_1^2}{2g} \tag{10-2}$$

Replacing P_1/γ and P_2/γ by H_1 and H_2, substituting these values in Eq. (10-2), and using continuity equation ($V_1A_1 = V_2A_2 = Q$),

$$Q = \frac{A_1A_2\sqrt{2g\,(H_1 - H_2}}{\sqrt{A_1^2 - A_2^2}} = \frac{A_1A_2\sqrt{2gh}}{\sqrt{A_1^2 - A_2^2}} \tag{10-3}$$

where

Q = pipe flow, m^3/s
H_1 = upstream piezometric head, m
H_2 = throat piezometric head, m
A_1 = force main area, m^2
A_2 = throat area, m^2
h = diferential piezometric head ($H_1 - H_2$), m

Under actual operating conditions and for standard meter tubes, including allowance for friction, Eq. (10-3) is reduced to

$$Q = C_1 K_1 A_2 \times \sqrt{2g}\sqrt{h} \tag{10-4}$$

where

C_1 = velocity, friction, or discharge coefficient (dimensionless)
K_1 = coefficient (dimensionless)

$$= \frac{1}{\sqrt{1-(A_2/A_1)^2}} = \frac{1}{\sqrt{1-(D_2/D_1)^4}} \tag{10-5}$$

D_1, D_2 = diameter of the pipe and the throat, m

For standard Venturi meter the diameter of the throat is one-third to one-half of the pipe diameter and the value of K_1 lies between 1.0062 and 1.0328. The value of C_1 will generally range from 0.97 to 0.99.[4] The typical variation in C_1 with respect to the Reynolds number is given in Refs. 7–11. The value of C_1 is normally provided by the manufacturer. Figure 10-1(a) illustrates the Venturi meter arrangement and various notations used in the above equations.

Step B: Unit Sizing and Calibration Curve

1. Determine constant K_1.
   The Venturi tube has $D_2/D_1 = 0.5$.
   Throat diameter $D_2 = 46$ cm (18 in.)

$$K_1 = \frac{1}{\sqrt{1-(0.5)^4}} = 1.0328$$

2. Develop calibration equation from Eq. (10-4).
   Assume

$$C_1 = 0.985$$

$$Q = 0.985 \times 1.0328 \times \frac{\pi}{4}(0.46 \text{ m})^2 \times \sqrt{2 \times 9.81 \text{ m/s}^2}\sqrt{h}$$

$$= 0.7489\sqrt{h} \text{ m}^3\text{/s} \tag{10-6}$$

3. Develop calibration curve.
   Assigning different values of differential head recorded by the meter, the pipe discharge can be obtained from Eq. (10-6). At maximum peak design and minimum initial flows of 1.321 and 0.152 m^3/s, the differential meter readings will be 3.111 and 0.041 m, respectively (122.48 and 1.61 in.). The calibration curve is shown in Figure 10-3. If mercury is used in the glass tube, then the differential pressure readings must be adjusted for the specific weight of mercury (13.58).

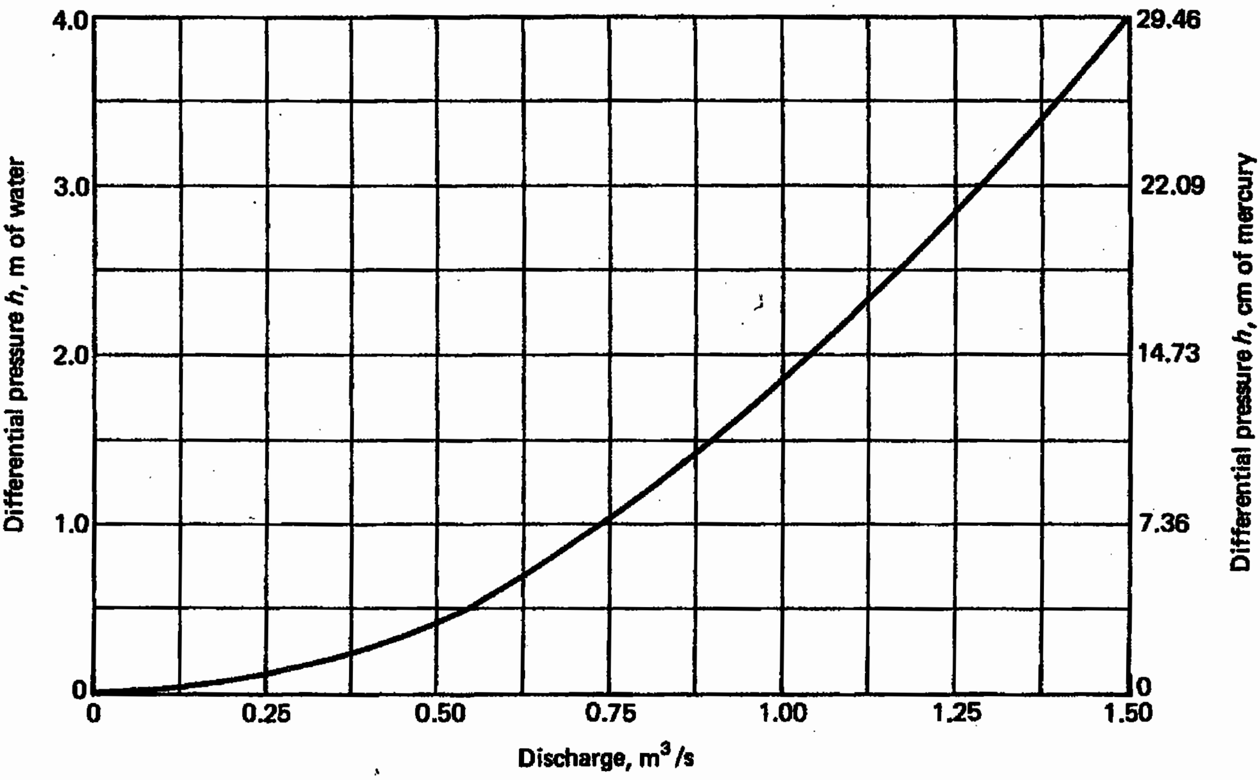


Figure 10-3 Calibration Curve of the Venturi Tube Meter Used in the Design Example.

Step C: Head Loss Calculations. In a Venturi tube, because of gradual contraction of the approach section, the head loss is considered negligible. Likewise, because of the short throat length, the head loss in this section can also be neglected. The head loss in the recovery section is estimated from Eq. (7-5).

$$h_L = K\,V_2^2/2g$$

Another method of calculating the head loss through a Venturi meter is based on the differential value. Generally, the head loss is 8–18 percent of the differential.
where

h_L = head loss through the Venturi meter, m
K = 0.14 for angles of divergence of 5° (Appendix B)

At maximum and minimum flows of 1.321 and 0.152 m³/s, the head losses are calculated as follows:

$$h_L \text{ at max flow} = \frac{0.14}{2 \times 9.81\ \text{m/s}^2}\left[\frac{1.321\ \text{m}^3/\text{s}}{\pi/4 \times (0.46\ \text{m})^2}\right]^2$$

$$= 0.45\ \text{m}$$

$$h_L \text{ at min flow} = \frac{0.14}{2 \times 9.81 \text{ m/s}^2} \left[\frac{0.152 \text{ m}^3/\text{s}}{\pi/4 \times (0.46 \text{ m})^2} \right]^2$$
$$= 0.006 \text{ m}$$

The above head loss values can be checked from the differential readings. The differentials at maximum and minimum flows are 3.111 and 0.041 m (Sec. 10-7-2, Step B, 3). The above head loss values are 14.5 percent of the differential readings of the meter at respective flows and are within the permissible limits.

Many manufacturers of Venturi meter give the value of the differential at a unit flow of 1.0 m^3/s. From this value, the differentials at maximum and minimum flows can be calculated. The manufacturers also provide the minimum permissible differential at the lower end of the range. They also provide a multiplying factor to calculate head loss from the calculated value of the differential. This factor is generally in the range of 0.1 to 0.15.

Step D: Design Details. The design details of the flow-measuring system provided in the Design Example are given in Figure 10-4. The design details include the universal Venturi tube, manual vent cleaners for high- and low-pressure connections, two pressure sensors, and a transmitter.

10-8 OPERATION AND MAINTENANCE

Before plant startup the Venturi flow meter shall be inspected for proper installation and freedom from debris. The pressure differential probe sensors shall be checked for proper connections. The manual vent cleaners in high- and low-pressure connections shall be cleaned daily. All tubing shall be checked frequently for clogging and slime growth. Periodic checks of indicated and recorded data against computed flows based on actual measured head conditions at the flow-measuring device or other backup system shall be made. The calibration curve shall also be checked frequently.

10-9 SPECIFICATIONS

Specifications of the Venturi tube flow meter, sensor, and transmitter are briefly presented below. The purpose of these specifications is to describe the various components that could not be fully covered in the Design Example. These specifications should be used as a tool to fully understand the design. Detailed specifications should be prepared by the design engineer for a specific design in consultation with the equipment manufacturers. These specifications are partly adapted from Refs. 11–13.

10-9-1 General

The Venturi tube flow meter shall be capable of measuring flows of raw wastewater. It shall consist of a primary flow Venturi element with manual vent cleaners for high- and

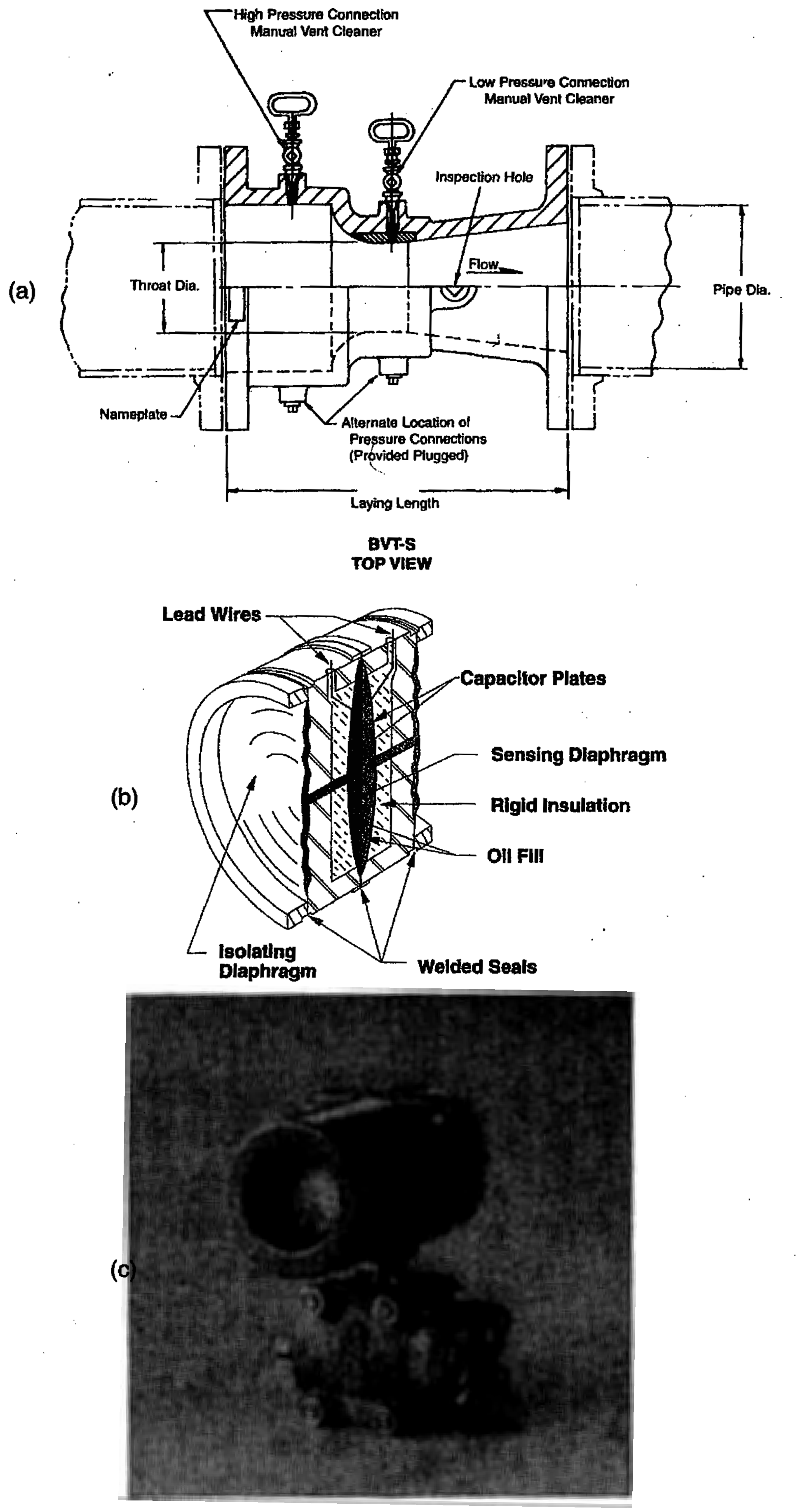


Figure 10-4 Details of Flow-Measuring System: (a) full-flanged primary flow element with manual vent cleaners (courtesy Badger Meter Inc.), (b) cross section of sensor, and (c) differential pressure transmitter (courtesy Rosemount, Inc.).

low-pressure connections, two pressure sensors, and a differential pressure transmitter, which shall translate differential pressure into an output signal. The Venturi meter, sensor, and transmitter shall be installed in a dry vault with an ambient temperature range of 0–38°C.

10-9-2 Primary Flow Element

A differential producing primary flow element shall be installed in the 92-cm pipe. The flow element shall be of the pressure differential producing type, utilizing pure static pressure sensed at the inlet and the throat. The flow element shall consist of four principal components, namely the inlet section, the throat section, the connection flange, and the pressure recovery section.

1. The inlet section shall be comprised of a cylindrical section similar to the force main (92-cm diameter) and shall provide a smooth, continuous transition for the fluid from the pipe into the throat section. The inlet cone shall have an angle of reduction of 20–23°.
2. The throat shall be a circular cylinder with suitable contours at each end to assure a smooth transition from inlet to throat and from throat to recovery section. The low-pressure sensor holes shall be located at the approximate midpoint of the throat.
3. The full-flanged primary element shall be designed for wastewater application with suspended solids.
4. The pressure recovery section shall be a frustum of a right circular cone. The cone shall have an incline angle of 5° and shall have a diameter similar to the force main at its cylindrical section.
5. The primary element shall be of a design for a beta ratio of 0.5.
6. The basic performance and accuracy of the primary element shall be unaffected by line pressure and/or temperature changes or by a butterfly or plug valve installed downstream of the Venturi.
7. The pressure taps shall assure stable measurement and minimize solids buildup at the taps. Manual vent cleaners shall be provided to clean any solids buildup.
8. The tube, throat, and other primary elements shall be constructed of suitable material for municipal wastewater applications, meeting ASTM specifications for stem, valves, flanges, and pipe fittings.
9. The manufacturer shall furnish a certified calibration curve that shall have a plot of discharge versus differential pressure (see Figure 10-3).
10. The manufacturer shall furnish certified data, substantiating tube proportions and performance. The data shall include (a) coefficient values, tolerance, and proof of their independence of the pipe beta ratio and a Reynold's number of 50,000 and greater; (b) effects of pipe convergence and divergence; (c) head losses as a function of velocity head expanded; (d) error less than ±0.75 percent of the actual rates of flow corresponding to the differential produced over the range of flow 10:1; and (e) 1:1 transfer of differential pressure at the Venturi tube and the differential pressure output of the sensors.
11. Performance of the primary element shall be capable of being checked by a ma-

nometer without breaking line integrity or draining the line. It shall also be possible to inspect the interior of the primary element in place without breaking the line.

10-9-3 The Sensor

The sensor shall utilize the isolating diaphragms to detect and transmit the process pressure to the oil-filled fluid, in turn transmitting the process pressure to the sensing diaphragm. The sensing diaphragm shall deflect in response to differential pressure across it to the capacitor plates on both sides of the sensing diaphragm. The transmitter electronics shall convert the differential capacitance between the sensing diaphragm and the capacitor plates into a 4–20 mA d-c signal and a digital output signal.

10-9-4 Differential Pressure Transmitter

The transmitter shall incorporate a capacitance actuated measuring cell coupled with microprocessor-based digital electronics. The transmitter shall utilize integral pushbuttons for setting zero and span and shall be field programmable using a handheld programmer to allow for configuring the transmitter output to be linear or square root. The transmitter shall have an accuracy of ±0.075 percent of span.

10-10 PROBLEMS AND DISCUSSION TOPICS

10-1 A Venturi meter is 30 cm × 20 cm. The differential gauge is deflected 5.52 cm when flow is 0.04 m^3/s. The specific gravity of the gauge liquid is 1.25. Determine the meter coefficient K and discharge coefficient C_1.

10-2 A Venturi meter with a 10-cm throat is installed in a pipe that is inclined upward at an angle of 45° to the horizontal. The diameter of the pipe is 20 cm. The distance between the pressure taps along the pipe is 1.5 m, and the differential pressure is 69 kPa. Estimate the flow of water in the pipe.

10-3 A Venturi meter has a 30-cm throat. Calculate the head loss if $K = 0.12$ and the discharge is 0.1 m^3/s.

10-4 The design of a proportional weir for velocity control is given in Chapter 8. Using this design, calculate the discharge through the channel if head over the weir crest is 0.7 m.

10-5 A Parshall flume is designed in Chapter 14. The throat width is 1.22 m (4 ft), and design dimensions are given in Figure 14-23. Calculate the discharge through the flume if the depth of flow H_a at the throat is 0.4 m.

10-6 Review the design of Palmer-Bowlus flume in Ref. 4. Study Example 3-2 on page 91 of this reference. Determine the discharge if the depth of flow is 0.3 m.

10-7 Develop the theoretical formula for flow over a rectangular weir.

10-8 The Francis equation is generally used to calculate the discharge over a rectangular weir. This equation takes into account the contracted width of the weir

$$Q = \tfrac{2}{3} C_d L' \sqrt{2g}\, H^{3/2}$$

where L' is the contracted width $(L - 0.1nH)$ and n is the number of end contractions. A sharp-crested rectangular weir is used to measure flow in a rectangular channel that is 3 m wide. The weir is 1 m above the floor of the channel and extends across the channel width. Calculate the depth in the channel upstream of the weir if discharge in the channel is 12.0 m^3/s.

10-9 The discharge over a triangular weir is expressed by the equation

$$Q = \frac{8}{15} C_d \sqrt{2g} \tan \frac{\theta}{2} H^{5/2}$$

where

Q = discharge in m^3/s
H = head over V-notch, m
θ = angle of the notch

An effluent weir plate contains 25 90° V-notches. Calculate the discharge if head over the notches is 20 cm. Assume $C_d = 0.6$.

10-10 Recorder and float system was used in a sewer manhole for recording flow. The depth of flow in the manhole is the same as that in the 46-cm (18-in.) diameter sewer connected in the manhole. The slope of the sewer is 0.0012. Calculate the flow if the recorded depth at any time is 16 cm ($n = 0.013$).

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11

Grit Removal

11-1 INTRODUCTION

Grit includes sand, dust, cinder, bone chips, coffee grounds, seeds, eggshells, and other materials in wastewater that are nonputrescible and are heavier than organic matter. It is necessary to remove these materials in order to (1) protect moving mechanical equipment and pumps from unnecessary wear and abrasion, (2) prevent clogging in pipes and heavy deposits in channels, (3) prevent cementing effects on the bottom of sludge digesters and primary sedimentation tanks, and (4) reduce accumulation of inert material in aeration basins and sludge digesters, which would result in loss of usable volume.

In this chapter the design information on various types of grit removal facilities is provided. Design procedure and equipment details, operation and maintenance, and equipment specifications for aerated grit removal facility are covered in the Design Example.

11-2 LOCATION OF GRIT REMOVAL FACILITY

With increasing mechanization of wastewater treatment plants, greater consideration is given to equipment protection. As a result, grit removal facilities are commonly provided at all treatment plants. It is desirable to locate grit removal facilities ahead of the raw wastewater pumps. However, in many instances the incoming sewers are at such depths that location of grit removal facilities ahead of the pumping station becomes undesirable. In such instances, the grit removal facilities are provided after the pumping station or in conjunction with the primary clarifiers. The advantages and disadvantages of having a grit removal facility installed at different locations are given in Table 11-1.

11-3 GRAVITY SETTLING

Grit is removed from wastewater by gravity settling or by centrifugal force. Gravity settling is encountered in many treatment units. Depending on the concentration and the tendency of particles to interact, four types of settling can occur in aqueous solutions: discrete (Type I), flocculant (Type II), hindered or zone (Type III), and compression (Type IV). Discrete settling occurs when the particles settle as individual entities (for example, grit channel). Flocculant settling occurs when solids flocculate or agglomerate (for example, primary sedimentation). Hindered settling occurs when the concentration

TABLE 11-1 Comparison of Various Locations of Grit Removal Facility

Location	Advantages	Disadvantages
Ahead of lift station	Maximum protection of pumping equipment	Frequently deep in the ground, high construction cost, not easily accessible, and difficult raising the grit to ground level
After pumping station	Ground level structure; accessible and easy to operate	Some abnormal wear to pumps
Degritter in conjunction with primary sludge	Usually low capital and operation and maintenance costs; cleaner and drier grit	Pumping equipment not adequately protected

of solids is high, and solids settle as a "mass" or "blanket" (for example, upper zone of final clarifier). Compression settling occurs when solids remain supported on top of each other; further settling is possible only by compression of the structure (for example, thicker and lower zone of the final clarifier). The principles of discrete settling are discussed below. Flocculant, hindered (zone), and compression settling behaviors of particles involved in wastewater treatment plants are discussed in Chapters 12, 13, and 16, respectively. Refs. 1–4 provide excellent discussion on four types of settling.

Discrete, or Type I, settling occurs where solids concentration is low. There is no significant interference between the settling particles, and the particles settle as individual entities. Examples are grit channels and settling of filter media after backwash. Equations that express the settling behavior of discrete particles are given below [Eqs. (11-1)–(11-4)]:

$$v = \frac{H}{t} \tag{11-1}$$

$$v = \left[\frac{4}{3}\frac{g(\rho_s - \rho)d}{C_D\rho}\right]^{1/2} \text{ or } \left[\frac{4g(S_s - 1)d}{3C_D}\right]^{1/2} \qquad \text{(Newton's Equation)} \tag{11-2}$$

$$C_D = \frac{24}{N_R} + \frac{3}{\sqrt{N_R}} + 0.34 \tag{11-3}$$

$$N_R = \frac{v\rho d}{\mu} \text{ or } \frac{vd}{\nu} \tag{11-4}$$

where

C_D = drag coefficient
d = particle diameter, m (ft)
g = acceleration caused by gravity, m/s^2 (ft/s^2)

H = depth of fall, m (ft)
N_R = Reynolds number
S_S = specific gravity of particle
t = settling time, s
v = settling velocity of particle, m/s (ft/s)
ρ = density of fluid, kg/m^3 (lb/ft^3)
ρ_S = density of particle, kg/m^3 (lb/ft^3)
μ = dynamic viscosity, N · s/m^2
ν = kinematic viscosity, m^2/s (ft^2/s)

Note: Several units in Eqs. (11-2)–(11-4) are

$S_S = \rho_S/\rho$
$\nu = \mu/\rho$
N = mass × acceleration (kg m/s^2)
$\rho = \gamma/g$ where γ = specific weight of fluid, N/m^3 (lb/ft^3); or

$$\rho = \frac{\text{kg} \cdot \text{m/s}^2 \cdot 1/\text{m}^3}{\text{m/s}^2} = \text{kg/m}^3$$

Equation (11-2) applies for spherical particles and is called Newton's law; Reynolds number is calculated from Eq. [11-4]. For N_R less than 0.3 (laminar flow), the first term in Eq. (11-3) predominates and $C_D = 24/N_R$. Substituting this value in Eq. (11-2) yields Stokes' law [Eq. (11-5)].

$$v = \frac{g(\rho_s - \rho)d^2}{18\mu} \text{ or } \frac{g(S_s - 1)d^2}{18\nu} \qquad (\text{Stokes' Equation, } N_R < 0.3\text{, laminar flow}) \quad (11\text{-}5)$$

A Reynolds number N_R greater than 10^4 indicates turbulent flow, and the third term in Eq. (11-3) predominates. Substituting $C_D = 0.34$, Eq. (11-2) is transformed into Eq. (11-6):

$$v = \left[3.92\frac{g(\rho_s - \rho)d}{\rho}\right]^{1/2} \qquad (N_R > 10^4\text{, turbulent flow}) \qquad (11\text{-}6)$$

The settling velocity equations given above apply for spherical particles falling through a fluid. If the Reynolds number is greater than 0.3 and less than 10^4, the settling is in the transition zone, and the settling velocity of a particle from Eq. (11-2) is determined by a trial and error solution. The values of C_D and N_R are calculated from Eqs. (11-3) and (11-4), respectively.

In the design of a settling basin, the settling velocity v_t (also called terminal velocity) of the smallest particle is selected. The basin is designed such that all particles having a settling velocity greater than the terminal velocity are fully removed. Particles having a settling velocity (v_i) smaller than the terminal velocity are partially removed [Figure 11-1(a)]. The settling velocity of discrete particles is obtained from a column analysis test. The column used in such test is shown in Figure 11-1(b). A typical settling

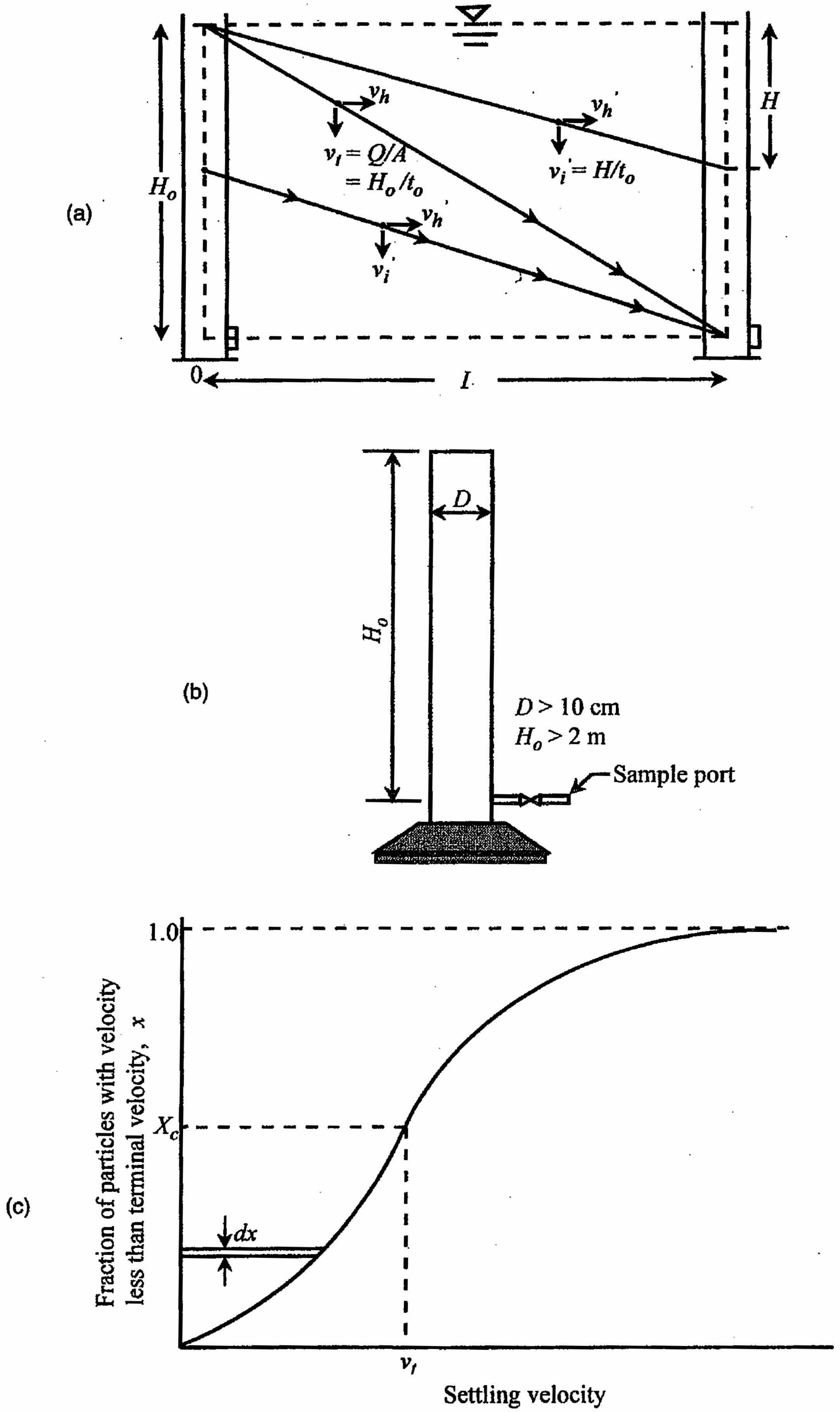


Figure 11-1 Settling Behavior of Discrete Particles: (a) settling trajectory to particles $v_i < v_t$, (b) column for settling test, and (c) fraction removal curve.

curve of discrete particles from a column analysis test is shown in Figure 11-1(c). The fraction removed in a settling basin having a terminal velocity of v_t is given by Eq. (11-7):

$$F = (1 - X_C) + \int_0^{X_C} \frac{v_i}{v_t}\, dx \tag{11-7}$$

where

F = fraction removed
v_i = settling velocity of the particles, v_i less than v_t
X_C = fraction of particles with velocity v_i less than v_t
$(1 - X_C)$ = fraction of particles removed with settling velocity greater than v_t
$\int_0^{X_C} \frac{v_i}{v_t} dx$ = fraction of particles removed with v_i less than v_t

The terminal velocity has great significance in the design of all settling basins. It is also called design overflow rate, surface settling rate, or hydraulic loading ($m^3/m^2 \cdot d$). It is also expressed numerically by Eq. (11-8):

$$v_t = \frac{Q}{A} = \frac{H_o}{t_o} \tag{11-8}$$

where

Q = flow rate to the settling basin, m^3/s (ft^3/s)
A = surface area or plan area of the basin, m^2 (ft^2)
H_o = effective depth of settling basin, m
t_o = settling or detention time, s (common units are min, h)

11-4 TYPES OF GRIT REMOVAL FACILITIES

The quantity and quality of grit and the effect of the grit on treatment units are important factors to be considered in selection of grit removal facilities. The choice of grit removal equipment may also be dictated by head loss and space requirements and the type of equipment used in other parts of the plant. Grit removal facilities fall into two general categories: (1) selective removal from wastewater and (2) removal with organic matter followed by degritting.

11-4-1 Selective Grit Removal from Wastewater

Grit is selectively removed from other organics in devices such as (1) a velocity-controlled grit channel, (2) an aerated grit chamber, or (3) a vortex-type grit chamber. These unit operations are commonly used and are discussed below in detail.

Velocity-Controlled Grit Channel. The grit in wastewater has a specific gravity in the range of 1.5–2.7. The organic matter in the wastewater has a specific gravity around 1.02. Therefore, differential sedimentation is a successful mechanism for separation of grit from organic matter. Also, the grit exhibits discrete settling whereas organic matter settles as flocculant solids.

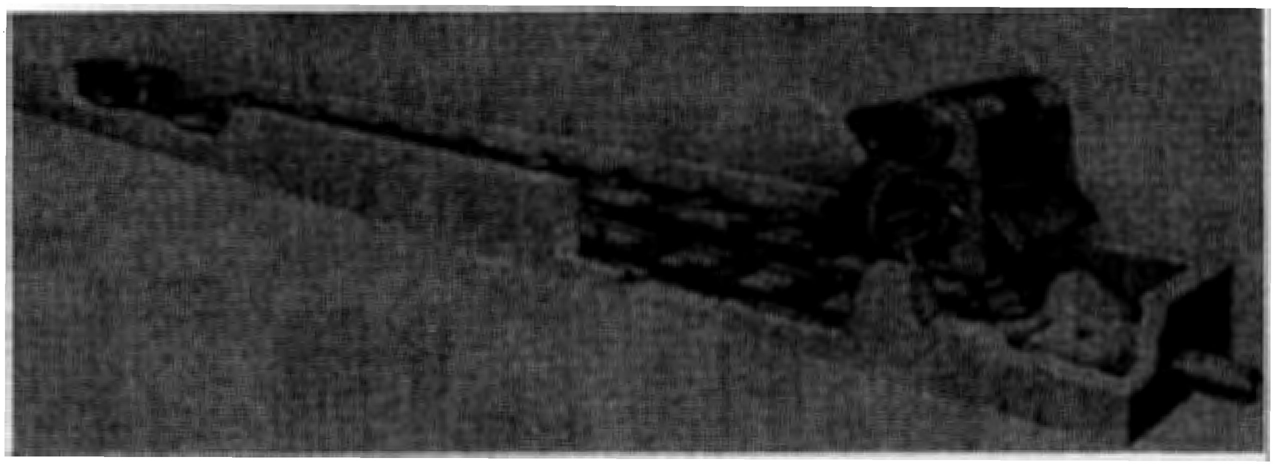
The velocity-controlled grit channel is a long, narrow sedimentation basin in which velocity is controlled. In some designs, attempts have been made to control the velocity by use of multiple channels. A more economical arrangement and better velocity control is achieved by the use of control sections at the outlet end of the channel. The control sections include proportional weir, Sutro weir, Parshall flume, parabolic flume, etc. These control sections maintain constant velocity in the channel at a wide range of flows. The procedure for designing grit channel and discharge equations for various types of control sections are given in Refs. 5–9. Typical values of detention time, horizontal velocity, and settling velocity for a 65-mesh (0.21-mm-diameter) material are, respectively, 60 s, 0.3 m/s, and 1.2 m/min (1730 $m^3/m^2 \cdot d$).[1] The head loss through the velocity-controlled grit channel is 30–40 percent of the maximum water depth in the channel.

The grit channel may be manually cleaned or mechanically cleaned. Manually cleaned channels are used only at small plants. The channels have hoppers at the bottom for grit storage and are normally drained for manual removal of grit. The mechanically cleaned grit channel utilizes a grit collection mechanism to move the grit to a sump. The grit is removed from the sump by mechanical means to a storage area. The grit collection and removal equipment is discussed in Sec. 11-5. Components of velocity-controlled grit channels are shown in Figure 11-2. Design calculations and details of a proportional weir and Parshall flume are provided in Chapters 8 and 14.

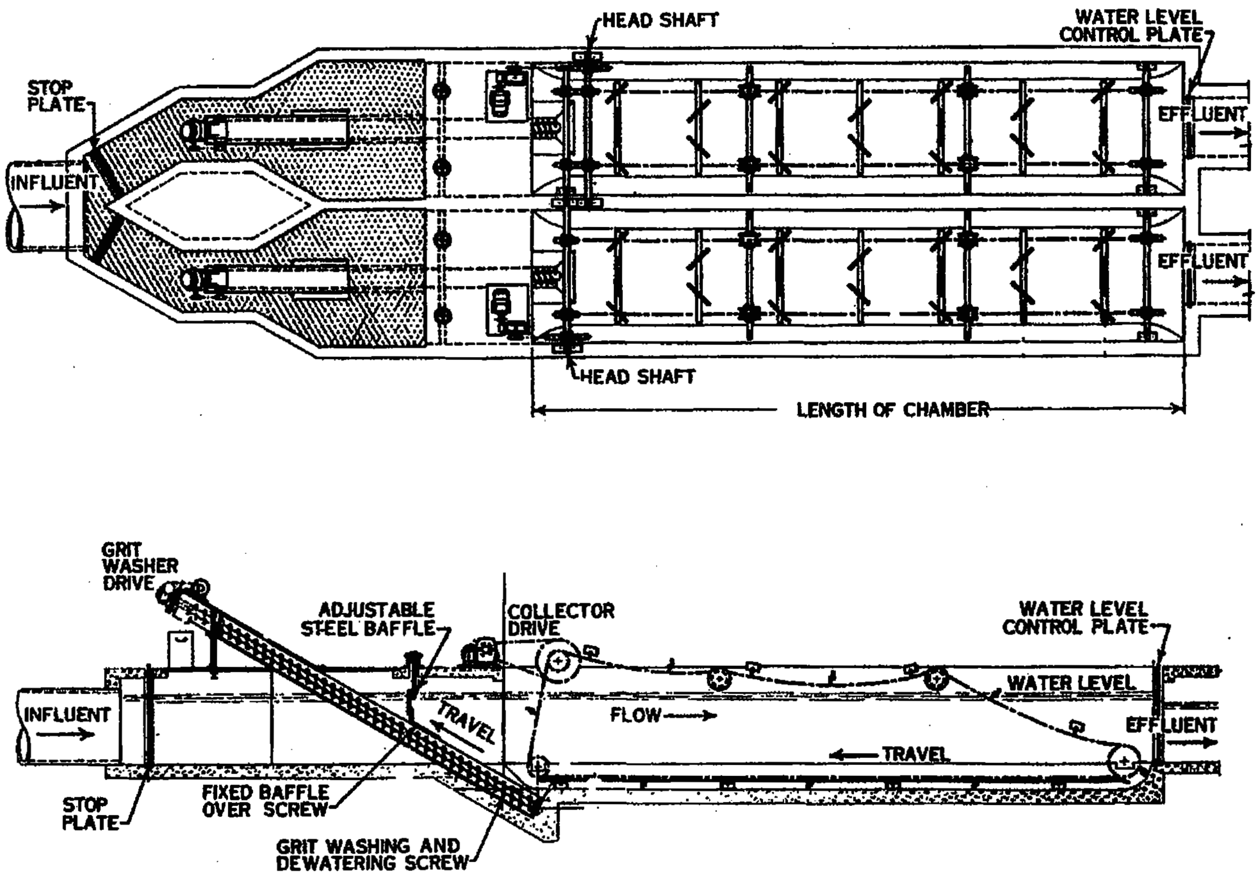
Aerated Grit Chamber. Aerated grit chambers are widely used for selective removal of grits. They are similar to standard spiral flow aeration tanks (Chapter 13). A spiral current within the basin is created by the use of diffused compressed air. The air rate is adjusted to create a velocity near the bottom, low enough to allow the grit to settle, whereas the lighter organic particles are carried with the roll and eventually out of the basin. The aerated grit chamber is normally designed to remove grit particles having a specific gravity of 2.5 and retained over a 65-mesh screen (0.21-mm-diameter); however, smaller particles may also be effectively removed by reducing the air supply that also reduces the velocity of spiral flow.

Advantages of Aerated Grit Chamber. The aerated grit chambers are extensively used at medium- and large-sized treatment plants. They offer many advantages over the velocity-controlled grit channels. Some of the advantages are as follows:

1. An aerated grit chamber can also be used for chemical addition, mixing, and flocculation ahead of primary treatment.
2. Wastewater is freshened by the air; thus, reduction in odors and additional BOD_5 removal may be achieved.
3. Minimal head loss occurs through the chamber.
4. Grease removal may be achieved if skimming is provided.



(a)



(b)

Figure 11-2 Details of Velocity-Controlled Grit Channel: (a) velocity-controlled grit channel (courtesy Envirex, U.S. Filter) and (b) plan and longitudinal section of a double channel grit collector (from Ref. 6; used with permission of Water Environment Federation, and American Society of Civil Engineers).

TABLE 11-2 Design Factors and Typical Design Values for Aerated Grit Chambers

Design Factor	Range of Value	Comment
Dimensions		The width of the basin is limited to provide roll action in the tank.
Depth, m	2–5	
Length, m	7.5–20	
Width, m	2.5–7.0	
Width/depth ratio	1:1–5:1	
Length/width ratio	2.5:1–5:1	
Transverse velocity at surface	0.6–0.8 m/s	
Detention time at peak flow, min	2 to 5	If grit chamber is used for preaeration or to remove grit less than 65-mesh (0.21-mm), a longer detention time may be provided.
Air supply	4.6 to 12.4 L/s·m of tank length (3–8 cfm/ft)	Higher air rate should be used for wider and deeper tanks. provision should be made to vary the air flow. An air flow rate of 4.6–8 L/s·m in a 3.5- to 5-m-wide and 4.5-m-deep tank give a surface velocity of approximately 0.5–0.7 m/s. The velocity at the floor of the tank is 75 percent of the surface velocity. A velocity of 0.23 m/s is required to move a 0.2-mm sand particle along the tank bottom.
Inlet and outlet structures	—	Inlet and outlet structures must be such as to prevent short-circuiting and turbulence. Inlet to the chamber should introduce the influent into circulation pattern. Outlet should be at a right angle to the inlet (Figure 11-3). The inlet and outlet are sized and built in such a way that the velocity exceeds 0.3 m/s under all flow conditions to minimize the deposits.
Baffles	—	Longitudinal and transverse baffles improve grit removal efficiency. If the grit chamber is much longer than the width, a transverse baffle should be considered.
Chamber geometry		Location of air diffusers, sloping tank bottom, grit hopper, and accommodation of grit collection and removal equipment should all be given consideration in chamber geometry. The diffusers are normally located approximately 0.6 m above the sloping tank bottom. Beneath the air diffusers would be the hopper for grit collection.

Source: Adapted in part from Refs. 1 and 6–10.

5. By controlling the air supply, grit of relatively low putrescible organic content can be removed.
6. By controlling the air supply, grit of any desired size can be removed. However, because of variable specific gravity and size and shape of the particles, there may be some limitations on removal.

Design Factor. It is important that the design engineer give consideration to those factors that control the design and performance of the process. Many of these factors include type of grit and other solids, detention time, air supply, inlet and outlet structures, dead spaces, tank geometry, and baffle arrangement. Different design factors, their significance, and their design values are given in Table 11-2. Many typical design configurations of aerated grit chambers are shown in Figure 11-3.

Grit collection and removal systems in aerated chambers are similar to those used for velocity-controlled channels. Often, an air lift decanting arrangement is used with aerated grit channels. Different types of grit collection and removal systems are discussed in Sec. 11-5.

Vortex-Type Grit Chamber. The vortex-type grit chambers (also called accelerated gravity separation device) utilize changing acceleration field. The devices thus utilize both gravitational and centrifugal forces for separation of grit. Two such devices are relatively recent developments and are gaining popularity for grit removal from storm water, combined sewer overflows, and municipal and industrial wastewaters. These devices are gravitational and centrifugal separators. Both types of devices are discussed below, and schematic flow diagrams of both types of devices are shown in Figure 11-4.[11-13]

Gravitational or Swirl-Flow. The Eutek free vortex grit removal unit, or Teacup solids classifier, is characterized by a dominant, strong free vortex caused by centrifugal and gravitational forces and secondary boundary layer velocities to separate and classify inorganic solids from organic solids and water.

The unit consists of a cylindrical section on top of a conical section. The influent enters tangentially around the upper midsection of this cylinder. The degritted effluent exits through the opening at the center of the top of the unit, and the discharge may be under gravity or pressurized. Grit is removed through the opening in the bottom of the conical section. The primary force for grit removal is derived from the strong free vortex similar to that of a hydrocyclone.

The high centrifugal forces within the cylinder minimize the release of solids. Particles, depending upon their size, density, and drag, are retained within the body of the free vortex near the center of the Teacup, while lighter and smaller particles are swept out of the unit. Grit and sand particles retained within the cylinder settle by gravity to the bottom of the unit, while organics, including those separated from the grit particles by centrifugal force, exit the unit with the degritted wastewater. The grit is discharged through the bottom port onto a belt escalator dewatering device called Grit Snail. The discharged grit to the snail's pool area settles on the unit's escalator steps and is slowly lifted out of the pool.

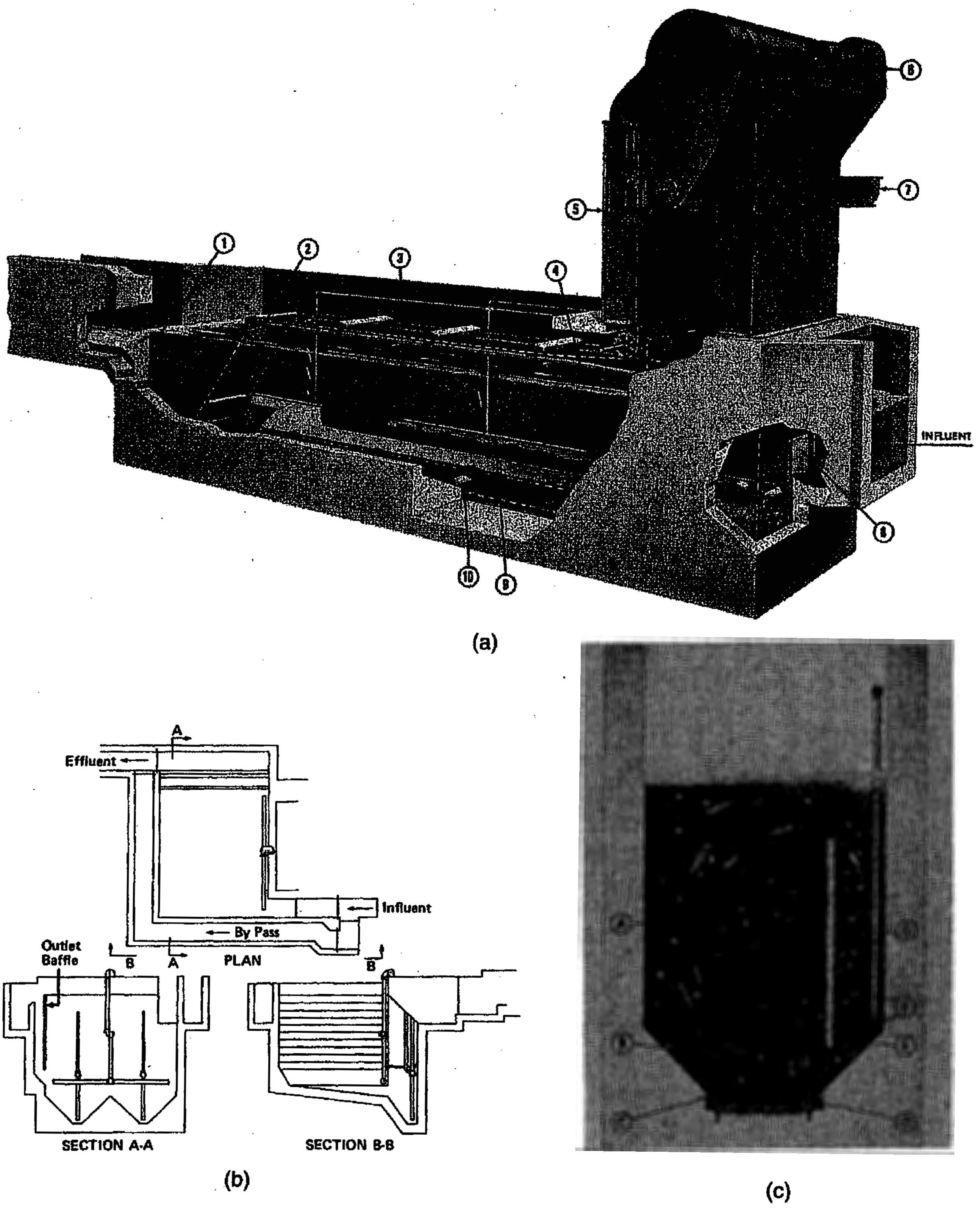


Figure 11-3 Details of Aerated Grit Chamber: (a) components of aerated grit removal facility: 1. effluent weir, 2. sprockets, 3. wooden circulation baffle, 4. guided chain support, 5. housing, 6. motorized drive unit, 7. screw conveyor, 8. inlet baffle, 9. air inlet pipe and headers, 10. chain and bucket collector mechanism (courtesy Envirex, U.S. Filter); (b) aerated grit chamber inlet and outlet arrangement (from Ref. 6; used with permission of Water Environment Federation and American Society of Civil Engineers); and (c) simplified cross section of aerated grit removal facility: A. flow through zone, B. grit and organic separation zone, C. collection chain guard, D. chain and bucket grit collector, E. circulation baffle, F. air inlet pipe and header, G. air circulation and grit conditioning zone (courtesy Envirex, U.S. Filter).

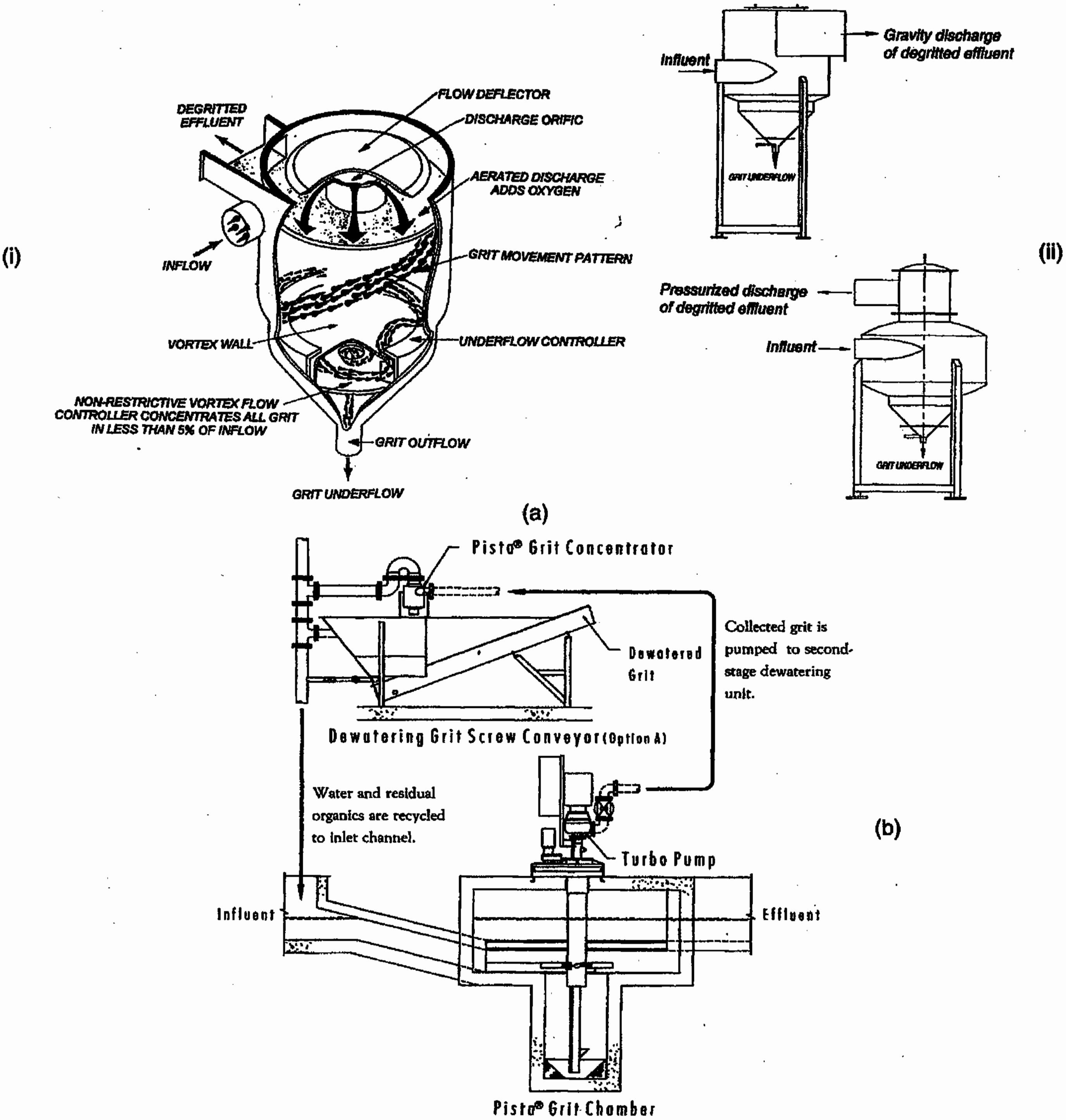


Figure 11-4 Schematic Flow Diagram of Vortex-Type Grit Chambers: (a) free vortex system, Teacup (courtesy Eutek® Systems, Inc.), (i) flow regime through the chamber, (ii) gravity and pressurized discharge; (b) vortex-type grit chamber (courtesy Smith & Loveless, Inc.).

The head loss requirement is a function of the particle size and their required removal. Teacup grit separators are generally designed to achieve 95 percent removal of 100-micron (150-mesh) grit. The head required to achieve this removal is a few meters. For 95 percent removal of 25-micron grit, the head requirement is 5 to 7 m. It may therefore be necessary to pump the flow into the Teacup. For municipal headworks applications, it is possible to perfectly match the sewer or pump hydraulics. With gravity flow, the unit will require installation deep in the ground, while proper influent pump station design may facilitate unit installation above grade. At peak design flow, the unit uses more head as the flow through the unit increases. This gives a corresponding improvement in performance efficiency. The units are sized to handle peak flow rates up to 0.31 m^3/s (7 mgd).

Centrifugal Vortex Induced Grit Separator. The Pista, or other, grit separators operate on the vortex principle. The unit has a cylindrical section on top of a conical section with a central grit hopper at the bottom. The influent is surface fed tangentially into the upper chamber and follows a 270-degree path before going out of the chamber. The rotating turbine maintains constant flow velocity. The rotating action of the turbine blades produces a toroidal flow path for grit particles. The adjustable pitch blades also promote separation of organics from the grit. The grit is propelled to the floor sufficiently in one revolution so as not to be within the influence of the outlet of the chamber. The flow continues to move circumferentially, and the grit is propelled along the bottom towards the center, while lighter organics are lifted and carried in the effluent. The grit moves inward and drops into the center of the storage hopper. The grit is removed or lifted from the hopper by an air pump or a turbo grit removal pump. Total head loss through the unit is small. The manufacturer claims that the overall average grit removal efficiency of a Pista grit separator is 95, 83, and 73 percent of 50-, 80-, and 140-mesh size grit. The available unit size ranges from 0.044 m^3/s (1 mgd) to 3.08 m^3/s (7 mgd). The hardware assembly is shown in Figure 11-4(b).

11-4-2 Combined Grit Removal with Organic Matter Followed by Degritting

Many systems are used to remove grit along with organic solids. Degritting of sludge is necessary to separate grit from the organic solids. Degritting devices include cyclones (hydrocyclones), centrifuges, and detritus tanks. All these devices are discussed below.

Cyclone (Hydrocyclone). Cyclones are cone-shaped devices into which dilute primary sludge is pumped tangentially. As fluid spirals inward, centrifugal force pushes the grit toward the walls. The accumulated grit slides spirally down to the apex of the cone and then discharges into a chamber through an orifice. The effluent containing organic solids is discharged from the top. The principles of operation are similar to those of vortex-type grit chambers discussed above.

The efficiency of the cyclone in removing grit depends on (1) the effective size of the particles, (2) specific gravity of the particles, (3) differential pressure across the cone, and (4) diameter of the cone. For degritting purposes it is critical that the volume of pumped sludge and the resultant pressure across the cyclone be specified by the manufacturer. For sludge less than 1.5 percent solids, a pressure drop of 70–80 kN/m^2 (10–13 psi) across the cone has been suggested.[6]

Centrifuge. Centrifugal force is used to cause selective separation of grits from organic matter in various types of centrifuges. Discussion of centrifuges may be found in Chapters 16 and 18.

Detritus Tanks. Detritus tanks are square sedimentation tanks in which grit and organic solids are removed collectively. The solids are raked by a rotating mechanism to a sump at the side of the tank, from which they are moved up an incline by a reciprocating rake mechanism. The organic solids are separated from the grit and fall back into the basin while passing up the incline. Grit washing in a separate step is also done to provide cleaner and drier grit. Design details and advantages and disadvantages of this device are given in Refs. 1 and 6.

11-5 GRIT COLLECTION AND REMOVAL

Mechanical grit collection in velocity-controlled channels and aerated grit chambers is achieved by conventional equipment with scrapers, screws, buckets, plows, or some combination of these. In some instances steep bottom slopes and artificial velocities (aerated chambers) are created that tend to move the settled grit to a central point for removal.

Grit removal is accomplished by tubular conveyors, bucket-type collectors and elevators, screw conveyors, grit pumps, and clamshell buckets. In small aerated chambers, air pumps are also used. The tubular conveyors use a pipe in which there is an endless chain coupled with rubber flights. A portion of the pipe is mounted in the bottom of the hopper with the top of the pipe opened to allow grit to fall into a container between the flights.

The bucket-type collectors and elevators are equipped with chain-and-bucket conveyors. The buckets run the full length of the storage troughs, which move the grit to one end of the trough and then elevate it above the wastewater level in a continuous operation. Screw-type conveyors utilize helical blades that rotate and move the grits up an incline. Although wear is the general problem with all these grit collectors and elevators, tubular and screw conveyors produce grit that has less organics (because of additional washing in vertical rise) than that obtained from the bucket-type elevator or grit pump. Emptying of the basin may be necessary for routine maintenance. For this reason many grit removal facilities utilize clamshell buckets that travel on monorails and are centered over the grit collection and storage trough. These grab buckets operate on an intermittent basis. Multiple tanks are needed so that one unit can be bypassed during operation of the clamshell bucket, and tank emptying may not be necessary. Many types of grit collection and removal systems are illustrated in Figure 11-5.

11-6 QUANTITY OF GRIT

The quantity of grit varies greatly, depending on (1) type of collection system (separate or combined), (2) climatic conditions, (3) soil type, (4) condition of sewers and grades, (5) types of industrial wastes, (6) relative use of garbage grinders, and (7) proximity to the sandy bathing beaches. The grit and scum quantity may range from 5 to 200 $m^3/10^6$ m^3. Typical value is 30 $m^3/10^6$ m^3.

(a)

(b)

Figure 11-5 Grit Collector and Removal System: (a) screw collector and elevator and (b) chain and bucket grit collector (courtesy Envirex, U.S. Filter).

11-7 GRIT DISPOSAL

Various methods of grit disposal include sanitary landfill, land spreading, and incineration with sludge. For small- and medium-sized plants it is best to bury and cover the grit because the residual organic contents can be a nuisance. Unwashed grit may contain 50 percent or more organic material. It attracts flies and rodents and causes serious odor problems. Large treatment plants utilize grit washing in conjunction with grit moving and raising equipment. Elevated grit storage facilities for direct truck loadings are also used at larger plants.

11-8 EQUIPMENT MANUFACTURERS OF GRIT REMOVAL FACILITIES

Several manufacturers of grit removal facilities are listed in Appendix D. Many design decisions must be made prior to selection of equipment. Information such as type and number of grit collection and removal systems, flow conditions, space available, existing conditions, head loss constraints, etc., are necessary to make equipment selection. Important considerations for equipment selection and design engineer's responsibilities are covered in Sec. 2-10.

11-9 INFORMATION CHECKLIST FOR DESIGN OF GRIT REMOVAL FACILITY

The following information must be developed and decisions made before a design engineer can start the design of a grit removal facility:

1. Characteristics of wastewater and type of grit to be removed
2. Design average, peak, and low initial flows
3. Information on existing facility if plant is being expanded, and site plan and topographic maps
4. Type of grit removal facility to be provided (velocity control grit channel, aerated grit chamber, or combined grit and organics removal system, including degritting)
5. Influent pipe data and static head, force main, and hydraulic grade line if grit removal is preceded by a pumping station
6. Head loss constraints for grit removal facility
7. Treatment plant design criteria prepared by the concerned regulatory agency
8. Equipment manufacturers and equipment selection guides (catalog)

11-10 DESIGN EXAMPLE

11-10-1 Design Criteria Used

The following design criteria are used for the Design Example:

1. Provide two grit chambers with spiral circulation, capable of removing grit particles of 0.21 mm (65 mesh) or larger. Each chamber shall be designed for half the peak

flow. The influent and effluent structures shall be designed to handle the emergency flow conditions when one unit is out of service.

2. The grit removal facility shall be designed for maximum flow delivered by the pumping station. The maximum capacity of the pumping station under minimum static head is 1.56 m^3/s (24,700 gpm). The station capacity is slightly more than the design peak flow of 1.321 m^3/s. Because of the uncertainties in forecasting the friction head losses coupled with confidence in predicting flow rates, a slightly higher flow rate for the pump is often preferred. In actual practice, the variable-drive units are provided to match the flow conditions. Therefore, a maximum design flow of 1.321 m^3/s will be used for the design of the grit removal facility.
3. Provide detention time of at least 4 min at peak design flow when both chambers are in operation.
4. Provide an air supply of at least 7.8 L/s per meter of the tank length. Provide nozzle diffusers with coarse bubbles. Provision shall be made for 150 percent air capacity for peaking purposes.
5. The inlet and outlet shall be sized to provide a minimum velocity of 0.3 mps.
6. Provide width of the chamber 3.5 m.
7. Provide screw conveyor to move the grit to the hopper and grab buckets for grit removal.

11-10-2 Design Calculations

Step A: Geometry of Grit Chamber

1. Calculate the dimensions of the grit chamber.
   Provide two identical grit chambers for independent operation.

Maximum design flow through each chamber	= (1.321 m^3/s)/2
	= 0.661 m^3/s
Volume of each chamber for 4-min detention period	= 0.661 m^3/s × 4 min × 60 s/min
	= 158.6 m^3
Provide average water depth at midwidth	= 3.65 m
Provide freeboard	= 0.8 m
Total depth of the grit chamber	= 4.45 m
Surface area of the chamber	= 158.6 m^3/3.65 m
	= 43.5 m^2
Provide length to width ratio	= 4:1
Length of the chamber	= 13.0 m
Width of the chamber	= 3.5 m
Surface area provided	= 45.5 m^2

2. Select diffuser arrangement.
   Locate diffusers along the length of the chamber on one side and place them 0.6 m above the bottom. The upward draft of the air will create a spiral roll action of the liquid in the chamber. The chamber bottom is sloped toward a collection channel located on the same side as the air diffusers. A screw conveyor is provided to move the grit along the channel length to a hopper at the downstream end.

3. Check actual detention period.

$$\text{Actual detention time at peak design flow when both chambers are in operation} = \frac{3.5\text{ m} \times 3.65\text{ m} \times 13.0\text{ m}}{0.661\text{ m}^3/\text{s} \times 60\text{ s/min}} = 4.2\text{ min}$$

$$\text{Actual detention time at peak design flow when one chamber is in operation} = \frac{3.5\text{ m} \times 3.65\text{ m} \times 13.0\text{ m}}{1.321\text{ m}^3/\text{s} \times 60\text{ min}} = 2.1\text{ min}$$

Step B: Design of Air Supply System

1. Determine air requirements.
Provide air supply at a rate of 7.8 L/s per meter length of the chamber.

$$\text{Theoretical air required per chamber} = 7.8\text{ L/s} \cdot \text{m} \times 13.0\text{ m} = 101.4\text{ L/s}$$

Provide 150 percent capacity for peaking purposes

$$\text{Total capacity of the diffusers} = 1.5 \times 101.4\text{ L/s} = 152.1\text{ L/s per chamber}$$

$$\text{Blower capacity (both chambers)} = 152.1\text{ L/s} \times 2 \times 60\text{ s/min} \times \frac{1}{1000\text{ L/m}^3} = 18.3\text{ standard m}^3\text{/min (s m}^3\text{/min)}$$

Provide two blowers 20 s m^3/min each, with one blower used as a standby unit. Air piping shall deliver a minimum of 0.15 m^3/s air to each chamber. Control valves and flow meters shall be provided on all branch lines in each basin to balance the air flow.

1. Design the diffusers and blowers.
Provide coarse diffusers with air pipe headers and hanger feed pipes having swing joint assembly. The procedure for designing the diffusers, pipings, and blower is covered in detail in Chapter 13. Similar procedure should be used to design the air supply system for the aerated grit chambers.

Step C: Surface Rise Rate

1. Check overflow rate when both chambers are in operation.

$$\text{Surface area of each chamber} = 3.5\text{ m} \times 13.0\text{ m} = 45.5\text{ m}^2$$

$$\text{Overflow rate (surface rise rate)} = \frac{0.661\text{ m}^3/\text{s} \times 86400\text{ s/d}}{45.5\text{ m}^2} = 1255.2\text{ m}^3/\text{m}^2 \cdot \text{d (30,805 gpd/ft}^2)$$

This value is considerably less than the allowable overflow rate in a velocity-controlled channel to remove 65-mesh (0.21-mm-diameter) grit (see Sec. 11-4-1).

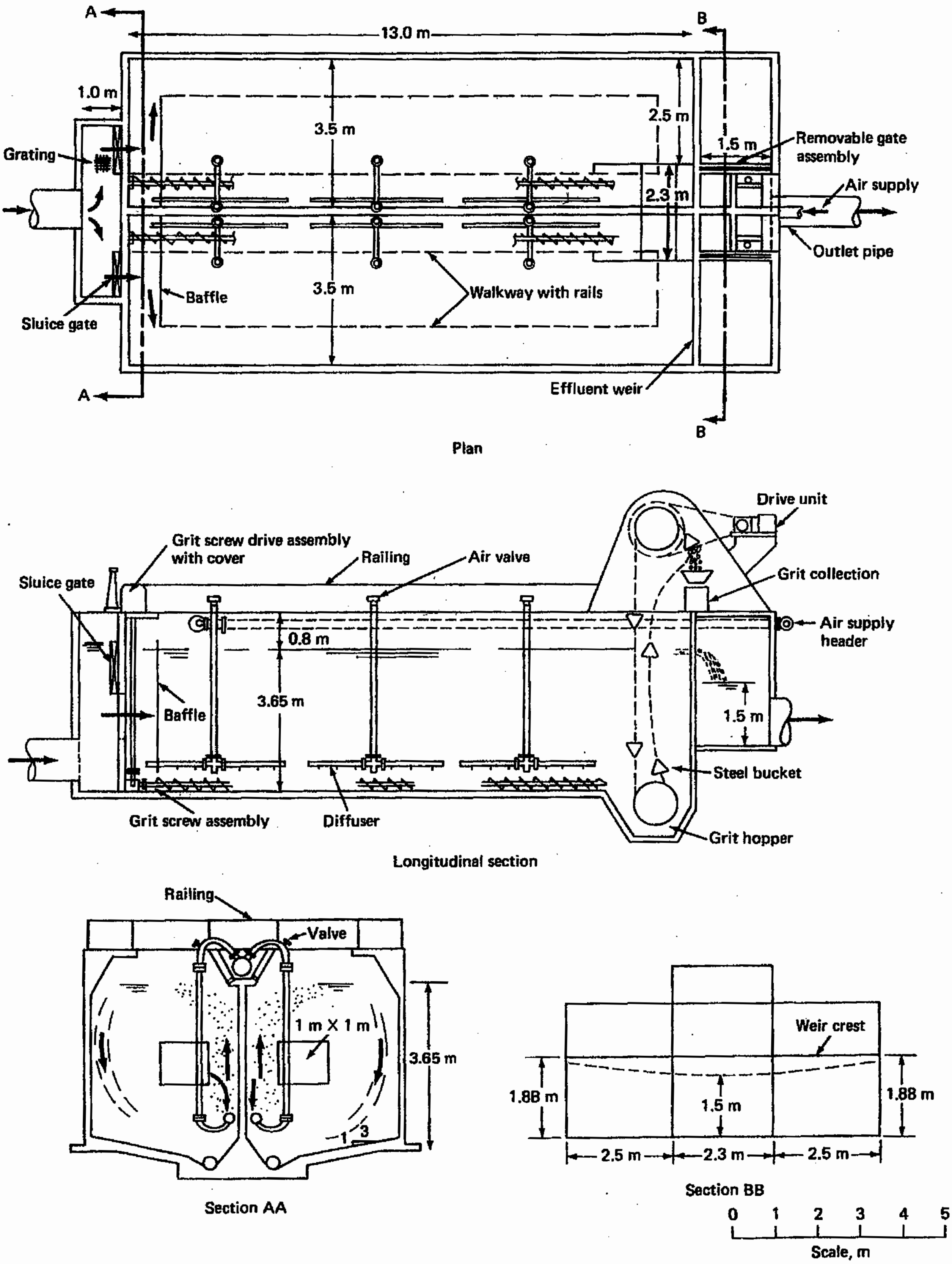


Figure 11-6 Design Details of Aerated Grit Chamber.

2. Check overflow rate when one chamber is out of service.

$$\text{Overflow rate (surface rise rate)} = 2 \times 1255.2 \text{ m}^3/\text{m}^2 \cdot \text{d}$$
$$= 2510.4 \text{ m}^3/\text{m}^2 \cdot \text{d}$$

Step D: Influent Structure

1. Select the arrangement of the influent structure.
   Provide 1-m-wide submerged influent channel that diverts the flow to two grit chambers. Each channel has one orifice 1.0 × 1.0 m that discharges the flow near the diffuser area. Provide a baffle at the influent to divert the flow transversally to follow the circulation pattern. Sluice gates are provided to remove one chamber from service for maintenance purposes. The details of the influent structure are shown in Figure 11-6.[a]
2. Calculate the head losses through the influent structure.
   The head loss through the influent structure is calculated by applying energy Equation [Eq. (8-4)] at sections (1) and (2) as defined in Figure 11-7. The energy equation is simplified in the form of Eq. (11-9).

$$\Delta H = \frac{v_2^2}{2g} - \frac{v_1^2}{2g} + h_L \tag{11-9}$$

where

v_1 = average velocity into the influent channel
v_2 = average velocity into the grit chamber
ΔH = the difference in elevation of free water surface into the channel and the chamber
h_L = combined head losses into the influent channel and the exit loss through the influent port. Since the head loss in the influent channel and velocity head difference is small, h_L is approximately equal to the piezometric difference across the influent port; therefore, in Eq. (11-10), h_L can be substituted for Δz.[14-16]

$$Q = C_d A \sqrt{2g\Delta z} \tag{11-10}$$

where

A = area of the orifice, m^2
C_d = coefficient of discharge = 0.61 for square-edged entrance[b]

Generally, v_1 and v_2 are small and the factor $[(v_2^2/(2g) - (v_1^2/2g)]$ is ignored. In the present situation for illustration purposes, these velocities, at peak design flow when only one chamber is in operation, are calculated below:

$$v_1 = \frac{1.321 \text{ m}^3/\text{s}}{1 \text{ m (channel width)} \times 4.06 \text{ m (assumed water depth in the channel)}}$$
$$= 0.33 \text{ m/s}$$

[a]An alternative arrangement is to provide a division box with two weirs to divide flow into each chamber.

[b]A conservative value is selected since slime growth may restrict the orifice area.

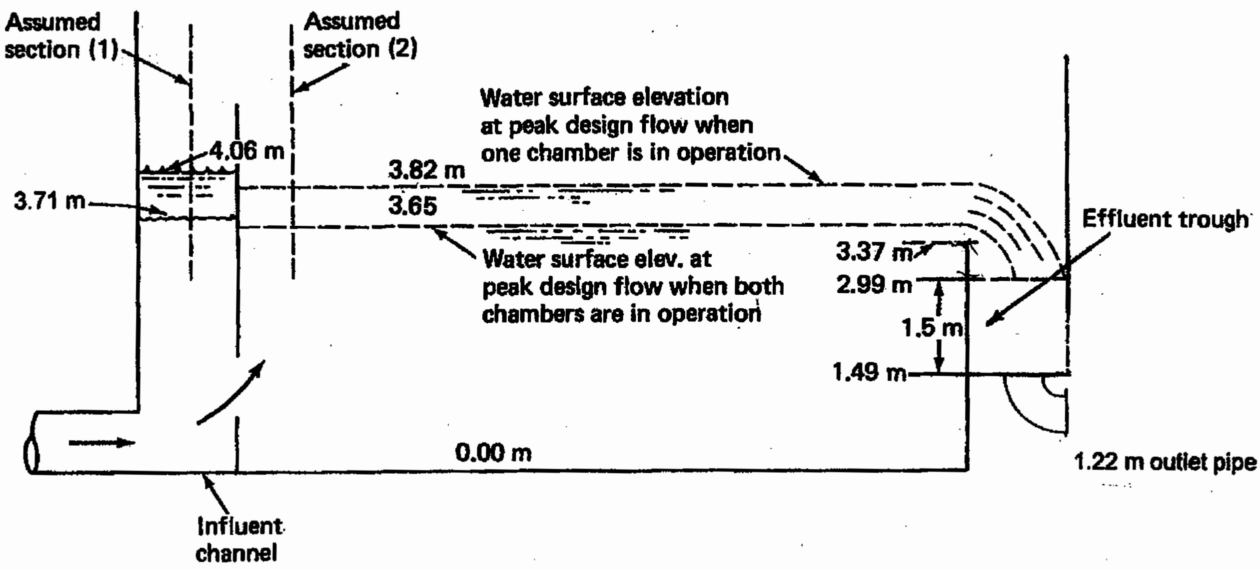


Figure 11-7 Hydraulic Profile Showing Differential Liquid Levels in the Influent Channel, Grit Chamber, and Effluent Trough at Peak Design Flow When One Chamber Is in Operation. All Elevations Are with Respect to the Assumed Datum at the Midwidth of the Chamber Bottom.

$$v_2 = \frac{1.321\ \text{m}^3/\text{s}}{3.5\ \text{m (chamber width)} \times 3.82\ \text{m (assumed water depth in the chamber)}}$$

$$= 0.10\ \text{m/s}$$

$$\frac{v_2^2}{2g} - \frac{v_1^2}{2g} = \frac{(0.10\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} - \frac{(0.33\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2}$$

$$= -0.005\ \text{m}$$

Equations (11-9) and (11-10) are therefore combined into one equation:

$$\Delta H = \frac{v_2^2}{2g} - \frac{v_1^2}{2g} + \left(\frac{Q}{C_d A \sqrt{2g}}\right)^2$$

At peak design flow of 1.321 m^3/s, when one chamber is in operation,

$$\Delta H = -0.005 + \left[\frac{1.321\ \text{m}^3/\text{s}}{0.61 \times 1\ \text{m} \times 1\ \text{m} \times \sqrt{2 \times 9.81\ \text{m/s}^2}}\right]^2 = 0.24\ \text{m}$$

If a larger influent port is selected, ΔH will be smaller.

Step E: Effluent Structure

1. Select an arrangement of the effluent structure.

   The effluent structure consists of a rectangular weir, an effluent trough, an effluent box, and an outlet pipe. The effluent weir is 2.5 m long, and the effluent trough is 2.5 m long × 1.5 m wide. The effluent box is common to both chambers and is

1.5 m × 2.3 m.[c] Removable gates are provided at the effluent box to drain the effluent trough when one chamber is removed from service. The outlet pipe carries flow to a collection-division box that precedes the primary sedimentation basin. Details of the effluent structure are given in Figure 11-6.

2. Compute the head over the effluent weir at average design flow when both chambers are in operation.
   The head over the rectangular weir is calculated from the weir equation [Eq. (11-11)]:[14,15]

$$Q = \frac{2}{3} C_d L' \sqrt{2gH^3} \tag{11-11}$$

where

Q = flow over weir, m^3/s
H = head over weir, m
C_d = coefficient of discharge, assume $C_d = 0.6$ (conservative value)
$L' = L - 0.1\,nH$
L = length of weir = 2.5 m
n = number of end contractions, in this case $n = 1$

At peak design flow when both chambers are in operation, $Q = 0.661\ m^3/s$ and H is calculated by trial and error. Assume:

$$L' = 2.47\ m$$

$$H = \left[\frac{0.661\ m^3/s \times 3/2}{0.6 \times 2.44\ m \times \sqrt{2 \times 9.81\ m/s^2}}\right]^{2/3}$$

$$= 0.28\ m$$

$$L' = 2.5\ m - 0.1 \times 0.28\ m = 2.47\ m \text{ (same as the initial assumption)}$$

3. Compute the height of the weir crest above the bottom of the chamber.

$$\text{Height of the weir crest} = 3.65\ m - 0.28\ m = 3.37\ m$$

4. Compute the head over the effluent weir at peak design flow when one chamber is out of service.

$$\text{Peak design flow} = 1.321\ m^3/s$$

Assume:

$$L' = 2.46\ m$$

$$H = \left[\frac{1.321\ m^3/s \times 3/2}{0.6 \times 2.46\ m \times \sqrt{2 \times 9.81\ m/s^2}}\right]^{2/3}$$

$$= 0.45\ m$$

$$L' = 2.5\ m - 0.1 \times 0.45\ m = 2.46\ m \text{ (same as the initial assumption)}$$

[c]2.3 m length of the effluent box includes 0.3 m thickness of the wall common in both chambers.

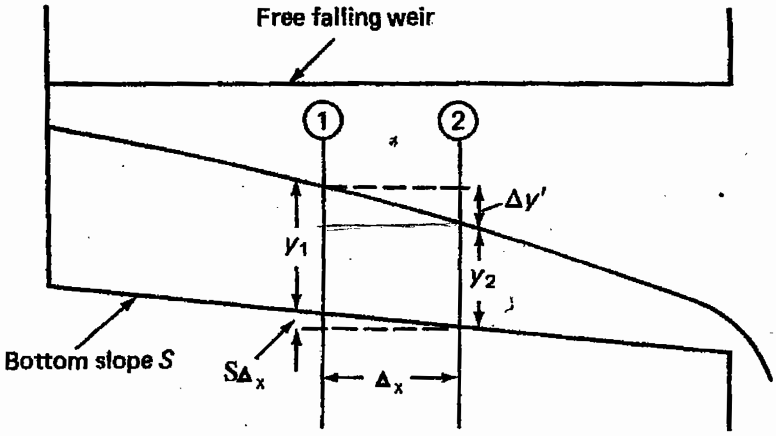


Figure 11-8 Water Surface Elevation in a Flume Receiving Weir Discharge.

5. Compute the water depth in the chamber at peak design flow when one chamber is out of service.

$$\text{Water depth} = \begin{pmatrix}\text{height of weir crest above} \\ \text{the bottom of the chamber}\end{pmatrix} + \text{head over weir}$$

$$= 3.37 \text{ m} + 0.45 \text{ m} = 3.82 \text{ m}$$

6. Calculate the depth of the effluent trough.
A free-falling weir discharging into a flume, trough, or launder has varying flow throughout its entire length. At the discharge point the flow is maximum. The water surface profile is shown in Figure 11-8. For uniform velocity distribution, Chow expressed the drop in the water surface elevation between sections 1 and 2 by Eq. (11-12):[16]

$$\Delta y' = \frac{q_1 v_{avg}}{g q_{avg}} \left[\Delta v + \frac{v_2}{q_1} \Delta q \right] + (S_E)_{avg} \Delta x \qquad (11\text{-}12)$$

where

$\Delta y'$ = drop in water surface elevation between sections 1 and 2, m (ft)
y_1 and y_2 = depth of flow at sections 1 and 2, respectively, m (ft)
q_1 and q_2 = discharge at sections 1 and 2, respectively, m^3/s (ft^3/s)
v_1 and v_2 = velocity at sections 1 and 2, respectively, m/s (ft/s)
Δv = $(v_2 - v_1)$, m/s (ft/s)
Δq = $q_2 - q_1$, m^3/s (ft^3/s)
Δx = horizontal distance between sections 1 and 2, m (ft)
$(S_E)_{avg}$ = average slope of the energy line, m/m (ft/ft). It is obtained from Eqs. (11-13) or (11-14).
v_{avg} = average velocity $(v_1 + v_2)/2$, m/s (ft/s)
q_{avg} = average discharge $(q_1 + q_2)/2$, m^3/s (ft^3/s)

$$(S_E)_{avg} = \frac{n^2 (v_{avg})^2}{(R_{avg})^{4/3}} \quad \text{(SI units)} \tag{11-13}$$

$$(S_E)_{avg} = \frac{n^2 (v_{avg})^2}{2.21 (R_{avg})^{4/3}} \quad \text{(U.S. customary units)} \tag{11-14}$$

where

$$R_{avg} = (R_1 + R_2)/2$$

The terms n and R are defined in Eq. (7-1).

The computational procedure for obtaining the depth of flow in the trough at the upstream section is given below:[17]

a. Determine the depth of flow at the lower end of the trough, which is generally fixed by the downstream control conditions.
b. Select an incremental distance Δx between two sections for computational purpose. Δx may be a constant or may be varied.
c. Take the first increment Δx at the lower end of the trough having water depths of y_2 and y_1 at lower and upper ends of the increment.
d. Assume $\Delta y'$ for the first increment.
e. Compute Δy from Eq. (11-15).

$$\Delta y = S\Delta x - \Delta y' \tag{11-15}$$

f. Compute the depth y_2 from Eq. (11-16).

$$y_1 = y_2 - \Delta y \tag{11-16}$$

g. Determine discharge q_2 and q_1 below and above the selected incremental distance Δx.
h. Compute velocity v_2 and v_1.
i. Use Eq. (11-12) to compute $\Delta y'$.
j. If the computed value of $\Delta y'$ in step i is not within a certain tolerance level set by the designer, repeat steps d through i.
k. After balancing the two sections, repeat steps c through j. At this time the computed depth y_1 at the upper end of the increment is replaced by the water depth y_2 at the lower end of the next selected increment.

Benefield et al. provided a computational scheme and solution of water surface profile for a lateral spillway channel receiving uniformly distributed flow along the entire channel length.[17] A similar solution using Eqs. (11-10)–(11-14) for point load discharges along the effluent channel is presented in Chapter 13 (Table 13-8). A computer program is given in Ref. 9.

The use of the above procedure is tedious and time-consuming. A widely used practice by the designers is to utilize an approximate solution given by Eq. (11-17). This equation was originally developed for flumes with level inverts and parallel sides; channel friction is neglected, and the draw-down curve is assumed to be parabolic.[17,18]

$$y_1 = \sqrt{{y_2}^2 + \frac{2Q^2}{gb^2y_2}} \tag{11-17}$$

where

y_1 = water depth at the upstream end of the trough, m
y_2 = water depth in the trough at a distance L from the upstream end, m
q = discharge in the trough at a distance L from the upstream end, m^3/s
b = width of the launder, m
g = acceleration due to gravity, m/s^2

At peak design flow, when one chamber is out of service, the following conditions apply:

$Q = 1.321\ m^3/s$ (flow in the trough at the lower end of the trough or $L = 2.5$ m from the upper end)

The water surface elevation in the effluent box is governed by the downstream conditions. Assume water depth in the effluent box at the exit point (center of the effluent pipe) is 1.5 m; therefore, the water depth in the trough at the effluent box y_2 is also 1.5 m[d] (see Chapter 21).

$$y_1 = \sqrt{(1.5\ \text{m})^2 + \frac{2 \times 1.321^2}{9.81\ \text{m/s}^2 \times (1.5\ \text{m})^2 \times 1.5\ \text{m}}} = 1.54^{\text{e}}\ \text{m}$$

Allow 12 percent additional depth to account for friction losses, and add 15 cm to ensure a free fall. Total depth of the effluent trough = 1.54 m × 1.12 + 0.15 m = 1.88 m. The flow is conveyed from the effluent box to a collection box upstream of the sedimentation basin. The hydraulic calculations for pipe sections between the grit chambers and division box preceding the primary sedimentation basin are given in Chapter 21.

Step F: Head Loss Through the Grit Chamber. The total head loss through the grit chamber includes (1) head loss at the effluent structure, (2) head loss at the influent structure, (3) head loss in the basin, and (4) head loss because of baffles.

1. Compute head loss at the influent and effluent structures.
   The head losses at the influent and effluent structures were calculated earlier in steps D and E.

[d]From Eq. (8-5) the critical depth is 0.43 m, which is considerably less than the 1.5-m depth provided at the control point. This presents a submerged outfall condition. Submerged outfalls reduce the stripping or odorous gases and are often preferred by the designers.

[e]The water depth at the upstream end of the effluent launder from Eqs. (11-12)–(11-16) is 1.528 m (see Problem 11-16).

2. Compute the head loss in the grit chamber.
   The head loss in the basin is small because of short length and small velocity of flow; therefore, it can be neglected.
3. Compute the head loss due to influent and effluent baffles.
   The influent and effluent baffles offer obstruction to the flow in the grit chamber. The momentum equation [Eq. (11-18)] is used to calculate the head loss due to the baffles.[19,20]

$$h_L = C_D \frac{v_2^2}{2g} \frac{A_b}{A} \tag{11-18}$$

where

h_L = head loss due to baffle
v_2 = horizontal component of the velocity in the chamber through the unobstructed area
A_b = vertical projection of the area of the baffle
A = cross-sectional area of the chamber
C_D = coefficient of drag. The value of C_D for flat plates is approximately 1.9.[1,16,17]

If influent baffles occupy 50 percent of the cross-sectional area of the chamber, h_L at peak design flow, when one chamber is out of service, is calculated as follows:

$$\text{Velocity through the chamber} = \frac{Q}{\text{area}} = \frac{1.321\ \text{m}^3/\text{s}}{(3.5\ \text{m width})\ (3.82\ \text{m water depth})}$$

$$= 0.099\ \text{m/s}$$

$$h_L = \frac{1.9\ (0.099\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} \times \frac{0.5}{1} = 0.0005\ \text{m}$$

This is a small head loss and can be neglected. Similarly, the head loss caused by the effluent baffle can also be calculated. The value will be small and can also be ignored.

Step G: Hydraulic Profile Through the Chamber. The hydraulic profile through the grit chamber at peak design flow when one chamber is out of service is shown in Figure 11-7. The hydraulic profile is with respect to the assumed datum at the bottom of the chamber. The velocity head caused by the approach velocity at the effluent weir is ignored.

Step H: Effluent Quality. Overall BOD_5 and total suspended solids removal in a grit chamber is small. It is therefore assumed that the concentrations of BOD_5 and total suspended solids in the effluent from the grit removal facility are the same as those in the raw wastewater (BOD_5 = 250 mg/L and TSS = 260 mg/L). However, the effluent from the aerated grit chamber is freshened by the air and dissolved oxygen; thus, some reduction in odors may occur.

Step I: Quantity and Characteristics of Grit. The quantity of grit that will be removed from the wastewater will vary greatly. Using the range and typical values given in Sec. 11-6, the following quantities are obtained:

$$\text{Average quantity of grit} = 30\ \text{m}^3/10^6\ \text{m}^3 \times 0.44\ \text{m}^3/\text{s} \times 86400\ \text{s/d} = 1.14\ \text{m}^3/\text{d}$$

The minimum and maximum quantities may range from 0.2 to 7.6 m^3/d.

The grit removal by the bucket elevator provides a certain degree of grit washing due to agitation in wastewater during the ascent. However, it is expected that the grit will contain 3–4 percent putrescible organic matter.

Step J: Grit Disposal. Combined disposal of grit, screenings, and unused digested sludge will be achieved by sanitary landfilling. Environmental considerations (air, water, land, aesthetics), along with the design and operation of sanitary landfills, are discussed in Chapter 19.

Step K: Design Details. The design details of the aerated grit removal facility are provided in Figure 11-6.

11-11 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT AERATED GRIT REMOVAL FACILITY

Operation and maintenance of aerated grit removal facility requires well-trained operators familiar with the peculiarities of sewer system and characteristics of wastewater.

It is important that grit removal is conducted at the highest efficiency level possible so that the wear on downstream equipment is minimum. The operator must adjust the air flow to allow the grit to settle but must also provide enough air to prevent organic material from settling. If short-circuiting is noticed, submerged baffles must be installed. The design of a grit removal system allows one unit to be taken out of service routinely for maintenance, without impairing the grit removal performance at peak design flow. Also, the swing-type diffusers will allow maintenance of aeration equipment without taking the units out of service. The following items should be considered as a troubleshooting guide:[21,22]

1. Rotten egg odor is an indication of hydrogen sulfide formation. Increase the aeration; inspect the walls, channels, and the chamber for debris. Wash walls, weir, and influent and effluent channels with chlorine or hydrogen peroxide solution.
2. Corrosion or wear on equipment indicates inadequate ventilation and the production of hydrogen sulfide. Increase the air supply, and stop operation for routine maintenance and painting.
3. Increase air supply if the grit that is removed is grey in color, smells, and feels greasy.
4. If surface turbulence is reduced, diffusers may be covered by rags or grit. Cleaning of diffusers is needed.
5. Low recovery of grit is an indication of excessive aeration, and inadequate retention time. Reduce air supply.

6. Overflowing grit chamber indicates a pump surge problem. Adjust the pump controls.
7. Percent volatile matter should be determined regularly to obtain percent organics in the grit. High organics indicate low aeration.

The grit contains putrescible organics and therefore can cause serious odor problems if not properly handled. Daily disposal of removed grit will reduce odor and insect problems. The area must be washed daily with chlorine or hydrogen peroxide solution.

11-12 SPECIFICATIONS

The specifications for the grit removal facility designed in this chapter are briefly presented below. The purpose of these specifications is to describe many components that could not be fully covered in the Design Example. These specifications are neither complete nor definitive. Detailed specifications should be prepared in consultation with the equipment manufacturers. The following specifications should be used only as a guide.

11-12-1 General

Each aerated grit chamber shall comprise a complete assembly of concrete basin, influent and effluent structures, air diffusers and aeration equipment, grit collection by spiral conveyor, and bucket elevator for grit removal. The manufacturer shall furnish and deliver, ready for installation, a grit collection and removal mechanism suitable for installation in two identical rectangular grit chambers of the following dimensions:

Number of identical chambers with common wall	= 2
Length	= 13.0 m (42.7 ft)
Width	= 3.5 m (11.5 ft)
Water depth at midwidth	= 3.65 m (12.0 ft)
Bottom slope along width (toward spiral conveyor)	= 3 horizontal : 1 vertical
Freeboard	= 0.8 m (2.5 ft)

11-12-2 Materials and Fabrication

All structural steel shall conform to the *Standard Specifications for Structural Steel for Buildings* of the American Society for Testing and Materials. All iron castings shall be tough, closed-grained, and free from blow holes, flaws, or excessive shrinkage.

11-12-3 Aeration Equipment

Air Diffusion Equipment. The air diffusion equipment in the aerated grit chamber includes air header piping, flow measurement, air control valve, riser pipe, air diffusers, and necessary supports and brackets. The air diffusers shall be nonmetallic, coarse bubble, fixed to a horizontal stainless steel tube, and connected to a stainless steel tube downcomer. The diffusers shall be capable of delivering at least 150 L/s air per chamber. The air diffusion system shall furnish a range of 3–12 L per second per meter of the basin measured along the longitudinal axis. The system shall be field-tested by filling each chamber with water and visually observing the operation of the air tubes.

Air Blowers. The air blower shall have a capacity not less than 20 standard m^3/min when operating against a pressure of 27.6 kN/m^2 (gauge) (4.0 psig) measured at the blower outlet. There will be two identical blowers, one used as a standby. The blowers shall be of a heavy-duty, centrifugal type having end suction and top discharge. The blowers shall be directly connected to a 1.5-kW (minimum), 440-V, 3-phase, 60-cycle motor through a flexible coupling. Motor and blower shall be mounted on an integral base plate. The blower's magnetic-motor starter shall have 3-phase overload protection, a cover-mounted, hands-off-automatic selector switch, and circuit breakers.

11-12-4 Conveyor and Elevator

The spiral conveyor shall be provided to move the grit into the hopper. The helix shall be made up from preformed heavy-steel flight sections welded to the shaft and fitted with replaceable wearing shoes. A plate and shroud shall be provided at the discharge end of the spiral to limit the feed to the bucket elevator and thus prevent overfeeding or stalling. The bearings shall be babbitted, and those above the water line shall be provided with suitable lubrication fittings. The head shaft shall be equipped with take-up bearings with an adjustment of specified tolerance.

The drive unit shall consist of an all-motor-type helical gear reducer. Gears shall be made of steel with teeth cut to accurate shape, enclosed in a moisture- and oil-proof case. The driving mechanism shall be enclosed in a suitable guard. The motor shall be totally enclosed with moisture-proof impregnated windings, and shall be 0.5 kW, 440 V, 3 phase, and 60 cycle.

The bucket elevator shall consist of a single strand of chain passing over the top and bottom sprockets, with steel buckets attached to the chain, driven by a motor and speed reducer. The elevator shall be the continuous bucket type with the lower bottom shaft extended for driving the spiral conveyor. The speed shall be approximately 1.5 m/min. The elevator above the top of the grit chamber shall be completely enclosed and shall be complete with discharge spout. The capacity of the system shall be approximately 0.5 m^3/h. The motor starter for the grit elevator is similar to that of the blower as specified above.

11-12-5 Painting

All submerged metal parts (except bearing surfaces, shafts, and chain) shall be painted with one shop coat of metal primer (rust-resistant). The bearing surfaces and shaft shall be coated with grease. The chain shall be dipped in a rust-inhibiting compound. All anchor bolts shall be hot-dipped galvanized steel.

11-13 PROBLEMS AND DISCUSSION TOPICS

11-1 Review proper sections in Chapters 11, 12, 13, and 16 and describe four types of settling (discrete, flocculant, hindered or zone, and compression) and their occurrence in various wastewater treatment units.

11-2 Newton's and Stokes' equations for settling of discrete particles are given by Eqs. (11-2) and (11-5). Develop these equations by equating gravitational force $(\rho_s - \rho)\, gV$ and frictional drag force $C_D A\, \rho v^2/2$ where A and V are cross-sectional area and volume of spherical particles. Other terms are defined with Eqs. (11-1)–(11-4).

11-3 A grit particle with $S_S = 2.65$ falls through water having kinematic viscosity $\nu = 1.1306 \times 10^{-6}$ m²/s. The diameter of the particle is 0.3 mm. Calculate the settling velocity.

11-4 The terminal velocity or overflow rate is expressed by Eq. (11-8). Using the dimensions length (L), width (W) and liquid depth (H) prove that $v_t = Q/(L \times W)$. What is the effect of depth on performance of the settling basin when detention time is kept constant? Q = flow through the basin. For notations and definitions of terms see Figure 11-1(a).

11-5 A grit channel is designed for a hydraulic loading of 1630 m³/m²·d (40,000 gpd/ft²). A sieve analysis was performed on a sample of dried grit. The average settling velocity of each fraction in the column test was experimentally determined. The results of the sieve analysis and column test are given below. Draw the fraction removal curve, and calculate the theoretical removal efficiency of the grit chamber.

Fraction retained on sieve	0.45	0.15	0.10	0.15	0.10	0.05
Average velocity from column test, m/min	3.05	1.53	0.61	0.31	0.23	0.15

11-6 A settling column analysis was run on a discrete suspension. Water samples were withdrawn at different time intervals from a port 2 m below the original water level. The total suspended solids (TSS) analysis was performed on each sample withdrawn. The results of the TSS analysis are provided below. Calculate the theoretical removal efficiency of a sedimentation basin that has an overflow rate of 30 m³/m²·d.

Time, min	0	60	90	120	180	260	400
Concentration, mg/L	250	153	148	138	120	70	24

11-7 Design a velocity-controlled grit channel for the Design Example. The maximum and average design flows are 1.321 and 0.440 m³/s, respectively. Detention time is 1 min, and maximum velocity through the channel is 0.34 m/s. If channel width is 1.83 m, calculate the water depth in the channel, and design the proportional weir using the procedure given in Chapter 8. The weir crest is 15.0 cm above the channel floor.

11-8 Design a velocity-controlled grit channel. The constant velocity through the channel is 0.3 m/s. The channel cross section is parabolic with $A = 2/3\ HT$, where A = cross-sectional area, H = depth, and T = top width. In the final design the parabolic section of the channel is approximated with straight lines. The peak wet weather, maximum dry weather, and average and minimum flows expected through the grit channel are 0.66, 0.44, 0.22, and 0.10 m³/s. The outlet section of the grit channel is rectangular with vertical sides and a well-rounded and smooth approach, so that the head loss may be assumed equal to 10 percent of the velocity head. The flow in the control section will be at critical depth. The top width of the parabolic section of the channel is kept at 2 m at maximum dry weather flow of 0.44 m³/s.

11-9 Design an aerated grit chamber for a municipal wastewater treatment plant. The average flow is 0.44 m³/s and peaking factor is 2.5. The detention time in the chamber at peak flow is 3.5 min. The width to depth ratio is 1.2:1 and depth is 3.2 m. The air supply is 10 L/s per m length of the basin. Also, estimate the average quantity of the grit that must be handled at the facility. Use typical values given in Sec. 11-6.

11-10 Calculate the dimensions of an aerated grit chamber treating a wastewater flow of 0.5

m^3/s. Detention time is 4 min, depth = 3 m, and the $L:W$ ratio is 5:1. Also calculate total air supply if aeration rate is 12 L/s per meter of the tank length. Also estimate average quantity of grit collected.

11-11 Develop the hydraulic profile through the grit chamber in the Design Example at an average design flow of 0.44 m^3/s when both chambers are in operation.

11-12 A weir trough is 10 m long and 1 m wide. The weir crest is on one side of the trough and covers the entire length of the trough. Calculate the depth of the trough if discharge through the basin is 0.3 m^3/s. Depth of flow at the lower end of the trough is 0.9 m. Assume friction loss is 15 percent of the depth of water at the upper end, and freefall allowance is 6 cm. Use Eq. (11-17).

11-13 List the necessary information needed by a design engineer to start the calculations for design of a grit removal facility.

11-14 List the advantages of providing a grit removal facility. Under what conditions would you prefer a velocity-controlled grit channel over an aerated grit chamber?

11-15 Determine the total head loss through a 4-m-wide grit chamber. The details of the grit chamber are given below:

(a) The influent channel is 1.5 m wide and has one submerged orifice 1.5 m × 1.5 m. The invert of the influent channel is 0.5 m above the floor of the chamber. The depth of floor in the chamber is 3.0 m.

(b) The head loss in chamber is small and can be ignored.

(c) The influent and effluent baffles occupy 65 and 60 percent of the cross-sectional area of the chamber, respectively.

(d) The flow through the grit chamber is 1.6 m^3/s.

(e) There is 0.4-m head loss into the effluent structure. This head loss is the difference in water surface elevations in the chamber at the effluent weir and the outlet box of the grit chamber.

Draw the hydraulic profile through the grit chamber.

11-16 Study the computational scheme of determining water surface profile in a flume using Eqs. (11-12)–(11-16). A solution procedure is given in Sec. 11-10-2, Step E, 6. Perform similar steps to determine the water depth in the effluent launder of grit removal facility in the Design Example. Use uniform increments of 0.5 m. Compare your result with that given in Sec. 11-10-2, Step E, 6.

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12

Primary Sedimentation

12-1 INTRODUCTION

The purpose of primary sedimentation (or clarification) is to remove the settleable organic solids. Normally, a primary clarification facility will remove 50–70 percent total suspended solids and 30–40 percent BOD_5. Primary clarification is achieved in large basins under relatively quiescent conditions. The settled solids are collected by mechanical scrapers into a hopper, from which they are pumped to a sludge-processing area. Oil, grease, and other floating materials are skimmed from the surface. The effluent is discharged over weirs into a collection trough.

The settling behavior of solids in a primary sedimentation basin is of a flocculant type as particles coalesce, mass increases, and particles settle faster. To determine the settling behavior, a settling column test may be needed.

Often, chemicals are added to enhance the removal efficiency of plain sedimentation. The colloidal particles are flocculated and removed with the settling floc. These particles would otherwise not be removed in plain sedimentation. Sometimes, chemicals are also added to bring about the precipitation of dissolved constituents such as soluble organics, phosphorus, and heavy metals.

The purpose of this chapter is to present (a) the theory of flocculant settling, (b) types of sedimentation basins, (c) basic design factors, and (d) theory and design of enhanced sedimentation. Additionally, the step-by-step design procedure, equipment selection, design details, facility operation, and equipment specifications for a plain primary sedimentation facility are covered in the Design Example.

12-2 FLOCCULANT SETTLING (TYPE II)

Unlike discrete settling, flocculant settling has no mathematical basis. Also, unlike discrete settling, detention time is an important parameter in the design of a flocculant settling basin.[a] Flocculant settling differs from discrete settling because the agglomeration

[a]A discussion on discrete settling (Type I) and governing mathematical equations has been presented in Sec. 11-3.

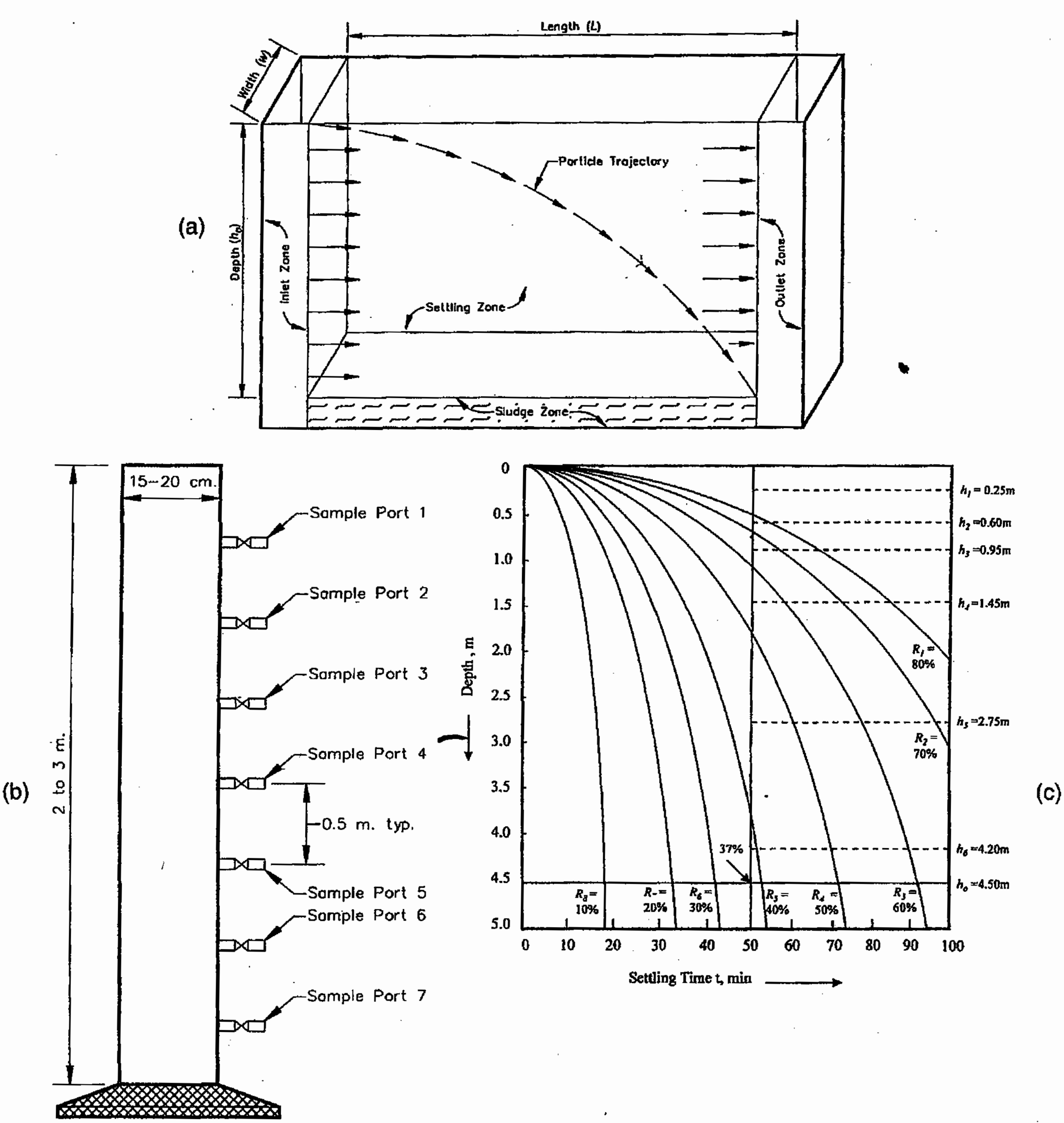


Figure 12-1 Flocculant Settling: (a) settling trajectories of flocculant particle in a sedimentation basin, (b) flocculant column for batch settling test, and (c) graph showing results of percent removal and isoremoval lines.

(flocculation or coalescence) of particles occurs as they collide. The resulting combined particles are heavier; as a result, settling velocity increases as shown in Figure 12-1(a). Because of unknown settling behavior of flocculant particles, no mathematical model has been developed. Design parameters are developed either by batch-type settling column tests or through experience with existing plants treating similar to wastewater. The batch settling tests are performed in laboratory settling columns similar to that shown in Figure 12-1(b). The suspension is thoroughly mixed and placed in the column to a desired depth. At certain time intervals (usually at 5 to 10 min) samples are withdrawn from different ports. The total suspended solids (TSS) concentration is determined for each sample. A test duration of 1 to 3 hours should yield sufficient data for design. Ideally, the test should be conducted twice to ensure repeatability of the results. The TSS concentration results are reduced to percent removals. A summary table with reduced results is generated, and a grid showing percent TSS removal at each port and at different time intervals is plotted. Lines of equal percentage removal or isoremoval are drawn [Figure 12-1(c)]. These lines also trace the maximum trajectories of particle settling paths for specific concentrations in a flocculant suspension. The overall percent removal of solids at a given detention time and depth of the column is determined from Eq. (12-1). The theoretical detention time and overflow rates are obtained from percent TSS removal efficiency curves. To account for less than optimum conditions encountered in the field, the design values of overflow rate and detention time are obtained by multiplying the corresponding theoretical values by factors of 0.65 to 0.85 and 1.25 to 1.50, respectively. The procedure for determining the design values of overflow rate and detention time from a settling column test is covered in Problem 12-1 (Sec. 12-11). For in-depth study on this subject readers should consult Refs. 1–4.

$$\text{Percent removal} = \frac{h_1}{h_0}(100 - R_1) + \frac{h_2}{h_0}(R_1 - R_2) + \ldots \frac{h_{n-1}}{h_0}(R_{n-1} - R_n) + R_n \tag{12-1}$$

where

$h_1, h_2 \ldots h_n$ = vertical distance from the top of the settling column to the midpoint between two consecutive lines of isoremoval at desired detention time [Figure 12-1(c)]

h_0 = total depth of settling column [Figure 12-1(c)]

$R_1, R_2, \ldots R_n$ = consecutive isoremoval curves (%)

As an example, the overall percent removal at 50 minutes detention time and at a settling column depth of 4.5 m is calculated as follows [Figure 12-1(c)]:

$$\begin{aligned}\text{Percent removal} &= \frac{0.25\text{m}}{4.5\text{m}}(100 - 80) + \frac{0.60\text{m}}{4.5\text{m}}(80 - 70) + \frac{0.95\text{m}}{4.5\text{m}}(70 - 60) \\ &+ \frac{1.45\text{m}}{4.5\text{m}}(60 - 50) + \frac{2.75\text{m}}{4.5\text{m}}(50 - 40) + \frac{4.20\text{m}}{4.5\text{m}}(40 - 37) + 37 \\ &= 1.1 + 1.3 + 2.1 + 3.2 + 6.1 + 2.8 + 37 \\ &= 53.6\end{aligned}$$

12-3 TYPES OF SEDIMENTATION BASINS

In general, the design of most of the sedimentation basins or clarifiers falls into several categories: (1) horizontal flow, (2) solids contact, (3) inclined surface, (4) stacked or two-tray, and (5) proprietary systems.

12-3-1 Horizontal Flow

In horizontal flow clarifiers, the velocity gradients are predominantly in the horizontal direction. The common types of horizontal flow clarifiers are rectangular, square, or circular. The selection of any shape depends on

1. Size of installation
2. Regulation and preference of regulatory authorities
3. Local site conditions
4. Preference, experience, and engineering judgment of the designer and plant personnel

The advantages and disadvantages of rectangular clarifiers over circular clarifiers are summarized below.

Advantages
1. Occupy less land area when multiple units are used
2. Provide economy by using common walls for multiple units
3. Easier to cover the units for odor control
4. Provide longer travel distance for settling to occur
5. Less short-circuiting
6. Lower inlet-outlet losses
7. Less power consumption for sludge collection and removal mechanisms

Disadvantages
1. Possible dead spaces
2. Sensitive to flow surges
3. Restricted in width by collection equipment
4. Require multiple weirs to maintain low weir loading rates
5. High upkeep and maintenance costs of sprockets, chain, and flights used for sludge removal

The details of rectangular and circular-horizontal flow clarifiers are illustrated in Figures 12-2 and 12-3.

12-3-2 Solids Contact

The solids contact clarifiers utilize the principle of solids contact.[5-7] The incoming solids are brought in contact with a suspended sludge layer near the bottom. The incoming solids rise and come in contact with the solids in the sludge layer. This layer acts as a blanket, and the incoming solids agglomerate and remain enmeshed within this blanket. The liquid rises upward while a distinct interface retains the solids below. These clarifiers have better hydraulic performance and have reduced detention time for equivalent solids removal in horizontal flow clarifiers.

Both circular and rectangular units are used for the solids contact clarifier. Solids contact clarifiers are efficiently used for chemical flocculant suspensions. These units are not suitable for biological sludges because long sludge-holding times may create undesirable septic conditions. The operational principles of solids contact clarifiers are illustrated in Figure 12-4(a).

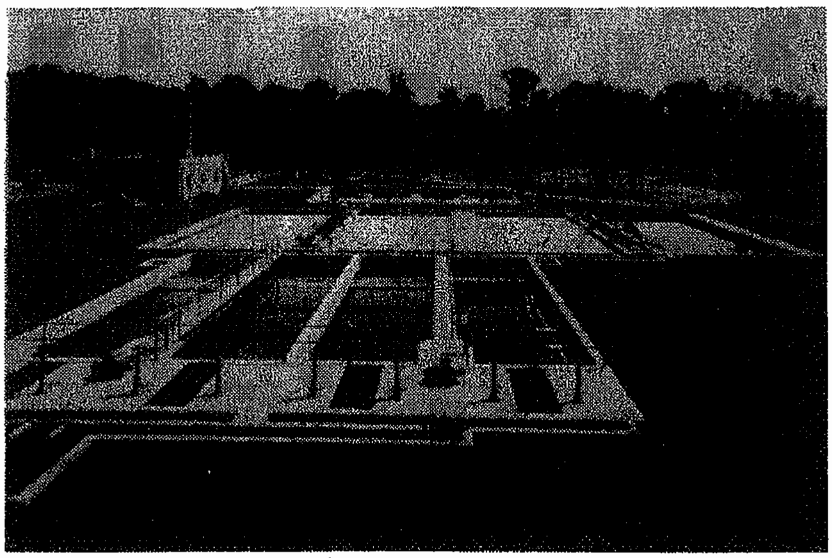
12-3-3 Inclined Surface

The inclined surface basins utilize inclined trays to divide the depth into shallower sections. Thus, the depth of fall of particles (and therefore the settling time) is significantly reduced. This concept is frequently used to upgrade the existing overloaded primary and secondary clarifiers. There are two design variations to the inclined surface clarifiers: tube settlers and parallel plate separators.

Tube Settlers. The inclined trays are constructed using thin-wall tubes. These tubes are circular, square, hexagonal, or any other geometric shape and are installed in an inclined position within a basin. The incoming flow enters these tubes and flows upward. The solids settle on the inside of the tubes and slide down into a hopper.

Parallel Plate Separators. The parallel inclined plate (also called lamella tube) separators have parallel trays covering the entire tank. The operational principles for parallel plate separators are the same as those for the tube settlers.

The inclined surface clarifiers provide a large surface area, thereby reducing the clarifier size. There is no wind effect, and the flow is laminar. Many overloaded horizontal flow clarifiers have been upgraded using this concept. The major drawbacks of inclined surface clarifiers include the following: (1) long periods of sludge deposits on inner walls may cause septic conditions; (2) effluent quality may deteriorate when sludge



(a)

Figure 12-2 Details of Horizontal Flow Rectangular Clarifiers: (a) four primary settling tanks preceding aeration tanks at a municipal wastewater treatment plant (courtesy FMC, U.S. Filter).

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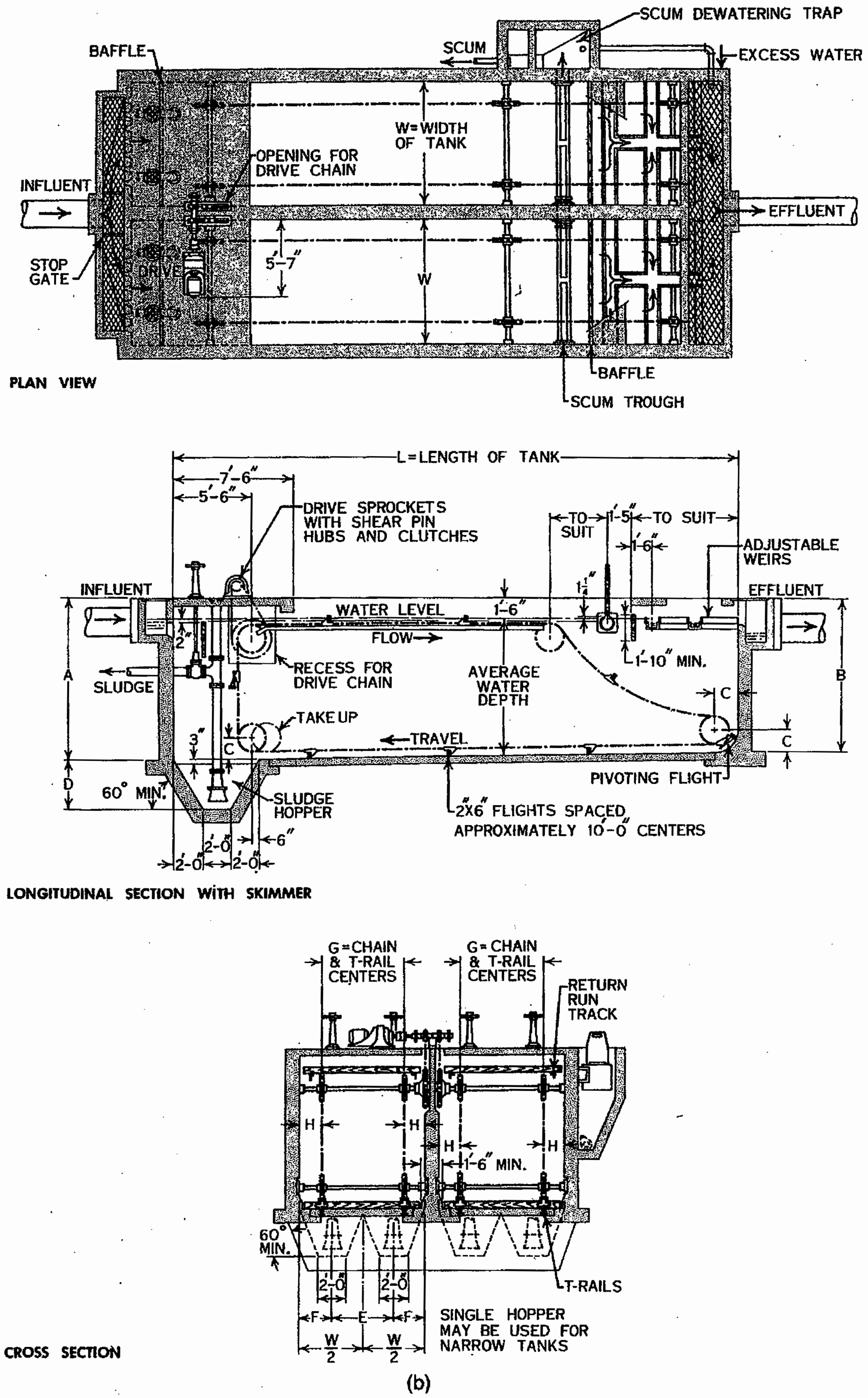


Figure 12-2—cont'd (b) plan, longitudinal and cross sections of a double rectangular sedimentation basin with skimmer (courtesy FMC, U.S. Filter).

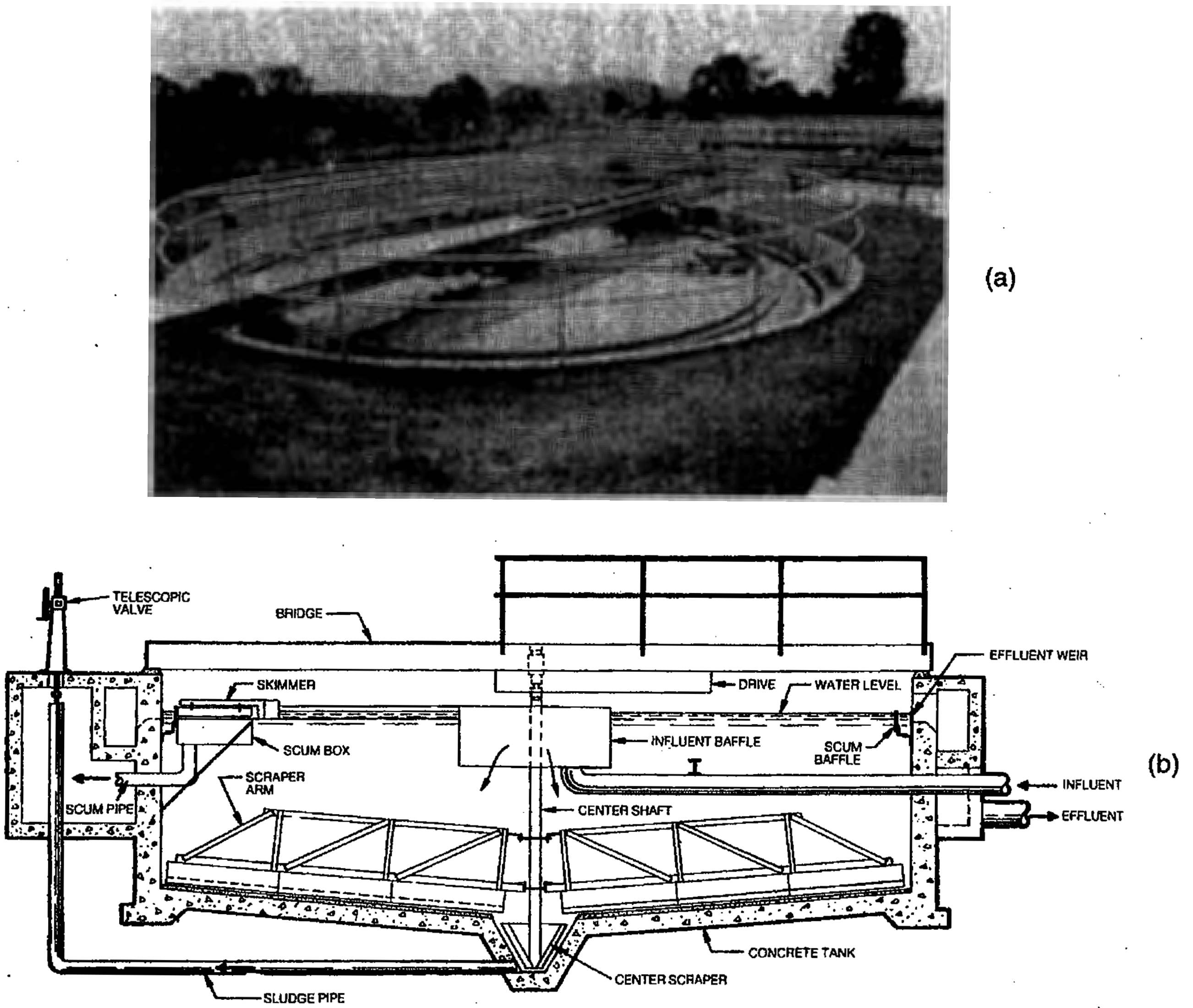


Figure 12-3 Details of Horizontal Flow Circular Clarifier: (a) one 9-m-diameter primary clarifier with skimmer and (b) sectional view of a circular clarifier with central feed (courtesy FMC, U.S. Filter).

deposits slide down; (3) there may be clogging of the inner tubes and channels; and (4) serious short-circuiting may occur when influent is warmer than the basin temperature. Some excellent references are available on this subject.[8-10] The operating principles of inclined surface clarifiers are illustrated in Figure 12-4(b) and (c).

12-3-4 Stacked or Two-Tray

In areas where land for treatment facilities is not available, stacked or two-tray basins for both primary and secondary clarification are used. The two-tray basins are series flow or parallel flow. In a series flow basin, the influent enters the lower tray, goes up into the second tray on the far end, and travels in the opposite direction. The effluent exits from the upper tray. The sludge from both trays is collected in a common hopper. Baffles straighten the flow paths and minimize turbulence at the influent point in the lower tray and at the turnaround on the top tray.

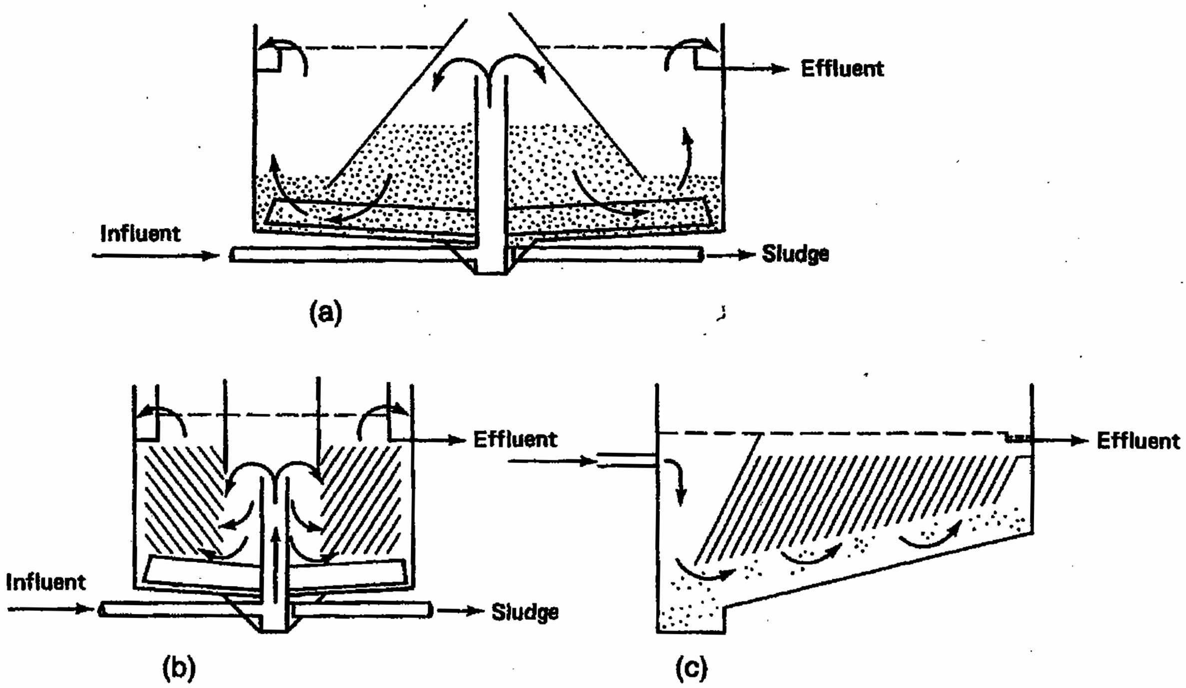


Figure 12-4 Types of Clarifiers: (a) circular solids contact clarifier, (b) parallel inclined plates in a circular clarifier, and (c) tube settlers in a rectangular clarifier.

In the parallel flow unit the influent flow enters both the upper and lower trays at the same end and travels longitudinally. Influent baffles straighten the flow path and minimize the turbulence. Effluent is removed from both trays by longitudinal launders along the top tray.

Chain and flight collectors are used for sludge collection and removal. Scum is removed from the upper tray. Stacked sedimentation basins are shown in Figure 12-5. Detailed discussion on the subject may be found in Refs. 11–15.

12-3-5 Proprietary Systems

A number of proprietary systems that claim to enhance sedimentation are on the market. One such system is Infilco Degremont's Densa Deg® high rate clarifier and thickener. The unique features are external solids circulation for solids contact effect and lamella tubes for added clarification. This system is shown in Figure 12-6(a).[16] Spiracone® solids contact clarifier of the General Filter Company was developed for water treatment, but it can also be applied for enhanced primary sedimentation. The clarification unit is a conical tank, as illustrated in Figure 12-6(b).[17] Influent enters at the bottom of the tank such that a circular flow pattern is developed in the tank. Theoretically, this keeps the sludge blanket in motion, prevents the development of channels in the sludge blanket, and produces a thicker sludge. The Boat Clarifier® is shown in Figure 12-6(c).[18] The Burns & McDonnell Treatment System (BMTS) developed by Advanced Environmental Enterprises, Inc. provides interchannel clarification. The system is constructed on the inside wall of an aeration basin. The arrangement of a BMTS is shown in Figure 12-6(d).[19] WesTech Engineering, Inc. has come up with a clarifier optimization package (COP) for

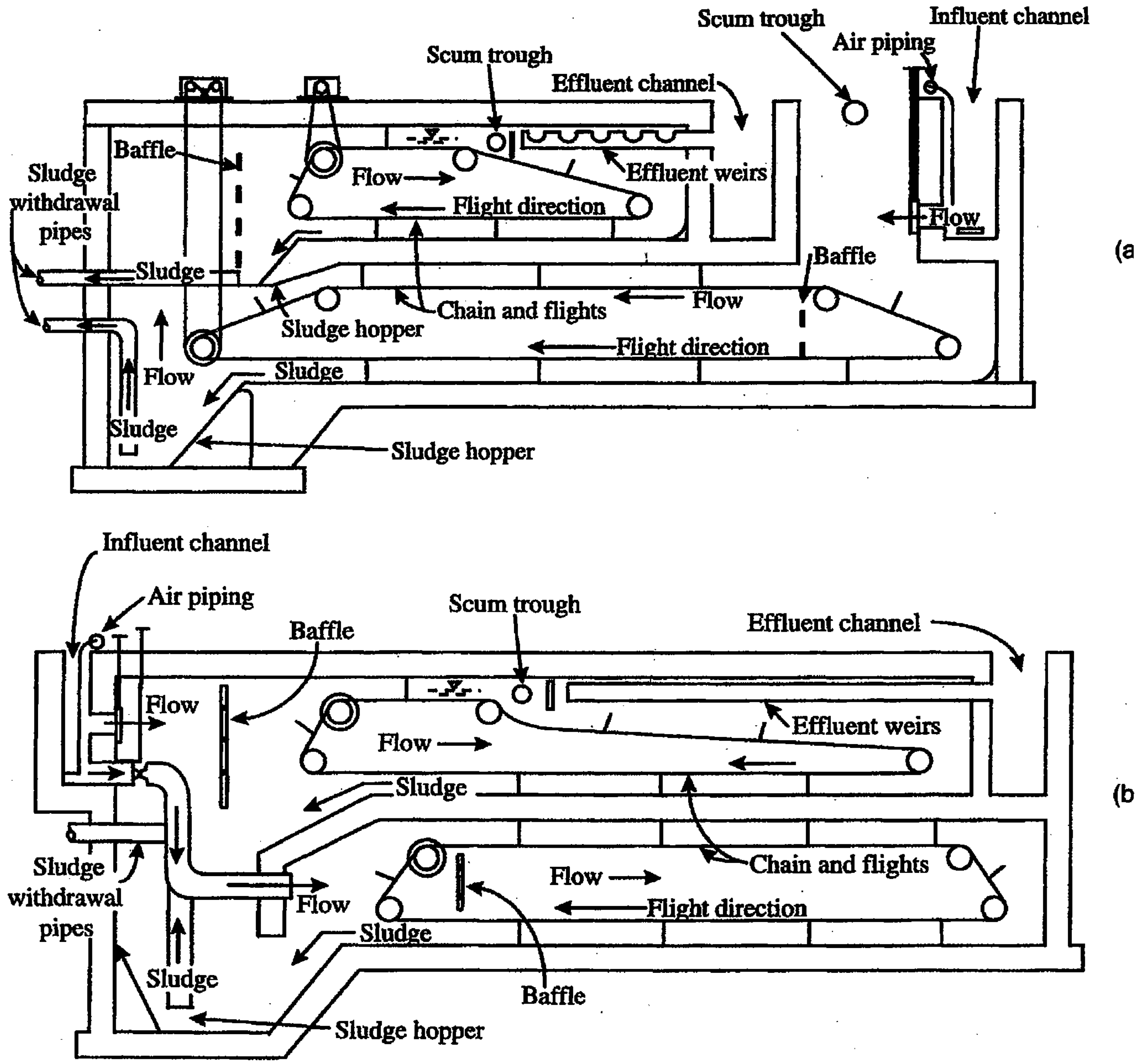


Figure 12-5 Stacked or Two-Tray Sedimentation Basin: (a) series flow and (b) parallel flow.

retrofitting an existing circular basin for enhanced clarification. Many of the retrofit features are center column modifications for energy dissipation, enhanced flocculation and reduction in floc shearing, and to impart tangential motion. The spiral rake blade increases sludge transport, and the unique design of the sludge hopper maintains high underflow concentration.[20] In addition to these specific proprietary systems, there are many more systems that are currently available. Some systems are effective while others are not. Engineers should carefully evaluate the claims and effectiveness of these proprietary systems through testing and discussions with operators of other full-scale installations. Readers are referred to manufacturers for obtaining details of these and other proprietary systems. A list of manufacturers is provided in Appendix D.

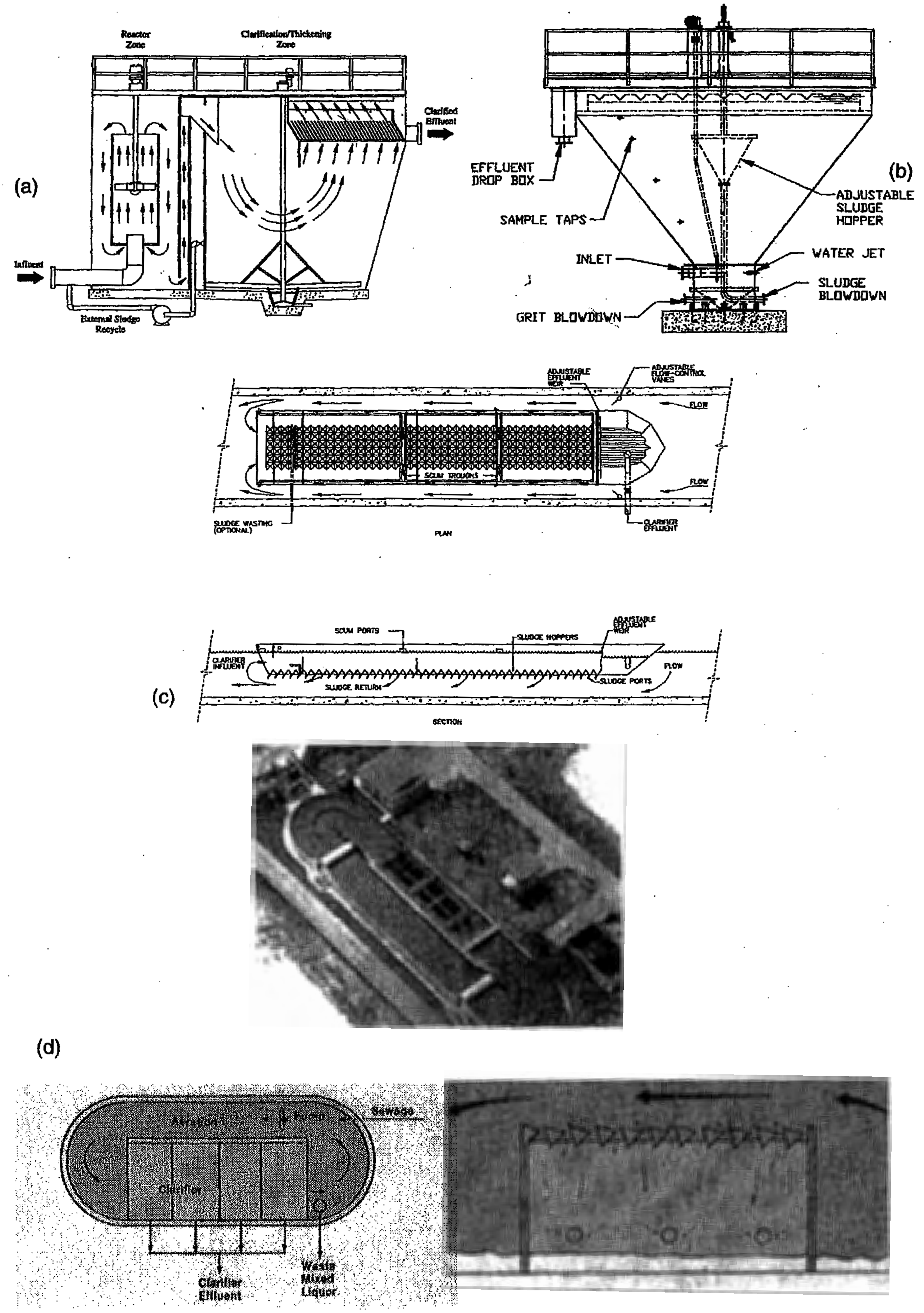


Figure 12-6 Proprietary Equipment for Sedimentation Basin: (a) DensaDeg® high-rate clarifier and thickener (courtesy Infilco Degremont, Inc.), (b) General Filter SPIRACONE® solids contact clarifier (courtesy U.S. Filter), (c) BOAT® intrachannel clarifier (courtesy United Industries, Inc.), and (d) Burns & McDonell BMTS intrachannel clarifier (courtesy Burns & McDonell).

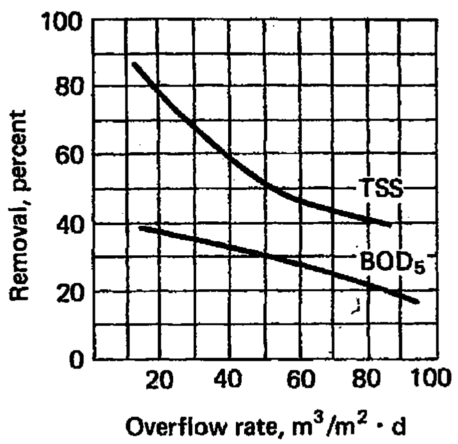


Figure 12-7 Removal of Suspended Solids and BOD_5 with Respect to Overflow Rate in Primary Clarifiers.

12-4 DESIGN FACTORS

The design objective of a primary sedimentation facility is to provide sufficient time under quiescent conditions for maximum settling to occur. The solids removal efficiency of a clarifier is reduced because of the following conditions:

1. Eddy currents induced by the inertia of the incoming fluid
2. Surface currents produced by wind action
3. Vertical currents induced by outlet structure
4. Vertical convection currents induced by the temperature difference between the influent and the tank contents
5. Density currents causing cold or heavy water to underrun a basin, and warm or light water to flow across its surface
6. Currents induced because of the sludge scraper and sludge removal system

Therefore, many factors, such as overflow rate, detention period, weir-loading rate, shape and dimensions of the basin, inlet and outlet structures, and sludge removal system, enter into the design of the sedimentation basin. Many of the design factors are discussed in the following sections.[1-4,7,21-25]

12-4-1 Overflow Rate or Surface-Loading Rate

Overflow rate is expressed in cubic meters per day per square meter ($m^3/m^2{\cdot}d$)[b] of the surface area of the tank. The effect of overflow rate on suspended solids removal varies widely, depending on the character of the wastewater, proportion of settleable solids, concentration of solids, and other factors. For a well-designed and -operated plain primary sedimentation basin, the typical removal of suspended solids and BOD_5 from municipal wastewater as a function of overflow rate is given in Figure 12-7. The overflow rate should be small enough to ensure satisfactory performance at peak flows.[22] The de-

[b] $1\ m^3/m^2{\cdot}d = 24.5424\ gpd/ft^2$.

TABLE 12-1 Design Overflow Rates for Plain Primary Sedimentation Basins

Condition	Range ($m^3/m^2 \cdot d$)	Typical ($m^3/m^2 \cdot d$)
Primary clarification prior to secondary treatment		
Average flow	30–50	40
Peak flow	70–130	100
Primary clarification with waste-activated sludge return[a]		
Average flow	25–35	30
Peak flow	45–80	60

[a]In many designs the waste-activated sludge is returned to the primary sedimentation basin. The objective is to concentrate the secondary sludge in the primary sedimentation basin.
Note: 1 $m^3/m^2 \cdot d$ = 24.5424 gpd/ft^2.

sign overflow rates for various types of clarification facilities are given in Table 12-1. Primary sedimentation facilities are generally designed for an overflow rate of 40 $m^3/m^2 \cdot d$ at average design flow.

12-4-2 Detention Period

For a given surface area, the detention time depends on the depth of a tank. The detention time for various overflow rates and tank depths are summarized in Table 12-2. Most of the designs utilize 1- to 2-h detention periods for primary tanks and 2–4 h for secondary clarifiers at average design flow. The removal of BOD_5 and TSS may vary greatly. Typical removals in a well-designed and -operated primary sedimentation basin as a function of detention time are shown in Figure 12-8.

TABLE 12-2 Detention Times for Various Overflow Rates and Tank Depths

	Detention Period (h)					
Overflow Rate ($m^3/m^2 \cdot d$)	2.0-m Depth	2.5-m Depth	3.0-m Depth	3.5-m Depth	4.0-m Depth	4.5-m Depth
30	1.6	2.0	2.4	2.8	3.2	3.6
40	1.2	1.5	1.8	2.1	2.4	2.7[a]
50	1.0	1.2	1.4	1.7	1.9	2.2
60	0.8	1.0	1.2	1.4	1.6	1.8
70	0.7	0.9	1.0	1.2	1.4	1.5
80	0.6	0.8	0.9	1.1	1.2	1.4

[a]A 4.5-m deep sedimentation basin having an overflow rate of 40 $m^3/m^2 \cdot d$ (982 gpd/ft^2) will provide a detention period of 2.7 h. This may be the most desirable design condition.

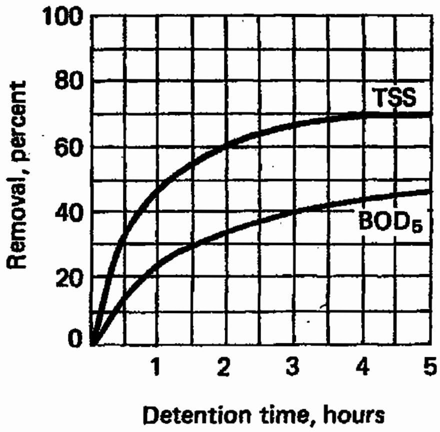


Figure 12-8 Removal of Suspended Solids and BOD_5 with Respect to Detention Time in Primary Clarifiers.

12-4-3 Weir-Loading Rate

Weir-loading rates have some effect on the removal efficiency of sedimentation basins. Sedimentation basins are generally designed for loading less than 370 m^3 per d per m length of the weir.[c] The Ten-States Standards recommend the following weir loadings:[22]

- weir-loading rate of 248 m^3/m·d for plants designed for peak design flows of 44 L/s or less[d] < 158 m^3/h
- weir-loading rate of 372 m^3/m·d for plants designed for peak design flows in excess of 44 L/s 158 m^3/h <

12-4-4 Dimensions

The dimensions of a primary sedimentation basin are selected to accommodate the standard equipment supplied by the manufacturers. Also, consideration is given to the size of the installation; local site conditions; regulations of the local water pollution control agencies; the experience, judgment, and preference of the designer; and the economics of the system. Multiple units are essential to keeping the process in operation during the repair and servicing of one unit. Basic dimensions of rectangular and circular clarifiers are summarized in Table 12-3.

12-4-5 Solids Loading

Solids loading is not an important deciding factor for the primary sedimentation facility. It varies from 1.5 to 34 kg per m^2 per d.[e] In final clarifiers, the solids loading rate may vary greatly, depending on the concentration of mixed liquor suspended solids and

[c] 1 m^3/m·d = 80.52 gpd/ft.

[d] 1 mgd = 43.813 L/s.

[e] 1 kg/m^2·d = 0.205 lb/ft^2·d.

TABLE 12-3 Dimensions of Rectangular and Circular Basins

Clarifier	Range	Typical
Rectangular		
Length, m	10–100	25–60
Length-to-width ratio	1.0–7.5	4
Length-to-depth ratio	4.2–25.0	7–18
Sidewater depth, m	2.5–5.0	3.5
Width, m[a]	3–24	6–10
Circular		
Diameter, m[b]	3–60	10–40
Side depth, m	3–6	4

[a]Most manufacturers build equipment in width increments of 61 cm (2 ft). If width is greater than 6 m (20 ft), multiple bays may be necessary.
[b]Most manufacturers build equipment in 1.5-m (5-ft) increments of diameter.

sludge volume index. A practical range of solids loading in final clarifier is 49–98 $kg/m^2 \cdot d$.[1,2,6,22]

12-4-6 Influent Structure

The influent structures are designed to serve many purposes. The objectives are

1. To dissipate energy in incoming flow by means of baffles or stilling basin
2. To distribute flow equally along the width
3. To prevent short-circuiting by disturbing thermal and density stratification
4. To promote flocculation
5. To create small head loss

In the design of influent structure, provision is also made to control flows, remove scum, and facilitate maintenance. Normally, inlets are uncovered or have removable gratings. The velocity at the inlet pipe is maintained at approximately 0.3 m/s. Many inlet details for rectangular tanks are shown in Figure 12-9.[2,6,21,23,24] Selection of any design may depend on tank size, flow conditions, and designer's preference and experience.

Based on the influent structures the circular clarifiers are classified as center and peripheral feed. In center feed circular clarifiers the inlet is at the center, and the outlet is along the periphery. A concentric baffle distributes the flow equally in radial directions. The advantages of center feed clarifiers are low upkeep cost and ease of design and construction. The disadvantages include short-circuiting, low detention efficiency, lack of scum control, and loss of sludge into the effluent.

The center feed square clarifier is a modification of the circular clarifier. Both the in-

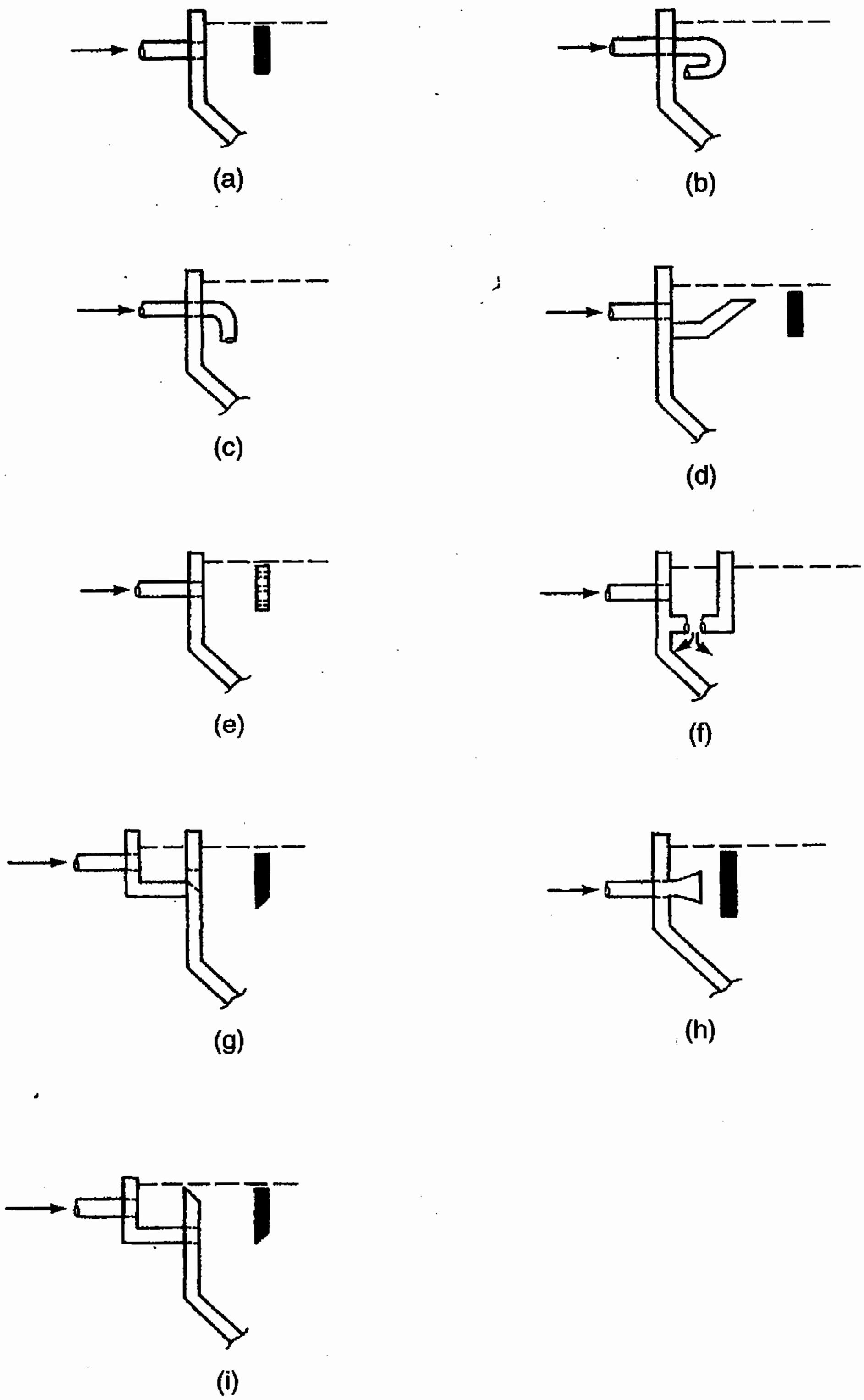


Figure 12-9 Details of Influent Structures for Rectangular Sedimentation Basin: (a) inlet pipes discharging against a baffle, (b) U-shape elbow discharging against the wall, (c) a series of inlet pipes spaced across the width with turned elbow, (d) an inclined weir with baffle, (e) perforated baffle, (f) a stilling basin with opening at the bottom, (g) pipe discharging in a channel that has series of openings discharging against a baffle, (h) a bell-shaped diverging pipe followed by a baffle, and (i) an overflow weir followed by a baffle.

let and outlet structures are similar to those of circular clarifiers. The inside corners are rounded to prevent solids accumulation. The benefit of square clarifiers is the opportunity to use the common walls in multiple units. The flow pattern of center feed circular clarifier is shown in Figure 12-10(a). In the peripheral feed clarifiers, the flow enters along the periphery. These clarifiers are considerably more efficient and have less short-circuiting than the center feed clarifiers.[1,2,23-27] There are two major variations of peripheral feed clarifiers:

1. The influent is distributed through the orifices in an influent raceway on the entire periphery. A skirt is put underneath the raceway. This skirt gives low initial velocity and minimizes the density currents and short-circuiting.
2. A circular baffle is suspended a short distance from the tank wall to form an annular space into which the flow is discharged in a tangential direction. The wastewater flows spirally around the tank and underneath the baffle.

Both the variations of peripheral feed clarifiers are illustrated in Figure 12-10.

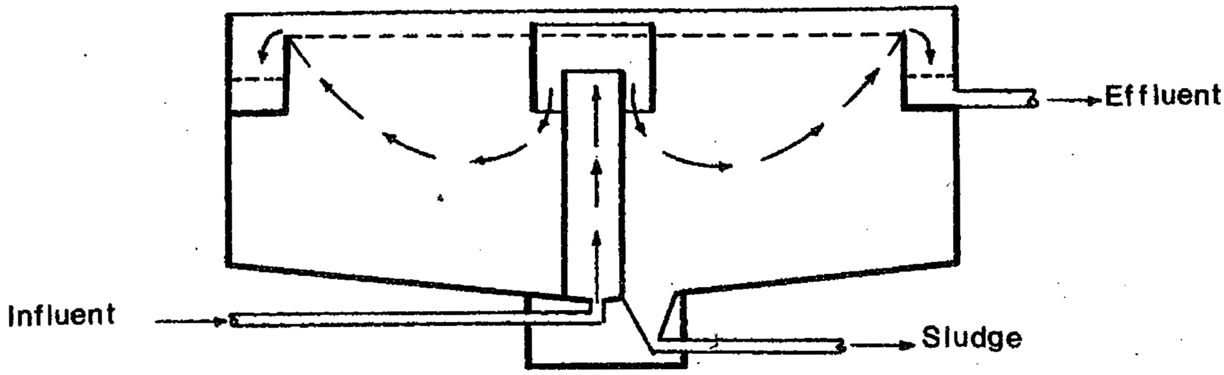
12-4-7 Effluent Structure

The effluent structures are designed to (1) provide a uniform distribution of flow over a large area, (2) minimize the lifting of the particles and their escape into the effluent, and (3) reduce the escape of floating matter to the effluent. Most common types of effluent structures for rectangular and circular tanks are weirs that are adjustable for leveling. These weir plates are sufficiently long to avoid high head that may result in updraft currents and lifting of the particles. A weir loading of 372 m^3 per day per linear meter at peak design flow is used for plants larger than 44 L/s.[22] Both straight edge and V notches, either on one side of the trough or both sides, have been used in rectangular and circular tanks. V notches provide uniform distribution at low flows. A baffle is provided in front of the weir to stop the floating matter from escaping into the effluent. Normally, the weirs in rectangular tanks are provided on the opposite end of the influent structure. Different weir configurations in rectangular basins may be utilized in order to obtain the desired length of the weir. Some arrangements for rectangular basins are shown in Figure 12-11.

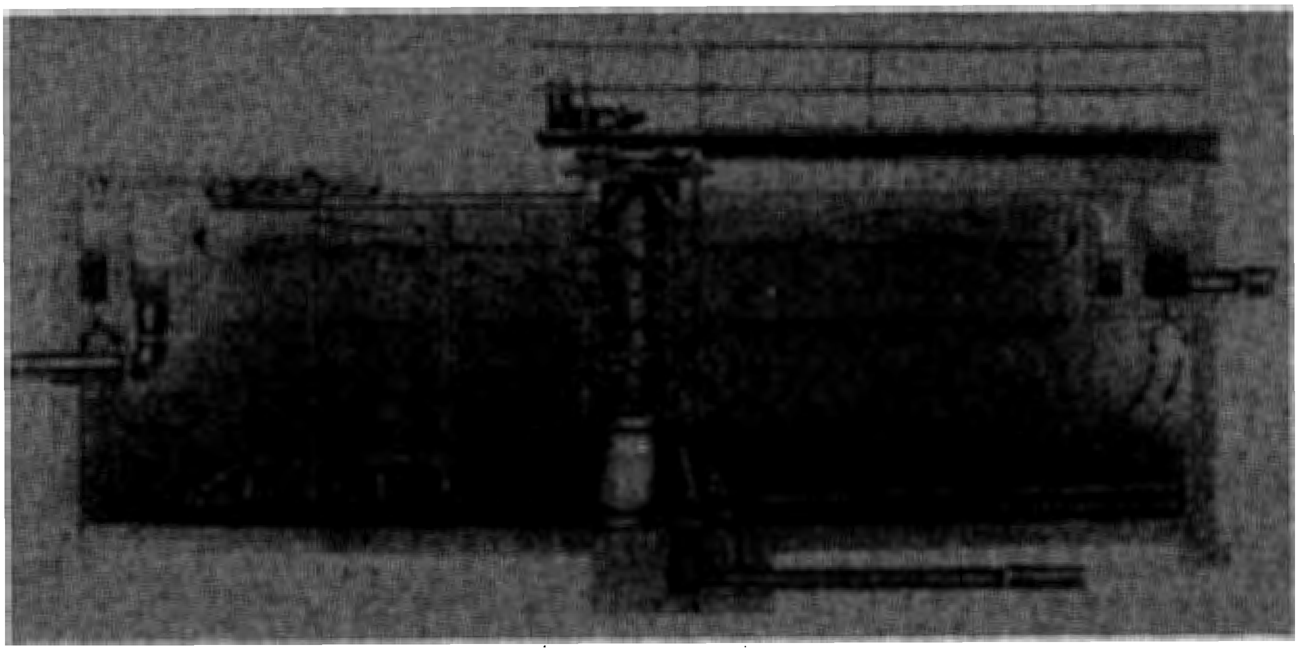
In circular clarifiers, the outlet weir can be near the center of the clarifier or along the periphery, as shown in Figure 12-10. The center weir generally provides high-velocity gradients, which can result in solids carryover. Figure 12-12 shows the arrangement of weir notches, effluent launders, and outlet channel in rectangular and circular clarifiers.

12-4-8 Sludge Collection

Bottom Slope. The floor of the rectangular and circular tanks are sloped toward the hopper. The slope is made to facilitate draining of the tank and to move the sludge toward the hopper. Rectangular tanks have a slope of 1–2 percent. In circular tanks, the slope is approximately 40–100 mm/m diameter.



(a)



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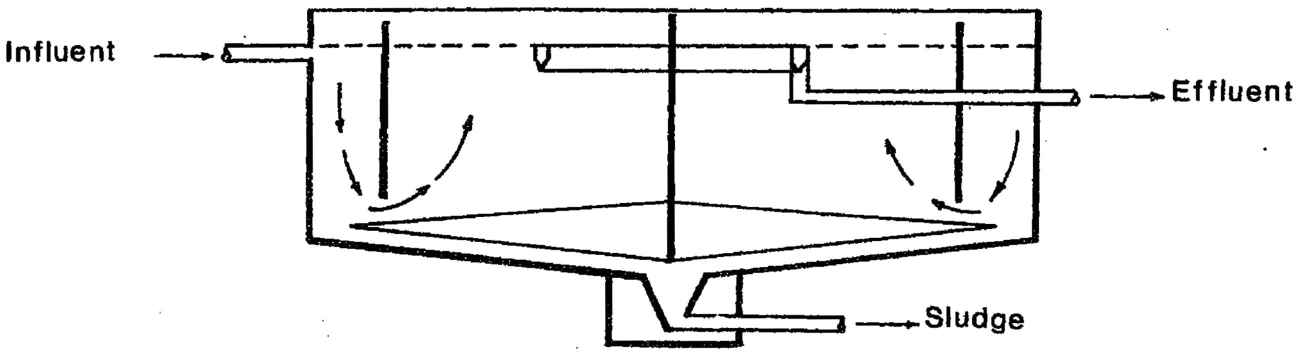


(c)

Figure 12-10 Influent and Effluent Structures for Circular Clarifiers: (a) circular clarifier with center feed, (b) peripheral feed circular clarifier with effluent and influent channels separated by a skirt (courtesy Envirex, U.S. Filter), and (c) peripheral feed circular clarifier with effluent weirs near the center of the basin.

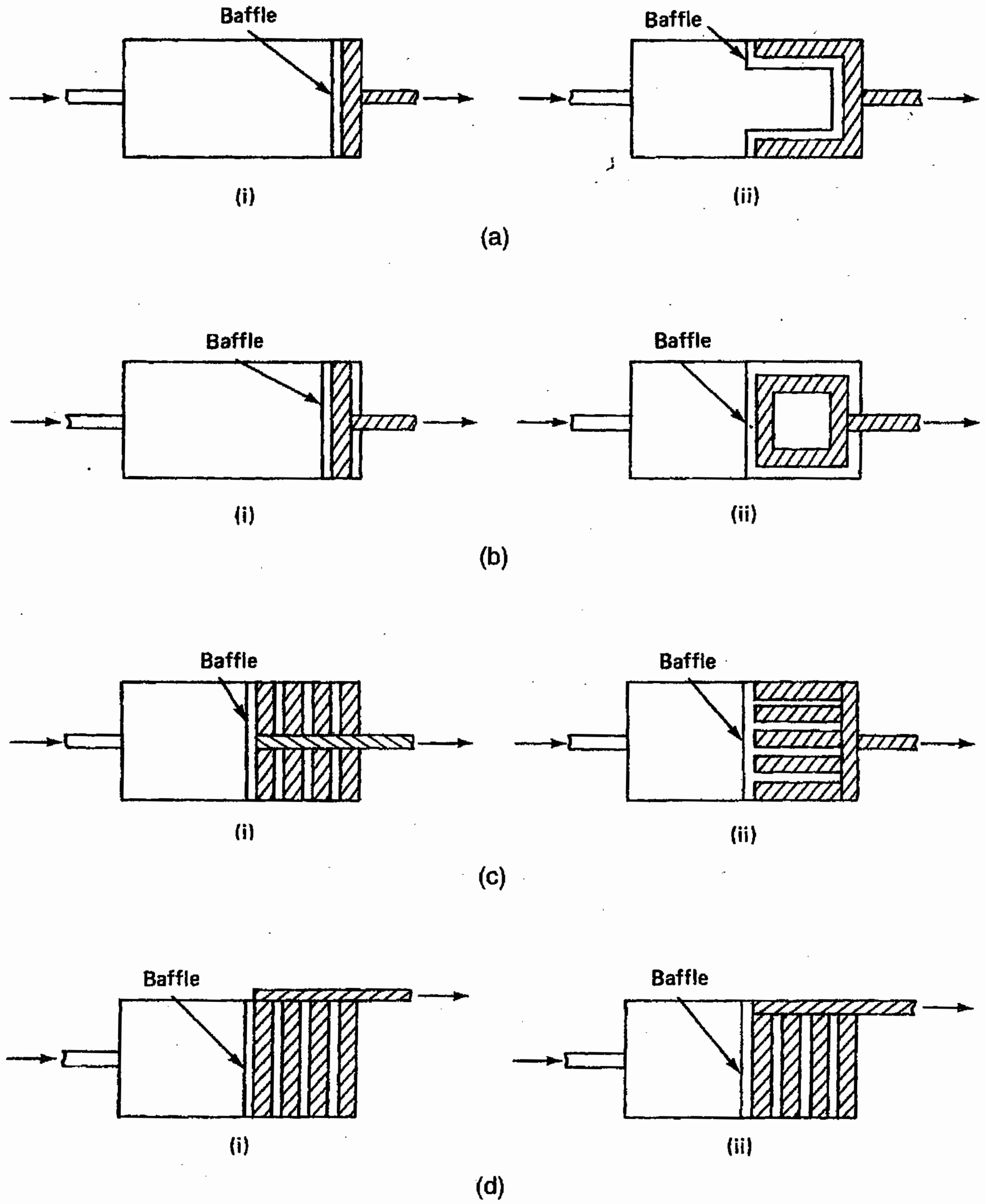
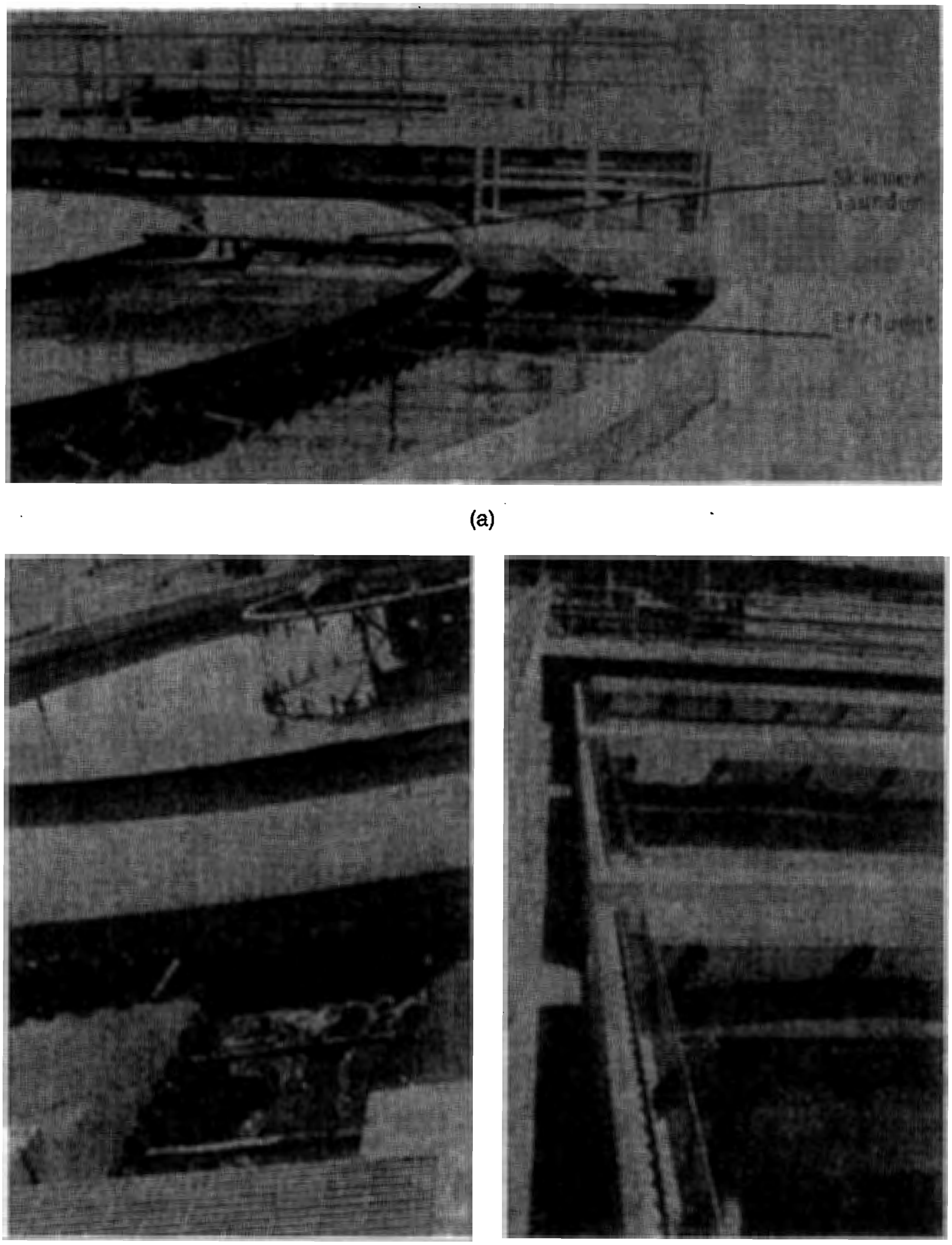


Figure 12-11 Various Configurations of Effluent Structure Used for Rectangular Clarifiers: (a) single weir and trough, (b) double weirs and trough, (c) multiple weirs and troughs with outlet channel at the middle, and at the end (d) multiple weirs and troughs with outlet channel at the side.



(a)

(b)

(c)

Figure 12-12 Effluent Weir, Notches, Launder, and Outlet Channel in Rectangular and Circular Clarifiers: (a) V notches and suspended effluent launder in circular clarifier; (b) effluent launders discharging into an outlet channel; and (c) effluent notches, launders, and outlet channel in a rectangular clarifier.

Equipment. In mechanized sedimentation tanks, the type of sludge collection equipment varies with size and shape of the tank. The sludge collection equipment for rectangular and circular clarifiers is discussed below.

Rectangular Tanks. In rectangular tanks the sludge collection equipment may consist of (1) a pair of endless conveyor chains running over sprockets attached to the shafts or (2) moving-bridge sludge collectors having a scraper to push the sludge into the hopper or a suction-type arrangement to withdraw the sludge from the basin. Design details and advantages and disadvantages of these types of sludge collection equipment are summarized in Table 12-4.[2,27,28] The details of endless conveyor chain and moving-bridge sludge collector arrangements are shown in Figures 12-13 and 12-14.

Circular Tanks. The circular tanks utilize two types of sludge collection equipment:

1. The scraping mechanism is installed with radial arms having plows set at an angle supported on center pier or on a beam spanning the tank. The clarifiers in excess of 10 m in diameter normally have a central pier, while the clarifiers with smaller diameter utilize beam support. The flight travel speed is 0.02–0.06 revolutions/min.
2. Suction-type units are used for handling light sludge. The suction mechanism is installed similar to the scraping mechanism. The equipment details of circular clarifiers are shown in Figures 12-3 and 12-12(a) and (b).

12-4-9 Sludge Removal

The sludge is removed from the hopper by means of a pump. The following design considerations are given to the sludge removal system:

1. Provision of continuous sludge pumping is desirable. Timer-control pumping cycle may be necessary to achieve self-cleaning velocity.
2. Each sludge hopper should have an individual sludge withdrawal pipe at least 15 cm in diameter.
3. In rectangular tanks, cross-collectors are preferred over multiple hoppers. The multiple hoppers pose operational difficulties such as sludge accumulating in corners and slopes and arching over the sludge-drawoff piping. Also, the cross-collectors provide withdrawal of more uniform and concentrated sludge.
4. Screw conveyors for sludge removal are also used.
5. In new plants, a photocell-type or a sonic-type sludge blanket detector is often used to provide an indication of the depth of the sludge blanket. Therefore, an automatic control of sludge pump or siphon pipes can be achieved. This arrangement is particularly desirable for final clarifiers. The sludge pump or the siphon starts or stops automatically when the predetermined sludge blanket depth is reached.
6. The sludge pump used is a self-priming centrifugal and normally discharges into a common manifold. One sludge pumping station can serve two rectangular clarifiers. The circular clarifiers are normally arranged in groups of two or four. The sludge is

TABLE 12-4 *System Description and Advantages/Disadvantages of Sludge Collectors for Rectangular Tanks*

Collector Type	Description	Advantages	Disadvantages
Conveyor chain	(1) One endless chain is connected to a shaft and a drive unit.	(1) Arrangement is simple to install.	(1) High maintenance cost of chain and flight removal mechanism
	(2) Linear conveyor speed is 0.3–1.0 m/min for primary and 0.3 m/min for final clarifier.	(2) Power consumption is small.	(2) Tank must be dewatered for gear or chain repair.
	(3) Cross-woods (flights) are attached to the chain at 3-m intervals and are up to 6 m in length. These flights scrape the sludge to the hopper. The cross-wood is 5 cm thick and 15–20 cm deep.	(3) Scum collection is efficient.	(3) Light sludge may resuspend.
	(4) For tanks greater than 6 m in width, multiple pairs of chains are used.	(4) Suitable for heavier sludge	
	(5) The floating material is pushed in opposite direction of sludge, and is collected in a scum collection box.		
Bridge drive scraper	(1) Standard traveling beam bridges for spans up to 13 m (40 ft) and truss bridge for spans over 13 m are used.	(1) All moving mechanisms are above water. No underwater bearings are used.	(1) High-power requirement to move the bridge.
	(2) Bridge travel is accomplished by the use of a gear motor.	(2) Standard designs permit scraper repair or replacement without tank dewatering.	(2) Units will not operate with ice-covered tanks.
	(3) The wheels run on rails that are attached to the footing wall along each side wall of the basin.	(3) No width restrictions as with chain type	(3) In long-span bridges, the wheels may climb frequently over rails, causing breakdowns.

Continued

TABLE 12-4 System Description and Advantages/Disadvantages of Sludge Collectors for Rectangular Tanks—cont'd

Collector Type	Description	Advantages	Disadvantages
	(4) Mechanical scrapers or rakes are hung from the top carriage that push the sludge to the hopper. (5) Separate blades are provided on top to move the scum.	(4) Longer operation life (5) Lower maintenance cost in low-span bridges	
Bridge drive sludge suction	(1) The bridge design is similar to the above arrangement. (2) The sludge removal mechanisms are attached to the bridge and provide continuous removal of sludge along the length of travel. (3) Pump, siphon, or airlift arrangements are used to suck and remove the sludge.	(1) Better pickup of light sludge (2) Other advantages same as above (3) Used for biological and chemical sludges	(1) Same as above (2) Not used in primary clarifier

Source: Adapted in part from Refs. 2, 27, and 28.

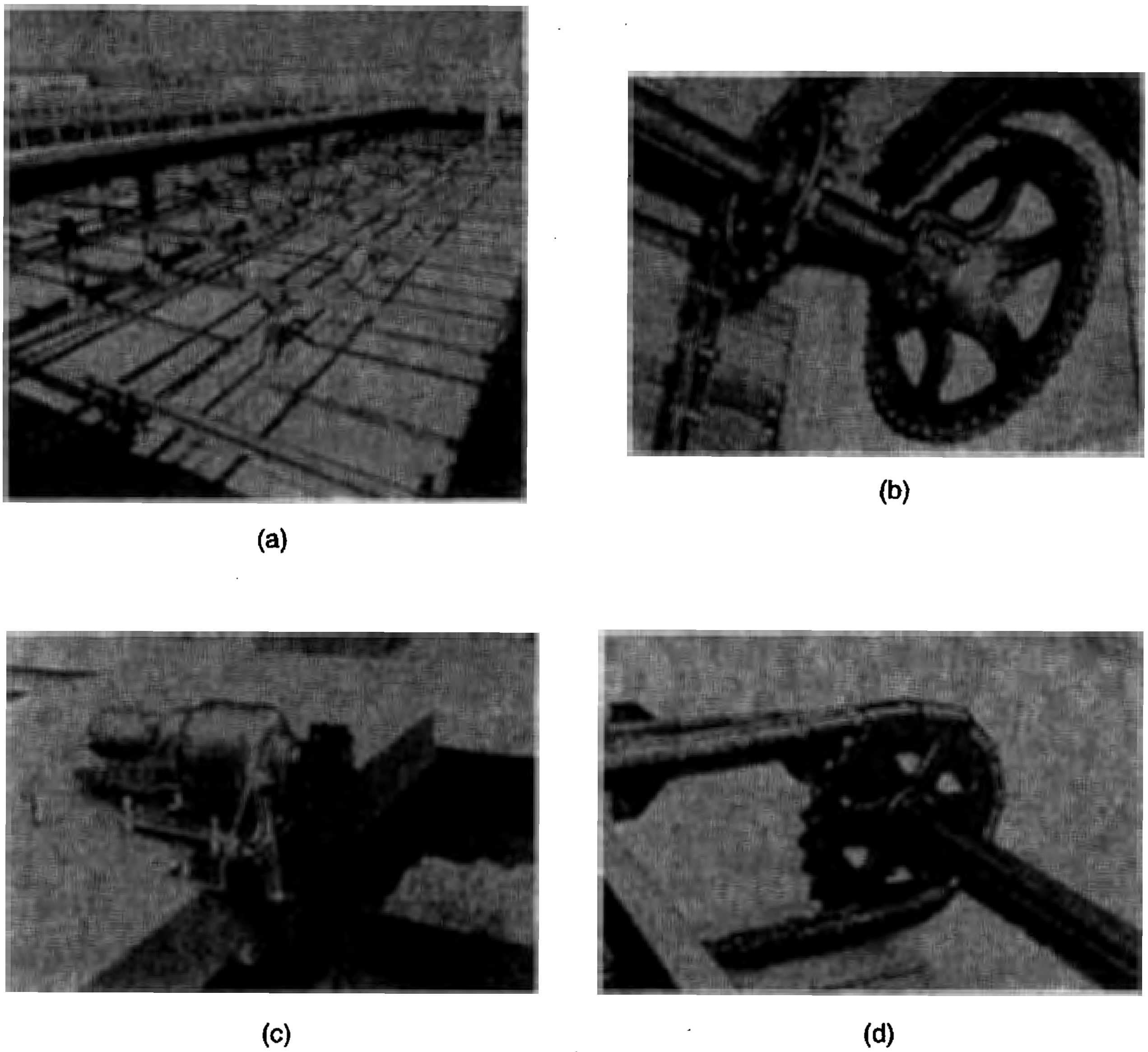


Figure 12-13 Endless Conveyor Chain Details: (a) conveyor sludge collectors with skimmer (courtesy Envirex, U.S. Filter), (b) drive sprockets, (c) chain drive with gear speed reducer, and (d) collector sprocket (courtesy FMC, U.S. Filter).

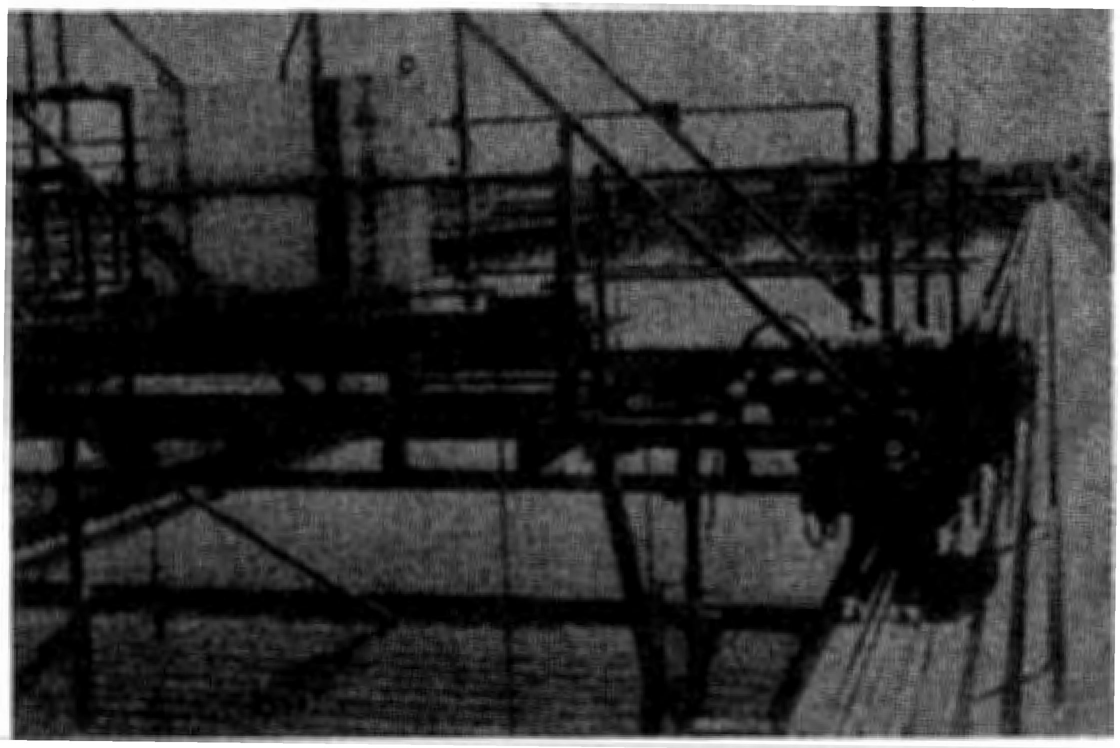
withdrawn to a control chamber located in the middle; from there it is pumped to other sludge-handling areas.

12-4-10 Scum Removal

Scum that forms on the surface of the primary clarifiers is generally pushed off the surface to a collection sump. In rectangular tanks the scum is normally pushed in the opposite direction by the flights of the sludge mechanism in its return travel. In circular clarifiers, the scum is moved by a radial arm that rotates on the surface with the sludge removal equipment. The scum can also be moved by water sprays.

The scum may be scraped manually or mechanically up an inclined apron. In small installations, a hand-tilt slotted pipe with a lever or screw is commonly used. This ar-

(a)



(b)

Figure 12-14 Moving Bridge Sludge Collector: (a) traveling truss bridge with mechanical scraper hung from the top. The effluent weir is the type shown in Figure 12-11(c-ii); (b) traveling truss bridge showing wheels that run on rails.

rangement may result in a relatively large volume of scum liquor. Various types of scum removal arrangements used in rectangular and circular basins are shown in Figures 12-3, 12-12, and 12-15.

All effluent weirs have baffles to stop the loss of scum into the effluent. A scum sump is provided outside the tank. A scum pump transfers scum to the disposal facility.

The scum usually has a specific gravity of 0.95. Solids content may vary from 25 to 60 percent. The quantity of scum at a plant may vary from 2 to 13 kg per 10^3 m^3 (17–110 lb per million gallon). Pumping of scum is difficult since it may form a clotty mass (called greaseballs) at lower temperatures. Pipings are often glass-lined and kept reasonably warm to minimize blockages. Scum has been digested in aerobic and anaerobic digesters, landfilled, or incinerated. Heating values may range from 16,000 to 40,000 kJ/kg of dry solids (7000–17,000 Btu/lb).

12-5 ENHANCED SEDIMENTATION

The performance of a primary sedimentation facility can be enhanced by preaeration or chemical coagulation and precipitation. These techniques are briefly presented below.

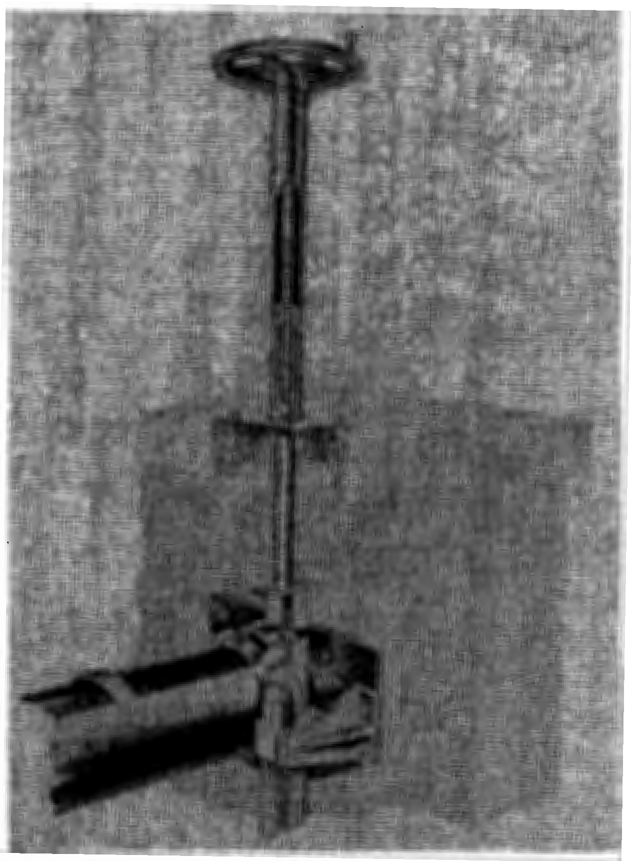
12-5-1 Preaeration

Preaeration of wastewater before sedimentation promotes flocculation of nonsettleable solids into more readily settleable floc. Consequently, both total suspended solids and BOD_5 removal efficiencies are increased by approximately 7–8 percent. An aerated grit chamber will also promote improved grit separation and enhance the performance of a primary sedimentation basin.

Preaeration in a channel ahead of sedimentation basin promotes uniform distribution of flow into the basin. Approximately 20–30 minutes of aeration at a minimum air supply of 0.82 L/L (0.11 ft^3/gal) is considered sufficient.[2] Other benefits of preaeration are addition of dissolved oxygen and prevention of septicity during primary sedimentation. The major drawback, however, is scrubbing of VOCs and intensified odor problems. Collection of gases from the preaeration channel and scrubbing at a centralized location is an effective method of controlling odors from the primary sedimentation basin.

12-5-2 Chemical Coagulation and Precipitation

Chemical coagulation of raw wastewater before sedimentation promotes flocculation of colloidal particles into more readily settleable floc. Some chemicals cause precipitation by chemically or physically modifying the solubility of many dissolved solids. Thus, the removal efficiencies of suspended solids, BOD_5, and phosphorus are increased. Sedimentation basins with coagulation may achieve 60–90 percent TSS, 40–70 percent BOD_5, 30–60 percent COD, and 70–90 percent phosphorus removals. The advantage of chemical coagulation is greater removal efficiencies, ability to use higher overflow rates, and a more consistent performance. The disadvantages of coagulation include increased sludge, production of sludge that is more difficult to thicken and dewater, and an increase in operation and maintenance costs.



(a)

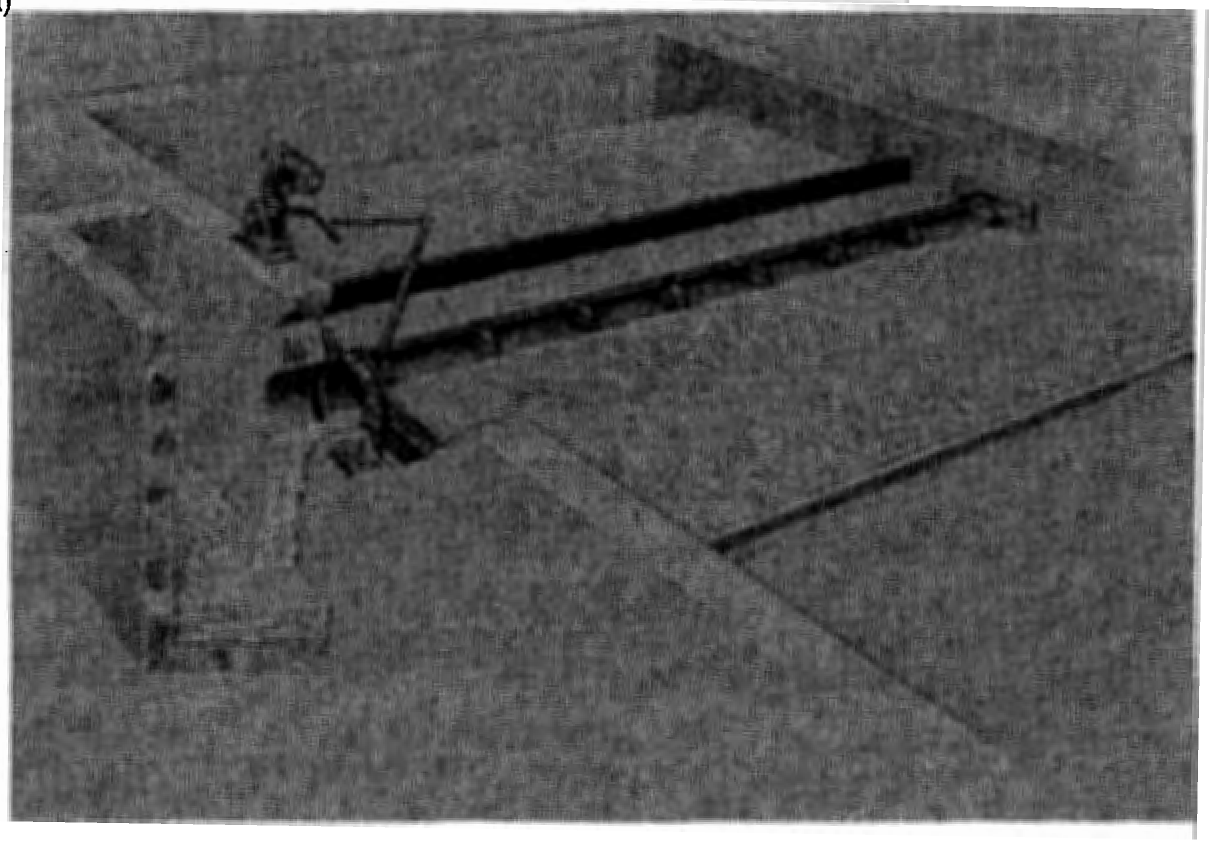
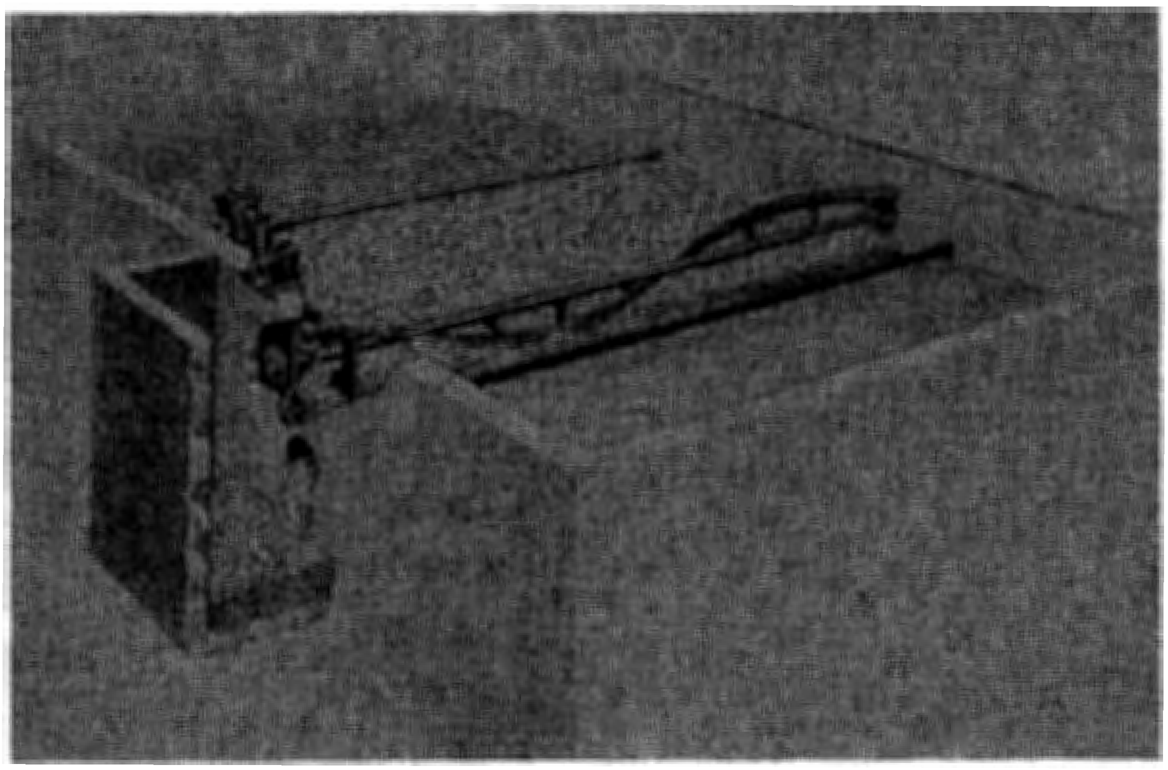


Figure 12-15 Scum Collection and Removal Arrangements: (a) hand- and motor-operated system to tilt the open-top pipe to remove and discharge scum into dewatering trap.



(b)

Figure 12-15—cont'd (b) motor-operated spiral skimmer that turns the blades to push and drop the scum into a scum trough (courtesy Envirex, U.S. Filter).

Process Description. All colloidal particles have a very large surface area to mass ratio. Their mass is so small that the gravitational force has little effect upon their behavior. Additionally, most colloids are electrically charged and therefore remain in suspension and in motion. This is called stability of colloid. Coagulation is a chemical treatment process used to destabilize the colloidal particles. In the coagulation process, chemicals are added to the wastewater, which either break down the stabilizing forces, enhance the destabilizing forces, or both. Traditionally, metal salts such as alum or aluminum sulfate, ferric sulfate, ferric chloride, or ferrous sulfate have been utilized as coagulants. Physical and chemical properties of commonly used chemicals are provided in Table 12-5. Selection of a chemical for enhanced sedimentation should be based on performance, dosages, and reliability. Standard jar test apparatus is utilized to evaluate performance and optimize dosages. Operating experience, cost, and other relevant information obtained from other similar plants must also be utilized. The physical and chemical properties of commonly used coagulants is summarized in Table 12-5. In recent years, however, polymers (long molecular-chain compounds) have been used in conjunction with or in lieu of metal salts to enhance the coagulation process. Coagulants destabilize colloids by a combination of three mechanisms: compression of the double layer, interparticle bridging, and enmeshment in a precipitate.

Chemical precipitation involves the addition of chemicals to decrease the solubility of a targeted constituent so that the precipitate can be removed by flocculation and sedimentation. Chemical precipitation is commonly used in water treatment to remove iron, manganese, calcium, magnesium, and other heavy metals. In wastewater treatment the

TABLE 12-5 Commonly Used Chemicals for Enhanced Sedimentation and Physicochemical Nutrient Removal

				Commercial-Grade Qualities						
Chemical Name	**Synonyms**	**Chemical Formula**	**Molecular Weight**	**Appearance**	**Bulk Density (kg/m^3)**	**Specific Gravity**	**Solubility (kg/m^3 of water)**	**Chemical Content (% w/w)**	**Water Content (% w/w)**	**pH**
Aluminum surface	Alum	$Al_2(SO_4)_3 \cdot 14.3H_2O$	599.77	White to light tan solid	1000–1096	1.25–1.36	approx. 872	Al: 9.0–9.3	—	approx. 3.5
	Liquid alum	$Al_2(SO_4)_3 \cdot 49.6H_2O$	1235.71	White or light gray to yellow liquid	—	1.30–1.34	Very soluble	Al: 4.0–4.5	71.2–74.5	—
Ferric chloride	Iron (III) chloride, Iron thichloride	$FeCl_3$	162.21	Green-black power	721–962	—	approx. 719	Fe: approx. 34	—	—
		$FeCl_3 \cdot 6H_2O$	270.30	Yellow-brown lump	962–1026	—	approx. 814	Fe: 20.3–21.0	—	—
	Ferric chloride solution	$FeCl_3 \cdot 13.1H_2O$	398.21	Raddish brown syrupy liquid	—	1.20–1.48	Very soluble	Fe: 12.7–14.5	56.5–62.0	0.1–1.5
Ferric sulfate	Iron (III) sulfate, Iron persulfate	$Fe_2(SO_4)_3 \cdot 9H_2O$	562.02	Red-brown power	1122–1154	—	—	Fe: 17.9–18.7	—	—
	Ferric sulfate solution	$Fe_2(SO_4)_3 \cdot 36.9H_2O$	1064.64	Raddish brown syrupy liquid	—	1.40–1.57	Very soluble	Fe: 10.1–12.0	56.5–64.0	0.1–1.5
Ferrous sulfate	Copperos	$FeSO_4 \cdot 7H_2O$	278.02	Green crystal lump	1010–1058	—	—	Fe: approx. 20	—	—
Calcium oxide	Lime, quick lime	CaO	74.09	Off-white power or lump	561–801	—	approx. 1.3	CaO: approx. 95	—	12.6
Calcium hydroxide	Hydrated lime	$Ca(OH)_2$	56.08	Off-white faint slurry	—	—	approx. 1.8	CaO: approx. 71	approx. 24	12.6

Note: $kg/m^3 \times 0.00835 = lb/gal$.

most common use of chemical precipitation is for phosphorus removal. Chemical precipitation of phosphorus is one of the add-on processes for advanced wastewater treatment. Therefore, the chemistry of phosphorus removal, basic design parameters, equipment needs, specific application points, and design criteria are discussed under add-on processes for advanced wastewater treatment in Chapter 24. Basic theory of coagulation and flocculation is presented below.

Immediately after dispersion of a coagulation chemical in the wastewater, a reaction occurs with calcium and magnesium bicarbonate. Insoluble metal hydroxide, a gelatinous floc, is produced. This floc, if slowly stirred, sweeps out and entraps the colloidal particles that have been conditioned by charge neutralization. This floc grows into a larger mass and settles rapidly. The chemical reactions between alkalinity and coagulants are expressed by Eq. (12-2)–(12-4). The numbers below different substances are their molecular weights, and therefore, the equations also denote the stoichiometric relationship.

Stoichiometric Reactions. The chemical reactions with coagulants added and alkalinity present in water are given below:

Alum

$$\underset{666.6}{Al_2(SO_4)_3 \cdot 18\,H_2O} + \underset{\substack{3 \times 100 \\ (\text{as } CaCO_3)}}{3Ca(HCO_3)_2} \rightarrow \underset{2 \times 78}{2Al(OH)_3} + \underset{3 \times 136}{3CaSO_4} + \underset{6 \times 44}{6CO_2} + \underset{18 \times 18}{18H_2O} \tag{12-2}$$

Ferric Chloride

$$\underset{2 \times 162.1}{2Fe(Cl)_3} + \underset{\substack{3 \times 100 \\ (\text{as } CaCO_3)}}{3Ca(HCO_3)_2} \rightarrow \underset{2 \times 78}{2Fe(OH)_3} + \underset{3 \times 111}{3CaCl_2} + \underset{6 \times 44}{6CO_2} \tag{12-3}$$

Ferric Sulfate

$$\underset{400}{Fe_2(SO_4)_3} + \underset{\substack{3 \times 100 \\ (\text{as } CaCO_3)}}{3Ca(HCO_3)_2} \rightarrow \underset{2 \times 78}{2Fe(OH)_3} + \underset{3 \times 136}{3CaSO_4} + \underset{6 \times 44}{6CO_2} \tag{12-4}$$

Lime

$$Ca(OH)_2 + Ca(HCO_3)_2 \rightarrow CaCO_3 + 2H_2O \tag{12-5}$$

The following important observations can be made in coagulation reactions:

1. The number of bound water molecules with coagulants may vary.
2. The alkalinity and hardness in water is generally expressed as $CaCO_3$, which has a molecular weight of 100. Therefore, the molecular weight of $Ca(HCO_3)_2$ is expressed in terms of the molecular weight of $CaCO_3$.
3. In all these reactions the alkalinity is removed. The removal rate is dependent upon

the coagulant dosage. As an example, at an alum dosage of 40 mg/L, the alkalinity removal is $\frac{3 \times 100 \text{ g/mole } CaCO_3 \times 40 \text{ mg/L alum}}{666.6 \text{ g/mole alum}} = 18.0 \text{ mg/L } CaCO_3$.

4. Most wastewaters contain sufficient natural alkalinity (50–200 mg/L as $CaCO_3$) for coagulation reaction to occur. If sufficient alkalinity is not present, lime or soda ash are commonly added for this purpose.
5. The reaction is analogous when the alkalinity-causing compound is magnesium bicarbonate.
6. The hardness is not removed. It simply changes from carbonate hardness to noncarbonate hardness.
7. CO_2 is produced, and the pH is slightly lowered after coagulation.

12-5-3 Chemical Addition and Mixing

Chemical Feed. Special attention must be given to the design of chemical storage, feeding, pumping, and control systems because of the corrosive nature of the chemicals used. The chemicals may be used in dry or liquid form. Dry chemical feeders are either volumetric or gravimetric. The volumetric-type feeder delivers a predetermined volume while the gravimetric type delivers a known weight of dry chemical. The dissolving operation is critical as water supply is controlled, and a mechanical mixer is used to insure the formation of a constant strength solution. The metering pump delivers this solution to the application point at measured rates.

The liquid chemical feed systems include a concentrated solution storage tank, transfer pump, day tank for delivering the solution, and wet chemical feed pump for distribution. The storage tank size is based on the stability of the chemical, availability and supply, and the consumption rate.

Application Points. For enhanced suspended solids, BOD_5, and phosphorus removal in the primary sedimentation basin, the chemicals are applied upstream of the sedimentation basin. If an aerated grit chamber is used for flocculation, the chemical addition point is above the grit chamber.

Rapid Mixing. Thorough dispersion of coagulating chemicals is achieved in a rapid mix or flash mix basin. This is done by a static mixer or mechanical mixer. Static mixers create turbulence in the flow stream by baffles, obstruction, contraction, or enlargement of flow area or by creating a hydraulic jump in an open channel.

The mechanical mixers utilize an impeller or propeller to create turbulence in the mixing chamber. These impellers are of two types: (a) radial flow and (b) axial flow. The radial flow impellers force water outward at a right angle to the axis of rotation similar to a radial flow pump. Axial flow impellers have pitched blades and force water parallel to the axis of rotation similar to an axial flow pump. The type of impeller used in a rapid mixer is dependent upon the geometry of the basin and the flow pattern desired within the basin. Figure 12-16 illustrates radial and axial flow impellers and the typical flow pattern in a rapid mix unit that has a radial flow mechanical mixer.

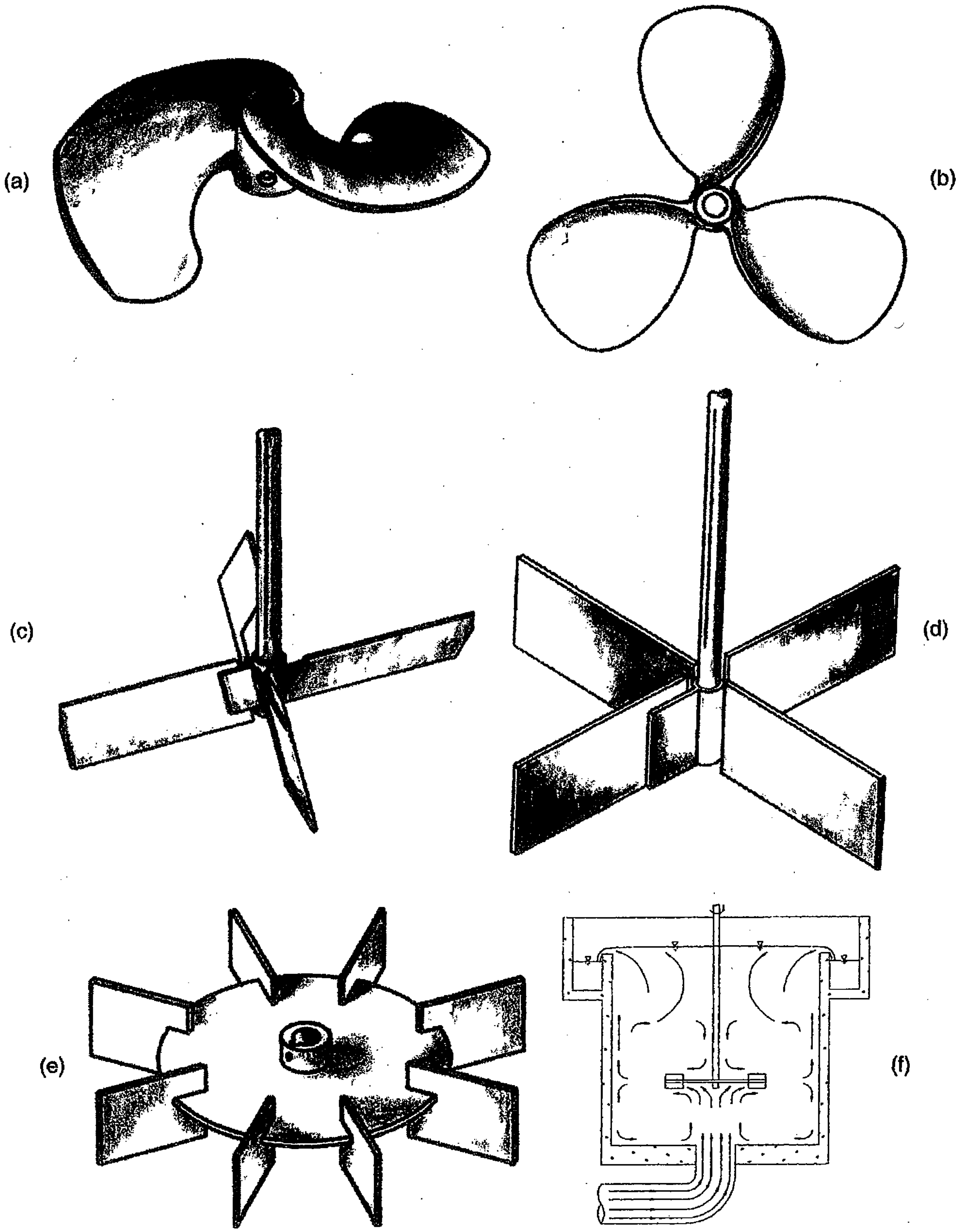


Figure 12-16 Radial and Axial Flow Impellers (courtesy Aqua-Aerobic Systems, Inc.) and Basin Geometry: (a) non-clog pitched-blade impeller, (b) marine-type pitched-blade impeller, (c) pitched-blade axial flow impeller, (d) flat-blade impeller, (e) flat-blade disk impeller, and (f) typical flow patterns in a radial flow mixer.

The degree of agitation in a mixing unit is measured by velocity gradient G. The velocity gradient for rapid mix impellers is generally in the range of 700 to 1000 per second. The velocity gradient is expressed by Eq. (12-6). The power imparted to the water by an impeller is given by Eqs. (12-7) and (12-8). Eq. (12-7) is used for laminar flow range (Reynolds number $N_R < 10$), and Eq. (12-8) is used for turbulent range ($N_R > 10{,}000$). The Reynolds number is expressed by Eqs. (11-4) and (12-9).

$$G = \left(\frac{P}{\mu V}\right)^{1/2} \tag{12-6}$$

$$P = N_p \mu n^2 d^3 \text{(laminar)} \tag{12-7}$$

$$P = N_p \rho n^3 d^5 \text{ (or } N_p\, n^3 d^5 \gamma/g\text{) (turbulent)} \tag{12-8}$$

$$N_R = d^2 n\rho/\mu \text{ (or } d^2 n\gamma/\mu g\text{)} \tag{12-9}$$

where

G = velocity gradient, s^{-1}
P = power imparted to the water, W or N·m/s (lb·ft/s)[f]
V = volume of the basin, m^3 (ft^3)
μ = dynamic viscosity of the fluid, $N{\cdot}s/m^2$ ($lb{\cdot}s/ft^2$)
n = impeller speed, revolutions per second (rps)
N_p = power number of the impeller (see Table 12-6)
d = impeller diameter, m (ft)
ρ = mass density of fluid, kg/m^3
γ = specific weight of water, N/m^3 (lb/ft^3)
g = acceleration due to gravity, m/s^2 (ft/s^2)
N_R = Reynolds number

12-5-4 Flocculation

The chemically stabilized colloidal particles after coagulation must be gently stirred to promote the growth of floc produced. This process is known as a flocculation process. Flocculation is also used in the precipitation process to promote growth of the precipitate into a rapidly settleable floc. Generally, in practice coagulation and precipitation are achieved in a rapid mix basin, and gentle mixing is carried out in a separate flocculation basin. Design considerations of a flocculation basin are given in this section.

The flocculators utilize static mixers or mechanical mixers. The static mixers usually employ baffles to produce the required turbulence. Even aerated grit chambers have been used for this purpose. Mechanical mixers used in flocculation basins are often paddle wheel, walking beam, or pulsation type. The typical velocity gradient G for a flocculator ranges from 20 to 50 per second. The velocity gradient for a mechanical mixer is calculated from Eq. (12-6). In the case of paddle wheel mixers, the water power

[f]N = mass × acceleration = $kg{\cdot}m/s^2$
Watt (W) = N·m/s or $kg{\cdot}m^2/s^3$

TABLE 12-6 Power Numbers of Various Rapid Mix Impellers

	Power Number (N_P)
Radial flow	
Straight blade turbine	2.6
4 blade (w/d = 0.15)	3.3
4 blade (w/d = 0.2)	
Disc turbine	
4 blade (w/d = 0.25)	5.1
6 blade (w/d = 0.25)	6.2
Axial flow	
Propeller 1:1 pitch	0.3
Propeller 1.5:1 pitch	0.7
45° Pitched blade	
4 blade (w/d = 0.15)	1.36
4 blade (w/d = 0.2)	1.94

Note: w/d = blade width to impeller diameter ratio.
Source: Adapted in part from Refs. 1, 2, and 4.

is calculated from Eq. (12-10). Use of this equation for calculation of the paddle wheel area is demonstrated in Chapter 16.

$$P = \frac{C_D A \gamma v^3}{2g} \text{ or } \frac{C_D A v^3 \rho}{2} \tag{12-10}$$

where

P = power requirement for mixing, W or N·m/s (lb·ft/s)
A = area of paddle, m^2 (ft^2)
C_D = coefficient of drag of flocculator paddles moving perpendicular to the fluid. For rectangular paddles, C_D is between 1.2 and 1.9.
v = velocity of the paddle relative to the fluid, m/s (ft/s); usually it is 0.7–0.8 times the paddle tip speed.
γ, ρ, g have been defined earlier

12-5-5 Chemically Enhanced Primary Sedimentation

The chemically assisted primary sedimentation basins are designed to remove both settleable organics and chemical floc. The basin design is generally based on surface loading. The recommended surface-loading rates for a chemically enhanced primary sedimentation basin is in the range of 69–81 $m^3/m^2{\cdot}d$ (1700–2000 gpd/ft^2).[2,29] These rates are almost twice the overflow rate of a conventional primary sedimentation basin (30–50 $m^3/m^2 \cdot d$). Research has shown that an overflow rate up to 98 $m^3/m^2 \cdot d$ (2400 gpd/ft^2) does not significantly affect the effluent quality of a chemically enhanced primary sedimentation basin.[30]

12-6 EQUIPMENT MANUFACTURERS OF SEDIMENTATION BASINS

Many manufacturers of pollution control equipment construct or assemble the hardware for sedimentation basins. These manufacturers specialize in different types of designs. The names and addresses of several manufacturers of sedimentation equipment are given in Appendix D. Basic considerations for equipment selection are covered in Sec. 2-10.

12-7 INFORMATION CHECKLIST FOR DESIGN OF A PRIMARY SEDIMENTATION BASIN

The following information is necessary for designing a primary sedimentation facility:

1. Average and peak design flows, including the returned flows from other treatment units
2. All sidestreams from thickener, digester, and dewatering facilities to be considered if flows are returned ahead of primary sedimentation
3. Treatment plant design criteria prepared by the concerned regulatory agencies
4. Equipment manufacturers and equipment selection guide (catalog)
5. Information on the existing facility if the plant is being expanded
6. Available space and topographic map of the plant site
7. Shape of the tank (rectangular, square, or circular)
8. Influent pipe data, to include diameter, flow characteristics, and approximate water surface elevation or hydraulic grade line
9. Head loss constraints for sedimentation facility

12-8 DESIGN EXAMPLE

12-8-1 Design Criteria Used

The following design criteria shall be used for the Design Example:

1. Two rectangular units shall be designed for independent operation. A bypass to the aeration basin shall be provided for emergency conditions when one unit is out of service. Most regulatory agencies will allow such bypass. This means that under peak wet weather flow of 1.321 m^3/s if one basin is out of service, the other sedimentation basin will receive a flow of only 0.661 m^3/s. The remaining flow of 0.661 m^3/s will be bypassed to the aeration basin.
2. Overflow rate and detention time shall be based on an average design flow of 0.44 m^3/s (0.22 m^3/s through each unit)
3. The overflow rate shall be less than 36 $m^3/m^2 \cdot d$ (at average design flow).
4. The detention time shall be not less than 1.5 h (at average design flow).

5. The influent structure shall be designed to prevent short-circuiting and reduce turbulence. The influent channel shall have a velocity less than 0.35 m/s at design peak flow (0.661 m^3/s through each basin).
6. All sidestreams shall be returned to aeration basins.
7. The weir loading shall be less than 372 m^3/m·d at peak design flow.
8. The launder and outlet channels shall be designed at the peak design flow of 1.321 m^3/s (0.661 m^3/s through each unit).
9. The average liquid depth in the basin shall be no less than 3 m.
10. The slope of the tank bottom shall be 1.35 percent.

12-8-2 Design Calculations

Step A: Basin Dimensions

1. Select basin geometry, and provide two rectangular basins with common wall.[g]

- Average design flow through each basin = 0.22 m^3/s
- Overflow rate at average design flow = 36 m^3/m^2·d
- Surface area $= \dfrac{0.22\ \text{m}^3/\text{s} \times 86{,}400\ \text{s/d}}{36\ \text{m}^3/\text{m}^2\cdot\text{d}} = 528\ \text{m}^2$
- Use length-to-width ratio $L:W::4:1$ (see Table 12-3)
- $(4W)(W) = 528\ \text{m}^2$
  $W = 11.5$ m

Since most of the equipment manufacturers' assembled equipment is in increments of 61 cm (2 ft), therefore:

Width	= 11.58 m (38 ft)
Length	= 46.33 m (152 ft)
Length-to-width ratio	= 4:1
Provide average water depth at mid-length of the tank	= 4.0 m (13.1 ft)
Length-to-depth ratio	= 46.33 m/4.0 m = 11.6 (acceptable, see Table 12-3)
Freeboard	= 0.6 m (2 ft)
Average depth to the top of the basin	= 4.6 m (15 ft)

2. Check overflow rate

$$\text{Overflow rate at average design flow} = \frac{0.22\ \text{m}^3/\text{s} \times 86{,}400\ \text{s/d}}{11.58\ \text{m} \times 46.33\ \text{m}}$$

$$= 35.4\ \text{m}^3/\text{m}^2\cdot\text{d}$$

[g]For illustration purposes, the design procedure for the rectangular primary sedimentation basins is given in this chapter. The design procedure for circular clarifiers is provided in Chapter 13.

$$\text{Overflow rate at peak design flow} = \frac{0.661 \text{ m}^3/\text{s} \times 86{,}400 \text{ s/d}}{11.58 \text{ m} \times 46.33 \text{ m}} = 106.4 \text{ m}^3/\text{m}^2\cdot\text{d}$$

3. Check detention time

$$\text{Average volume of the basin} = 4.0 \text{ m} \times 11.58 \text{ m} \times 46.33 \text{ m} = 2146.0 \text{ m}^3$$

$$\text{Detention time at average design flow} = \frac{2146.0 \text{ m}^3}{0.22 \text{ m}^3/\text{s} \times 3600 \text{ s/h}} = 2.7 \text{ h}$$

$$\text{Detention time at peak design flow} = \frac{2146.0 \text{ m}^3}{0.661 \text{ m}^3/\text{s} \times 3600 \text{ s/h}} = 0.9 \text{ h}$$

Step B: Influent Structure

1. Select the arrangement of the influent structure.
   The influent structure includes a 1-m-wide influent channel that runs across the width of the tank. Eight submerged orifices, 34 cm (13 in.) square each, are provided in the inside wall of the channel to discharge the flow into the basin. The purpose of these orifices is to distribute the flow over the entire width of the basin. A submerged influent baffle is provided 0.8 m in front, 1.0 m deep, and 5 cm below the liquid surface. The influent structure is shown in Figure 12-17.
2. Compute the head loss in the influent pipe connecting the junction box located downstream of the grit chamber and the influent structure of the sedimentation basin (Figure 12-17).
   The influent pipe to the primary sedimentation basin connects the junction box downstream of the grit chamber with the influent channel. Normally, a straight sewer line or pressure pipe is used to avoid solids deposition in the pipe. The elevation of the water surface in the influent channel of the basin is lower than the water surface elevation in the junction box downstream from the grit chamber. The difference (ΔH) is the sum of the head loss in the connecting pipe due to the entrance, friction, bends, and fittings and exit loss into the influent channel of the sedimentation basin. These losses are calculated in Chapter 21 when the hydraulic profile is prepared through the entire treatment plant.
3. Compute the head losses at the influent structure.
   Chao and Trussell have provided the design procedure of a distribution channel.[31] For a precise mathematical solution of losses at the influent structure, the boundary layer theory and energy equations for manifolds are used.[32-34] In this design only an approximate solution using the energy equation is given. Refer to Eq. (11-1) and sections (1) and (2) as defined in Figure 12-17(b). The horizontal velocity in the sedimentation basin (v_2) is small and is ignored. The average velocity in the influent channel (v_1) is calculated at peak design flow. Half of the flow divides on each side of the basin.

$$\text{Discharge in each channel} = \frac{\text{peak design flow per basin}}{2} = \frac{0.661 \text{ m}^3/\text{s}}{2} = 0.331 \text{ m}^3/\text{s}$$

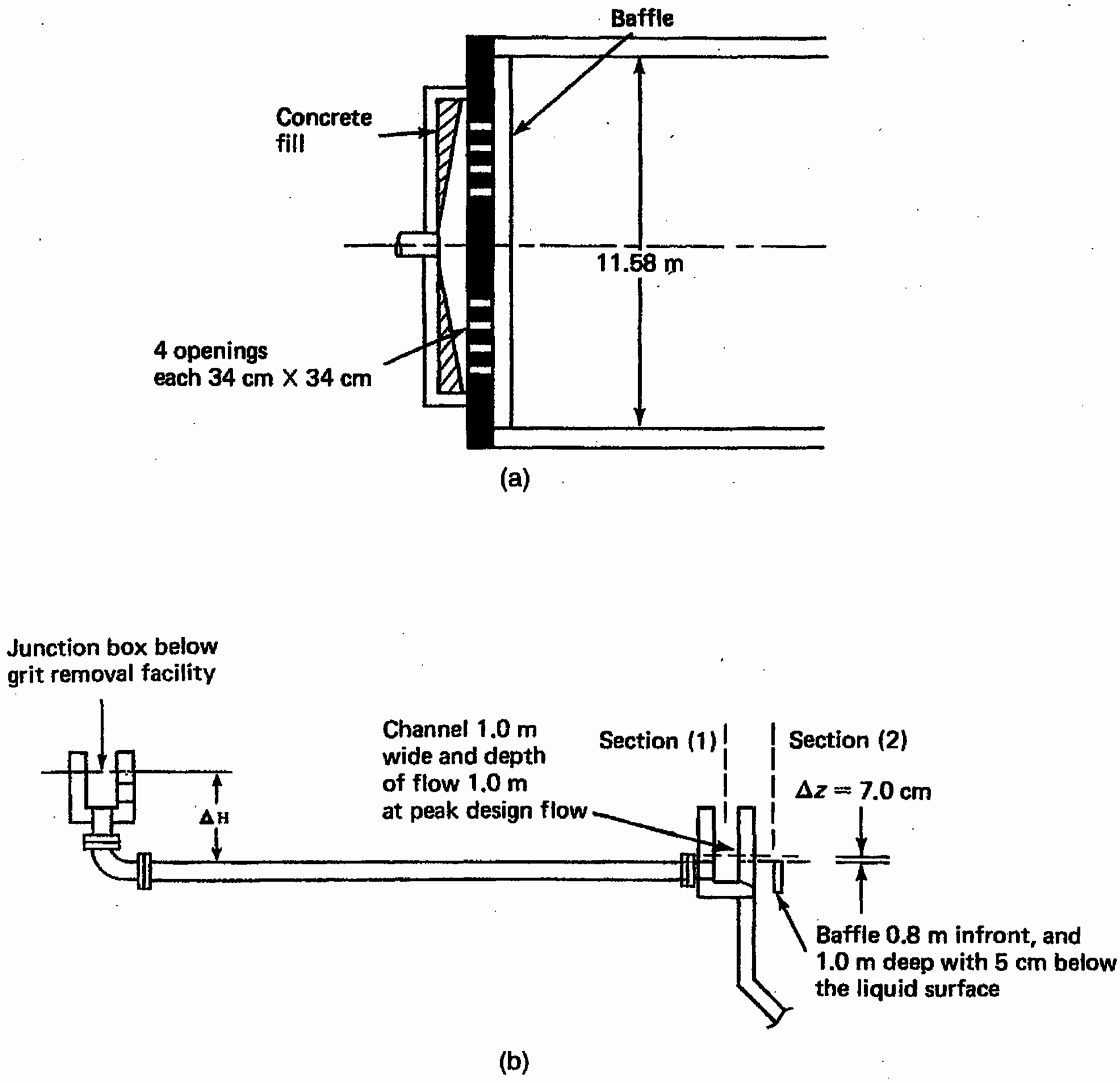


Figure 12-17 Example of Influent Structure and Piping Details: (a) plan of influent structure and (b) connecting pipes from junction box below the grit removal facility to the influent structure of the sedimentation basin.

The depth of water into the influent channel is fixed by the designer. Suppose the depth of water at the entrance of the influent channel is 1.0 m and the width of the influent channel is 1.0 m.

$$\begin{array}{c}\text{Velocity in the channel}\\ \text{at peak design flow}\end{array} = \frac{0.331 \text{ m}^3/\text{s}}{1 \text{ m} \times 1 \text{ m}} = 0.331 \text{ m/s}$$

The velocity in the channel will change as the flow is successively reduced at various orifices. At the same time the width of the channel and depth of flow away from the entrance of the influent pipe are also reduced. Consequently, the discharge and velocity in the channel and through the submerged orifices will vary.[35] Using approximation, the term ΔH in Eq. (11-9) is equal to the sum of friction losses into the channel and the head loss through the short pipes, referred to as submerged orifices. The discharge through each orifice = $(0.661\ m^3/s)/8 = 0.083\ m^3/s$. Using Eq. (11-9) for a submerged orifice, ΔH is calculated as follows:

$$\Delta H^{h} = \left[\frac{0.083\ m^3/s}{0.6 \times (0.34^{i}\ m)^2 \times \sqrt{2 \times 9.81\ m/s^2}}\right]^2$$

$$= 0.07\ m$$

Step C: Effluent Structure

1. Select the arrangement of the effluent structure.
   The effluent structure consists of weirs, launder, and outlet box and an outlet pipe. The weirs are either sharp-crested, freefalling, or V-notch [Figure 12-18(a)]. The total length of the weir is calculated from the weir-loading rate. The weir length is normally large and may cover a significant portion of the basin area. Normally, the weir plate is installed over concrete walls that form the launder. Often, metal weirs and launder are made as an integral part and then installed into the tank. The weir edge must be kept perfectly leveled to ensure uniform water depth at the notches or over the weir crest.

   If straight-edge weirs are provided, the head calculations using the classical weir equations give extremely small heads over the weirs (1–2 mm). At such small heads the capillary clinging effects at the weir crest are significant, and the use of the classical weir equations does not provide satisfactory head calculations.[32] At small heads the solids and slime tend to accumulate at the weir crest. Also, if the weir is not perfectly level, portions of the weir do not have any flow, which creates nonuniform loading and solids accumulation problems. Due to these difficulties, generally V-notch weirs are preferred in the design of sedimentation basins.
2. Compute the length of the weir.

$$\text{Weir loading} = 372\ m^3/m \cdot d \text{ at peak design flow}$$

$$\text{Peak design flow per basin} = 0.661\ m^3/s \times 86{,}400\ s/d = 57{,}110\ m^3/d$$

$$\text{Weir length} = \frac{57{,}110\ m^3/d}{372\ m^3/m \cdot d} = 153.3\ m$$

Provide weir notches on both sides of the launder, as shown in Figure 12-18(b):

$$\text{Total length of the weir plate} = 2(29.5 + 10.38)\ m + 2(28.3 + 9.18)\ m - 1.0\ m$$

$$= 153.72\ m$$

$$\text{Actual weir loading} = \frac{57{,}110\ m^3/d}{153.72\ m} = 371.6\ m^3/m \cdot d$$

[h]Various terms are defined in Eq. (11-9).

[i]The orifice is 34 cm × 34 cm.

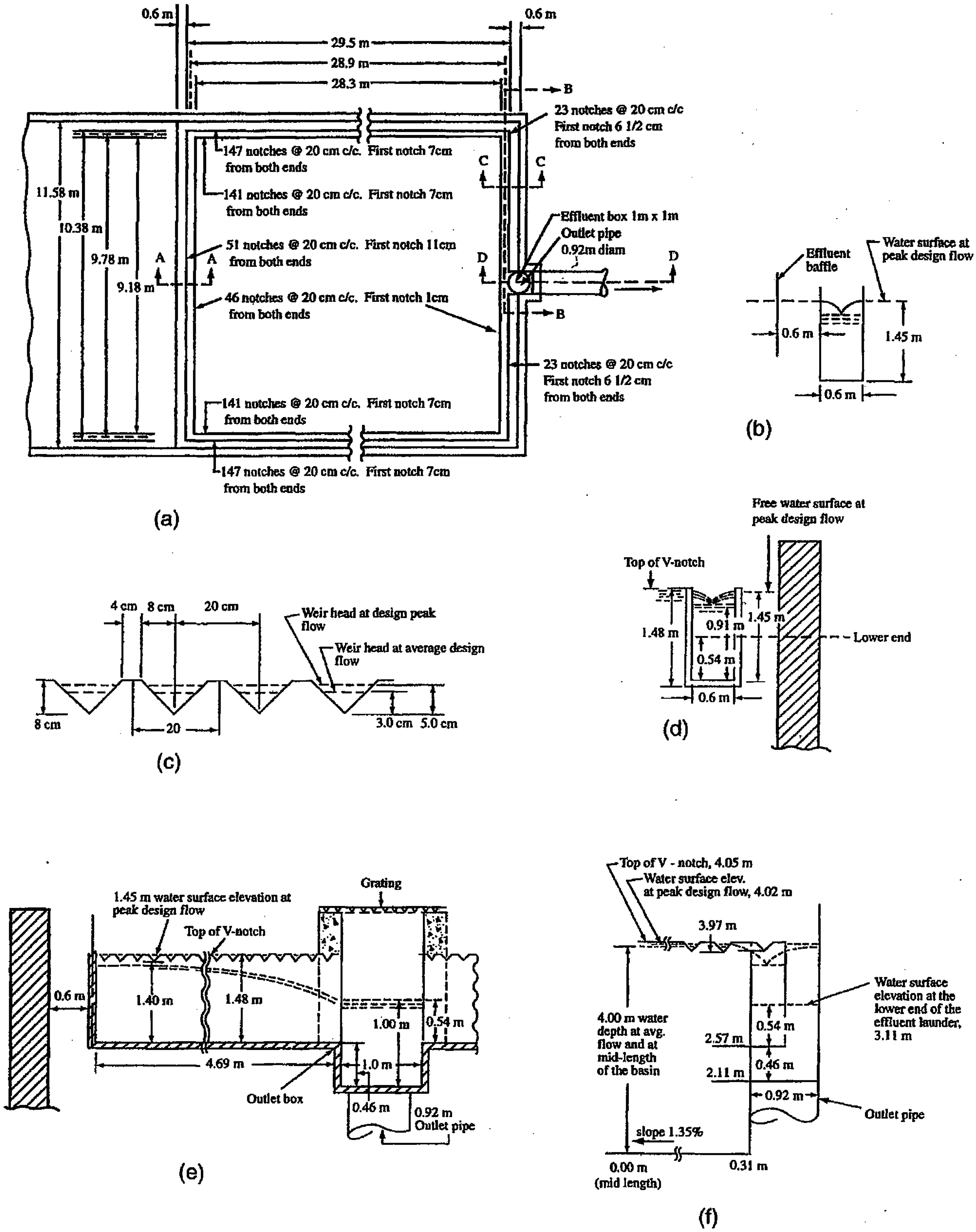


Figure 12-18 Weir Arrangement and Design Dimensions of the Effluent Structure: (a) plan view, weir arrangement; (b) section AA, weir trough (effluent launder) with weir notches on both sides; (c) details of the V-notches; (d) section CC, details of the effluent launder; (e) section BB, water surface profile in the effluent launder; and (f) section DD, details of the outlet channel. All Elevations Are with Respect to the Floor Level of the Basin at Midlength.

3. Compute the number of V-notches.
   Provide 90° standard V-notches at a rate of 20 cm center to center on both sides of the launders. The weir arrangement and details of the notches are shown in Figure 12-18.

$$\text{Total number of notches} = 5 \text{ notches per m} \times 153.72 \text{ m} = 769$$

   In order to leave sufficient space on the ends of the weir plate, provide a total of 765 notches. The arrangement of V-notches is shown in Figure 12-18(a).
4. Compute the head over the V-notches at the average design flow.

$$\begin{aligned}\text{The average discharge per notch at average design flow} &= \frac{0.22 \text{ m}^3\text{/s}}{765 \text{ notches}} \\ &= 2.88 \times 10^{-4} \text{ m}^3\text{/s per notch}^{j}\end{aligned}$$

   The discharge through a V-notch is given by Eq. (12-11):[32-34]

$$Q = \frac{8}{15} C_d \sqrt{2g} \tan \frac{\theta}{2} H^{5/2} \tag{12-11}$$

   where

Q = flow per notch, m^3/s
C_d = coefficient of discharge = 0.6
H = head over notch, m
θ = angle of the V-notch = 90°

   The head over the notches at average design flow is obtained as follows:

$$2.88 \times 10^{-4} = \frac{8}{15} \times 0.6 \times \sqrt{2 \times 9.81 \text{ m/s}^2} \tan \frac{90}{2} H^{5/2}$$

   where

$$H = 0.03 \text{ m} = 3.0 \text{ cm}$$

5. Compute head over V-notches at peak design flow.

$$\text{Discharge per notch at peak design flow} = \frac{0.661 \text{ m}^3\text{/s}}{765} = 8.64 \times 10^{-4} \text{ m}^3\text{/s}$$

$$8.64 \times 10^{-4} \text{ m}^3\text{/s} = \frac{8}{15} \times 0.6 \sqrt{2 \times 9.81 \text{ m/s}^2} \tan \frac{90}{2} H^{5/2}$$

   where

$$H = 0.05 \text{ m} = 5.0 \text{ cm}$$

[j] Flow over weir = average design flow − sludge withdrawal rate. Sludge withdrawal rate is small and is ignored in these calculations.

6. Check the depth of the notch.
   The total depth of the notch is 8 cm. Maximum liquid head over the notch at peak design flow is 5.0 cm. This gives a safe allowance of 3.0 cm (1.2 in.) against submergence.
7. Compute the dimensions of the effluent launder.
   The approximate solution of the water profile in the effluent launder is obtained from the equation developed for flumes with level inverts and parallel sides. This equation was discussed in Chapter 11 [Eq. (11-17)]. A more accurate solution may be obtained by using Eqs. (11-12)–(11-16). A computer program for the solution of these equations is given in Ref. 23.

   The effluent launder discharges into the effluent box. An outlet pipe is connected to the effluent box that carries the primary treated effluent to the next junction-splitter box preceding the aeration basin. The dimensions of the effluent launder, effluent box, outlet pipe, and the water surface elevations in the effluent launder and effluent box are shown in Figure 12-18. The following dimensions may be noted:[k]

Width of the launder b	= 0.6 m
Width of the effluent box	= 1.0 m
Diameter of the outlet pipe	= 0.92 m
The depth of water in the effluent box[l]	= 1.00 m

Provide the invert of the effluent launder 0.46 m above the invert of the effluent box.

$$\text{Depth of water in the effluent launder at the exit point, } y_2 = 1.0\text{ m} - 0.46\text{ m} = 0.54\text{ m}$$ [m]

Half of the flow divides on each side of the launder.

$$\text{Flow on each side of the launder at exit point} = \frac{0.661\text{ m}^3/\text{s}}{2} = 0.33\text{ m}^3/\text{s}$$

$$y_1 = \sqrt{(0.54\text{ m})^2 + \frac{2(0.33\text{ m}^3/\text{s})^2}{9.81\text{ m/s}^2(0.60)^2 \times 0.54\text{ m}}} = 0.64\text{ m}$$ [n]

Generally, an allowance for losses due to friction, turbulence, and bends is 10–30 percent. In this case, provide an allowance of 25 percent. Add a liberal free fall of 0.6 m.

[k]Various terms are defined in Eq. (11-17).

[l]The water depth in the effluent box is fixed by the designer. In this case the depth of water in the effluent box is equal to the diameter of the outlet pipe plus the entrance and other losses. Calculations for these losses are given in Chapter 21.

[m]The critical depth of flow in the launder from Eq. (8-6) at exit point is 0.5 m. Since the actual depth at the lower end of the launder is greater than the critical depth, the outfall is submerged.

[n]The water depth at the upstream end of the effluent launder from Eqs. (11-12)–(11-16) is 0.644 m (see Problem 12-12).

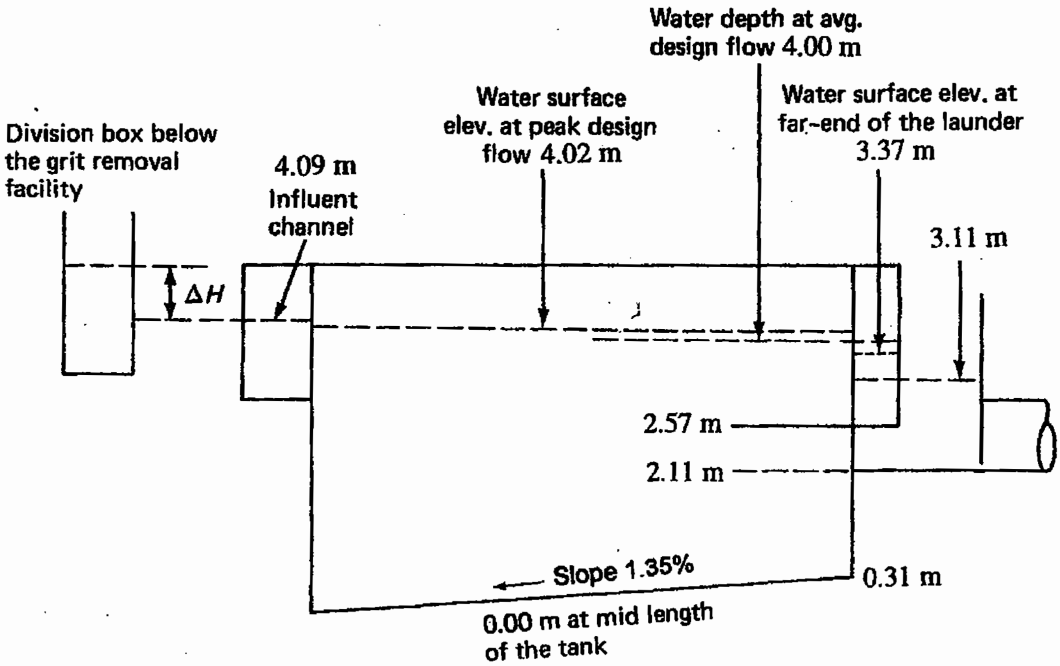


Figure 12-19 Hydraulic Profile Through Primary Sedimentation Basin at Peak Design Flow. (The hydraulic profile is prepared with respect to the assumed datum at the bottom of the basin at midlength.)

$$\text{Water depth at the far end of the trough} = 0.64\ \text{m} \times 1.25\ \text{m} = 0.80\ \text{m}$$

$$\text{Total depth of the effluent launder} = 0.80\ \text{m} + 0.60\ \text{m} = 1.40\ \text{m}$$

In this design a 1.40-m-deep effluent launder can handle 1.321 m^3/s flow without surcharge. This situation may develop when one basin is removed from service and the entire wet weather flow is routed through one basin. The details of the effluent structure and various dimensions are given in Figure 12-18.

Step D: Head Loss Through Sedimentation Basin. The total head loss through the primary sedimentation basin includes (1) head losses at the influent structure, (2) head losses at the effluent structure, (3) head loss in the basin, and (4) head losses at the influent and effluent baffles.

The head loss calculations for the influent and effluent structures are given above. The head loss through the basin is small and is ignored. The influent and effluent baffles offer little obstruction to the flow. The momentum equation can be used to determine the head losses due to the baffles.[32-34] The use of the momentum equation [Eq. (11-18)] for the calculation of head losses due to the baffles has been shown in Chapter 11. These losses are small and can be ignored.

Step E: Hydraulic Profile Through the Basin. Figure 12-19 illustrates the hydraulic profile through the sedimentation basin at peak design flow. A total head loss of 0.99 m (3.25 ft) occurs at the peak design flow.

Step F: Sludge Quantities

1. Establish sludge characteristics.
   It is desirable to produce a sludge that is as thick as possible. Normally, the primary sludge has a specific gravity of 1.03 and a solids content of 3–6 percent. The sludge produced from the facility designed in this example will have a typical solids content of about 4.5 percent.
2. Compute the average quantity of sludge produced per day.

$$\text{Amount of solids produced per basin per day at a removal rate of 63 percent}^{a} = 260\ \text{g/m}^3 \times (0.63) \times 0.22\ \text{m}^3/\text{s} \times 86{,}400\ \text{s/d} \times \text{kg}/1000\ \text{g} = 3113.5\ \text{kg/d}$$

$$\text{Average quantity of sludge produced per day from both basins} = 2 \times 3113.5\ \text{kg/d} = 6227\ \text{kg/d}$$

3. Compute the volume of sludge produced per minute per basin.

$$\text{Volume of sludge at specific gravity of 1.03 and 4.5 percent solids} = \frac{3113.5\ \text{kg/d per basin}}{1.03 \times \dfrac{1\text{g}}{\text{cm}^3} \times \dfrac{1}{1000\ \text{g/kg}} \times 0.045 \times (100\ \text{cm})^3/\text{m}^3 \times 1440\ \text{min/d}}$$

$$= 0.0467\ \text{m}^3/\text{min per basin}$$

4. Determine sludge pump size and pumping cycle.
   Provide separate pumps for each basin. Arrange piping such that each pump will be able to serve both basins in case one pump is out of service. Operate each pump on a time cycle, at 16.5-min intervals with a 1.5-min pumping cycle per basin (total time is 18 min per cycle).

$$\text{The desired pumping capacity of the pump} = \frac{0.0467\ \text{m}^3/\text{min per basin} \times 18\ \text{min per cycle}}{1.5\ \text{min pumping per cycle}}$$

$$= 0.56\ \text{m}^3/\text{min per basin (150 gpm)}$$

When one pump is used to remove the sludge from two basins, the pump cycle time is calculated below.

$$\text{Cycle interval in min for two basins} = \frac{0.56\ \text{m}^3/\text{min} \times 1.5\ \text{min pumping per cycle}}{0.0934\ \text{m}^3/\text{min for both basins}}$$

$$= 9\ \text{min per cycle}$$

The pump cycle time of 9 min will be alternated between the two basins.

Step G: Effluent Quality from Primary Sedimentation Basin

1. Establish BOD_5, TSS, organic nitrogen, ammonia nitrogen, total nitrogen, and total phosphorus removal.

[a] Influent TSS = 260 mg/L (see Table 6-9).

The percent removal of BOD_5 and suspended solids in a well-designed and -operated primary sedimentation basin may be estimated from Figures 12-7 or 12-8. Based on the actual overflow rate of 35.4 $m^3/m^2 \cdot d$, the following removal rates are expected:

$$\begin{aligned} BOD_5 \text{ removal} &= 34 \text{ percent} \\ \text{Suspended solids removal} &= 63 \text{ percent} \end{aligned}$$

The organic nitrogen and total phosphorus removal are 30 and 16 percent, respectively (Table 4-2). There is no ammonia nitrogen removal.

2. Compute BOD_5, TSS, Org.-N, NH_4^+-N, TN, and TP in the primary sludge and primary effluent. Based on the concentration of BOD_5 and suspended solids, Org.-N, NH_4^+-N, TN and TP in the raw wastewater, and average removal rates, the effluent quality from the primary basins is determined. It should be mentioned, however, that if the flows from the thickeners, digesters, and dewatering facilities are returned to the primary sedimentation basin, the load contributed by these return flows must be estimated properly by using material mass balance techniques. Many designers overlook this fact, which may cause erroneous effluent quality estimates and unit design. The material mass balance analysis for the Design Example may be found in Chapter 13. In these calculations the sidestream from the thickeners, digesters, and dewatering facilities is returned to the aeration basin. Thus, the primary treatment facilities do not receive any additional load. Readers should refer to Figure 13-25 and Table 13-13 for more information on material mass balance analysis.

$$\begin{aligned} &\text{Influent flow to the} \\ &\text{primary sedimentation basin} = 0.440 \text{ m}^3/\text{s} \times 86{,}400 \text{ s/d} = 38016 \text{ m}^3/\text{d} \end{aligned}$$

$$\begin{aligned} &\text{TSS reaching the} \\ &\text{primary sedimentation basin} = 260 \text{ g/m}^3 \times 38016 \text{ m}^3/\text{d} = 9884 \text{ kg/d} \end{aligned}$$

$$\text{TSS in primary sludge} = 9884 \text{ kg/d} \times 0.63 = 6227 \text{ kg/d}$$

$$\text{Volume of primary sludge} = \frac{6227 \text{ kg/d}}{0.045 \text{ g/g} \times 1030 \text{ kg/m}^3} = 134.3 \text{ m}^3/\text{d}$$

The total mass and concentration of various pollutants in raw wastewater, primary effluent, and primary sludge are summarized in Table 12-7. Readers may find additional information in Table 13-13.

Step H: Scum Quantity. Average quantity of scum is 8 kg/1000 m^3

$$\begin{aligned} \begin{matrix}\text{Average quantity of scum} \\ \text{having sp. gr. of 0.95}\end{matrix} &= \frac{8 \text{ kg}}{1000 \text{ m}^3} \times 38{,}016 \text{ m}^3/\text{d} = 304 \text{ kg/d} \\ &= \frac{304 \text{ kg/d}}{950 \text{ kg/m}^3} \\ &= 0.32 \text{ m}^3/\text{d} \end{aligned}$$

TABLE 12-7 Characteristics of Raw Wastewater, Primary Treated Effluent, and Primary Sludge

Components	Raw Wastewater		Primary Sludge		Primary Effluent	
Flow, m^3/d	38016		134		37882	
	Concentration (mg/L)	Mass (kg/d)	Removal (%)	Mass (kg/d)	Mass (kg/d)	Concentration (g/m^3 or mg/L)
BOD_5	250	9504	34	3,231	6273	166[a]
TSS	260	9884	63	6227	3657	97
Org.-N	17	646	30	194	452	12
NH_3-N	19	722	0	2.6[b]	720	19
NO_3-N	0	0	0	0	0	
TN[c]	36	1369	—	197	1172	31
TP	6	228	16	37	192	5

[a]$BOD_5 = 6273\ \text{kg/d} \times 1000\ \text{g/kg} \times 1/37882\ \text{m}^3/\text{d} = 165.6\ \text{g/m}^3$
[b]Soluble ammonia in primary sludge $= 134.3\ \text{m}^3/\text{d} \times 19\ \text{g/m}^3 \times (1000\ \text{g/kg})^{-1} = 2.6\ \text{kg/d}$
[c]TN = Org.-N + NH_4^+-N + NO_3^--N
Note: The values are rounded to whole number.

Step I: Design Details. The design details of a primary sedimentation facility are given in Figure 12-20.

12-9 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT PRIMARY SEDIMENTATION FACILITIES

The primary sedimentation facilities can create serious odors if the units are not carefully operated and maintained. When wastewater reaches the primary sedimentation basin, many odorous compounds are released into the atmosphere because these tanks offer a relatively large exposed surface area and there is a certain amount of turbulence at the influent and effluent structures. Furthermore, scum on the surface and at the collection devices offers highly favorable conditions for odor emission from the primary sedimentation basins. Settled sludge, if not pumped frequently, continues to undergo anaerobic breakdown, thus intensifying the odor problems. Improper operation of the primary sedimentation tank may cause solids and BOD overloading in the secondary processes, and grease carryover may upset the biological processes in particular. Therefore, proper operation and maintenance of primary treatment facilities is essential for control of odors and smooth operation of downstream treatment units. The following items are considered important for the design and operation of the primary treatment facilities.

12-9-1 Common Operational Problems and Troubleshooting Guide

A number of operational problems may occur at a primary sedimentation facility. Some troubleshooting guidelines are given below:[36,37]

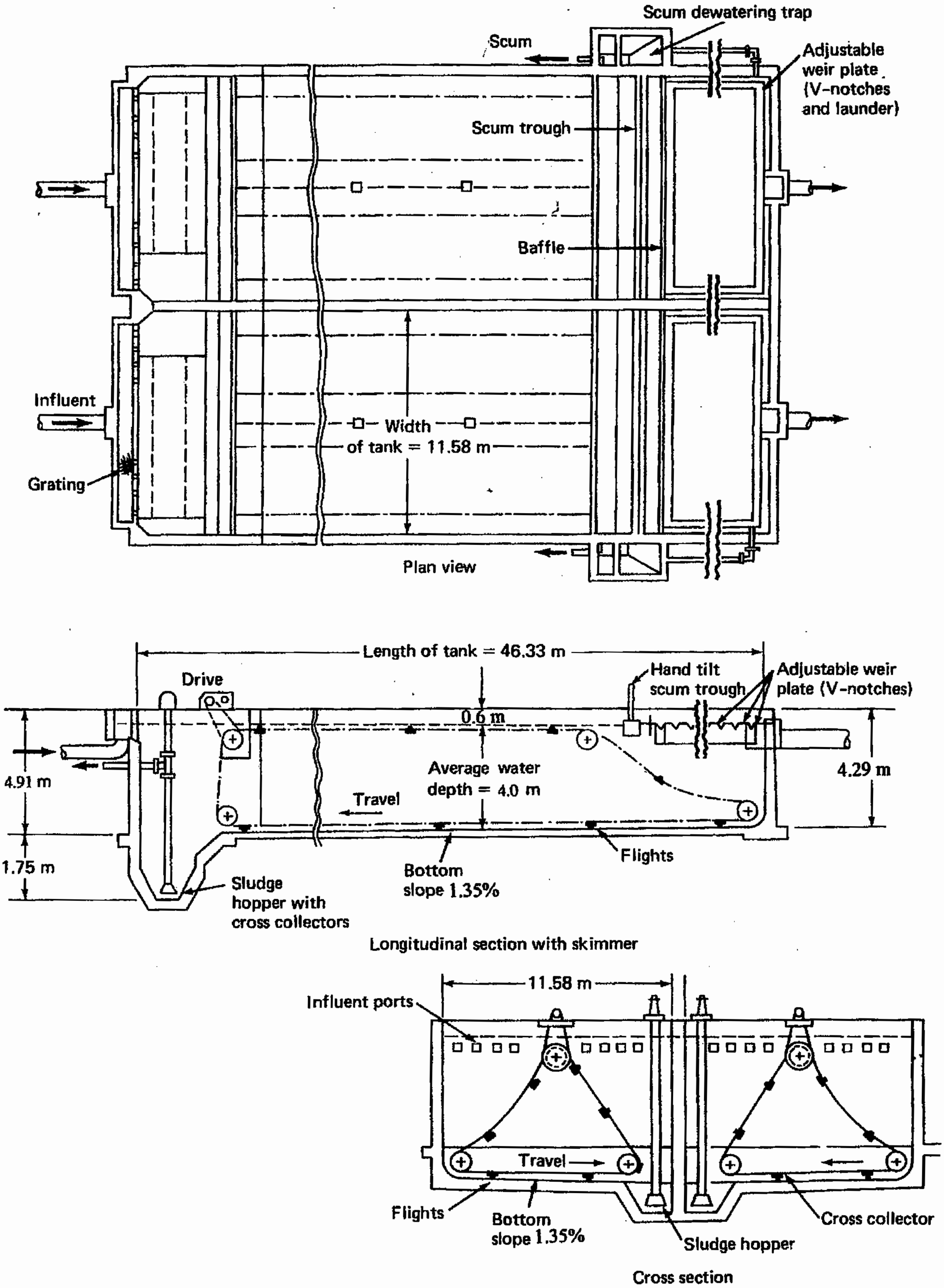


Figure 12-20 Layout of the Primary Sedimentation Basins in the Design Example (for equipment detail see Figure 12-1).

1. Black and odorous septic wastewater in the primary treatment facility is an indication of decomposing wastewater in the collection system, recycling excessively strong digester supernatant, or inadequate pretreatment of organic discharges from the industries. Possible solution procedures include preaeration, chlorination or hydrogen peroxide treatment of wastes in the collection system, control of digester supernatant, and strict enforcement of industrial pretreatment regulations.
2. Floating sludge indicates excessive sludge accumulation in the basin, decomposing organics, or return of well-nitrified, waste-activated sludge. The scrapers may be worn or damaged, the sludge withdrawal line may be plugged, or the sludge withdrawal rate may be insufficient. Remove the sludge more frequently or at a higher rate, clean the sludge lines, or repair or replace sludge collection and pumping equipment.
3. Scum overflow is caused by inadequate frequency of scum removal, excessive industrial contribution, worn or damaged scum wiper blades, or improper alignment of the skimmer. Remove scum more frequently, limit industrial waste contribution, clean or replace wiper blades, and adjust wiper blade alignment.
4. Sludge that is hard to remove from the sludge hopper may be caused by excessive grit accumulation. Check the grit removal facility.
5. Low solids in the sludge may be caused by excessive sludge withdrawal, short-circuiting, or surging flow. Reduce sludge withdrawal, check and install baffles, and check and modify influent pumping rate.
6. Excessive sedimentation in the influent channel is caused by low velocity. Agitate the influent channel with air or wastewater to resuspend solids and prevent decomposition.
7. Excessive slime growth on the surfaces and weirs may be due to accumulation of solids and scum. Inspect surfaces and clean them frequently.
8. Excessive corrosion of metals may be caused by hydrogen sulfide gas. This may be because of septic sewage or sludge. Check items 1 and 2 above. Paint surfaces with corrosion-resistant paint.
9. Erratic operation of a sludge collection mechanism may be caused by a broken shear pin or damaged collection mechanism, excessive sludge accumulation, or rags or debris entangled around collector mechanism. Repair or replace damaged parts, remove debris, and increase sludge pumping rate.
10. Frequent broken scraper chain and shear pin failures are caused by improper shear pin sizing and flight alignment, ice formation, or excessive loading on the sludge scraper. Realign flights and change shear pin size, break ice, or remove sludge more often.
11. A noisy chain drive, a chain that climbs sprockets, or a loose or stiff chain may be caused by misalignment or improper assembly, worn out parts, faulty lubrication, or excessive rust or corrosion. Inspect and correctly align the entire drive mechanism; replace the chain, bearings, or sprockets and outer parts; remove dirt and rust; and lubricate properly.
12. A broken chain or broken sprockets in the chain drive system may be caused by

shock, overloading, wrong chain size, misalignment, excessive wear, or lack of lubrication. Replace parts, avoid shock and overload, correct corrosive condition, and lubricate properly.

13. Bearing or universal joint failure is caused by excessive wear and lack of lubrication. Replace joints or bearing, and lubricate properly.

12-9-2 Operation and Maintenance

The following operation and maintenance items are necessary to keep the primary sedimentation facility in satisfactory operating condition:

1. Remove accumulations from influent and effluent baffles, weirs, and scum box. Clean all inside exposed walls and channels by squeegee on a regular basis.
2. Inspect all mechanical equipment at least once each shift.
3. Hose down and remove all wastewater and sludge spills as soon as possible.
4. Determine sludge level and underflow concentration and adjust primary sludge pumping rate accordingly. Observe scum pump operation and provide hosing as required. Both these items are considered regular duties.
5. Check daily the electrical motors for overall operation and bearing temperature. Also check the electrical motor overload detector daily.
6. Check oil levels in gear reducers and bearings on a regular basis.
7. Drain each primary basin annually and inspect the underwater portion of the concrete structure and all mechanical parts. Patch up defective concrete. Inspect all mechanical parts for wear and corrosion, and set proper clearance for flights at tank walls. Replace flights when necessary and supply protective coatings. Clean and paint the exposed metal surfaces as necessary.

12-10 SPECIFICATIONS

The specifications for the primary sedimentation facility designed in this chapter are briefly summarized here. The purpose of this section is to describe many components of the design that could not be fully covered in the Design Example. These specifications should be used only as a guide. Detailed specifications must be prepared for each design in consultation with the equipment manufacturers.

12-10-1 General

Each primary sedimentation basin shall include a complete assembly of a sludge collector mechanism with drive and collector chains with flights, access bridge and walkway, influent and effluent structures, pumping facilities, and overload alarm system. The manufacturer shall furnish and deliver ready for installation a conveyor-type sludge collector mechanism suitable for installation in two identical rectangular primary sedimentation basins of the following dimensions:

Number of Basins	Two Identical Basins with Common Wall
Length	46.33 m (152 ft)
Width	11.58 m (38 ft)
SWD[p] at effluent end	3.69 m (12.1 ft)
SWD at influent end	4.31 m (14.1 ft)
SWD at midlength	4.0 m (13.1 ft)
Bottom slope	1.35 percent
Freeboard	0.6 m (2 ft)

12-10-2 Materials and Fabrication

The structural steel shall conform to proper American Society for Testing and Materials (ASTM) standards. The minimum thickness of all submerged metal shall not be less than 6.4 mm ($\frac{1}{4}$ in.) and of all-above-water metal 4.8 mm ($\frac{3}{16}$ in.). All iron casting shall also conform to proper ASTM standards. Design and construction shall conform to all American Institute of Steel Construction (AISC) standards for structural steel buildings.

12-10-3 Collectors

Two longitudinal collectors and one cross-collector shall be provided in each basin.

Longitudinal Collectors. The longitudinal collectors shall consist of flights 7.6 × 20.3 cm (3 × 8 in.) nominal size and select hard redwood scrapers spaced approximately 3.05 m (10 ft) apart on two strands of chain. Scrapers shall be equipped with wearing shoes to run on tee rails that will flush with tank bottom and on angle tracks for return run where required. The collector chain shall run over four sets of sprocket wheels at a speed of 0.61 m (2 ft) per min to clean the sludge from the entire tank bottom and skim the water surface in the basin.

Cross-Collectors. The cross-collector shall have flights 5.1 × 15.2 cm (2 × 6 in.) nominal size and select hard redwood, spaced 1.5 m (5 ft) apart, mounted on two strands of chain. These chains shall run over three sets of sprocket wheels at a speed double that of the longitudinal collectors.

12-10-4 Drive Unit

The drive assembly shall consist of a gear motor, gear reducer, drive base, shear pin coupling, overload alarm device, and drive sprocket and chain.

The motor for each drive shall be totally enclosed and shall have a sealed conduit

[p]*Note:* All side water depths (SWD) are at average design flow when both basins are in operation.

box. The motor shall be for operation on 460 V, 3-phase, 60 Hz. It shall be industrial duty and shall have stainless steel hardware, a corrosion-resistant fan, epoxy paint, and cast iron end shields. All gears in the mechanism shall be of a heavy-duty type. All reduction gears shall be of the oil bath type, with an external oil level indicator. A shear pin for protecting the mechanism shall be provided at the gear motor output shaft and shall be set at the torque specified. A torque indicator scale shall be provided and shall be calibrated from 0 to the maximum stall torque of the individual mechanisms.

Collector Chains. The collector chain shall have a specified ultimate strength. The links shall be made from a suitable cast iron steel, which shall have a specified tensile strength and hardness.

Sprockets, Shafts, and Bearings. The sprockets for the drive and collector chains shall be semisteel, cast in a chill, and have a specified hardness. Sprocket teeth shall be accurately ground to fit the chain and shall have not less than the specified number of teeth. The driving sprockets shall be keyed firmly to the headshaft. All shafting shall be solid, cold finished steel that is straight and true and shall be held in alignment with collars and screws. The shafting shall contain keyways with fitted keys where necessary and shall be of sufficient size to transmit the power required. All shafting shall extend across the full width of the tank and shall turn in the bearings mounted on the tank walls. All bearings shall be bolted directly to the concrete wall in a manner that permits easy adjustment. All underwater bearings shall be babbitted and of the water-lubricated, ball-and-socket, self-aligning type, especially designed to prevent accumulation of settled solids on its surface.

12-10-5 Sludge Pump

The sludge pump shall be self-priming, centrifugal, and nonclogging. It shall be suitable for handling the maximum solids concentration anticipated in the primary sedimentation basin. The specified pumping rate is approximately 0.56 m^3/min (150 gpm) at an elevation difference from the sedimentation basin to the sludge-blending tank. Each basin shall have an individual pump assembly capable of servicing both basins in case one pump is out of service. The sludge line shall be sized to handle maximum pump capacity. The pump operation shall be programmed to provide operational flexibility.

12-10-6 Effluent Weir

The manufacturer shall provide an overflow weir box to be located on the outlet end of the basin as required. The weir box shall have 90° V-notch weirs 20 cm center-to-center as detailed. The effluent box shall have vertical adjustment for leveling. The weir box structure shall also provide baffle as shown in drawings to prevent escape of scum into the effluent.

12-10-7 Skimmer

Each sedimentation basin shall be equipped with a hand-operated scum trough for removal of floating oil, grease, and scum from the surface of the tank. The hand-operated scum trough shall have rotating steel pipes open at the top and running across the entire width of the tank. The skimming troughs shall remove the scum from the surface and deposit it into the scum-dewatering trap. The floating scum shall flow over an adjustable weir into the scum pit and be piped to the digester. The excess water shall return to the primary effluent.

12-10-8 Painting

All nonsubmerged ferrous materials shall be brushed cleaned, and submerged ferrous material shall be sandblasted to white metal. Cleaned surfaces shall be primed with approved epoxy primer and have a finished coat of approved paint or epoxy. All field welds shall be touched up with compatible paints.

12-11 PROBLEMS AND DISCUSSION TOPICS

12-1 A batch settling test was performed in a laboratory settling column. The suspension was thoroughly mixed and placed in the column to a desired depth. At 10-min intervals samples were withdrawn from different ports. The suspended solids concentration was determined for each sample. A test duration of 90 min yielded sufficient data for design. A grid was plotted showing percent TSS removal at each port at different time intervals. These results are given in Table 12-8.

Lines of equal percentage removal (isoremoval) were drawn. These lines are drawn similar to contour lines and are shown in Figure 12-21. Determine the theoretical overflow rate and detention time for 80 percent TSS removal. Using the factors of safety of 0.7 and 1.6 obtain the design values of overflow rate and detention time for the sedimentation basin.

TABLE 12-8 Reduced Batch Settling Test Results of Percent TSS Removal in Samples Withdrawn from Various Ports at Different Time Intervals

		Time (min)								
Port No.	**Depth (m)**	**10**	**20**	**30**	**40**	**50**	**60**	**70**	**80**	**90**
1	0.5	33	62	74	90					
2	1.0	21	41	65	71	80	89	90		
3	1.5	16	36	59	67	78	81	86	91	
4	2.0	17	33	56	64	71	78	86	88	91
5	2.5	14	32	54	64	66	76	82	85	88
6	3.0	14	30	52	63	62	75	81	83	85
7	3.5									

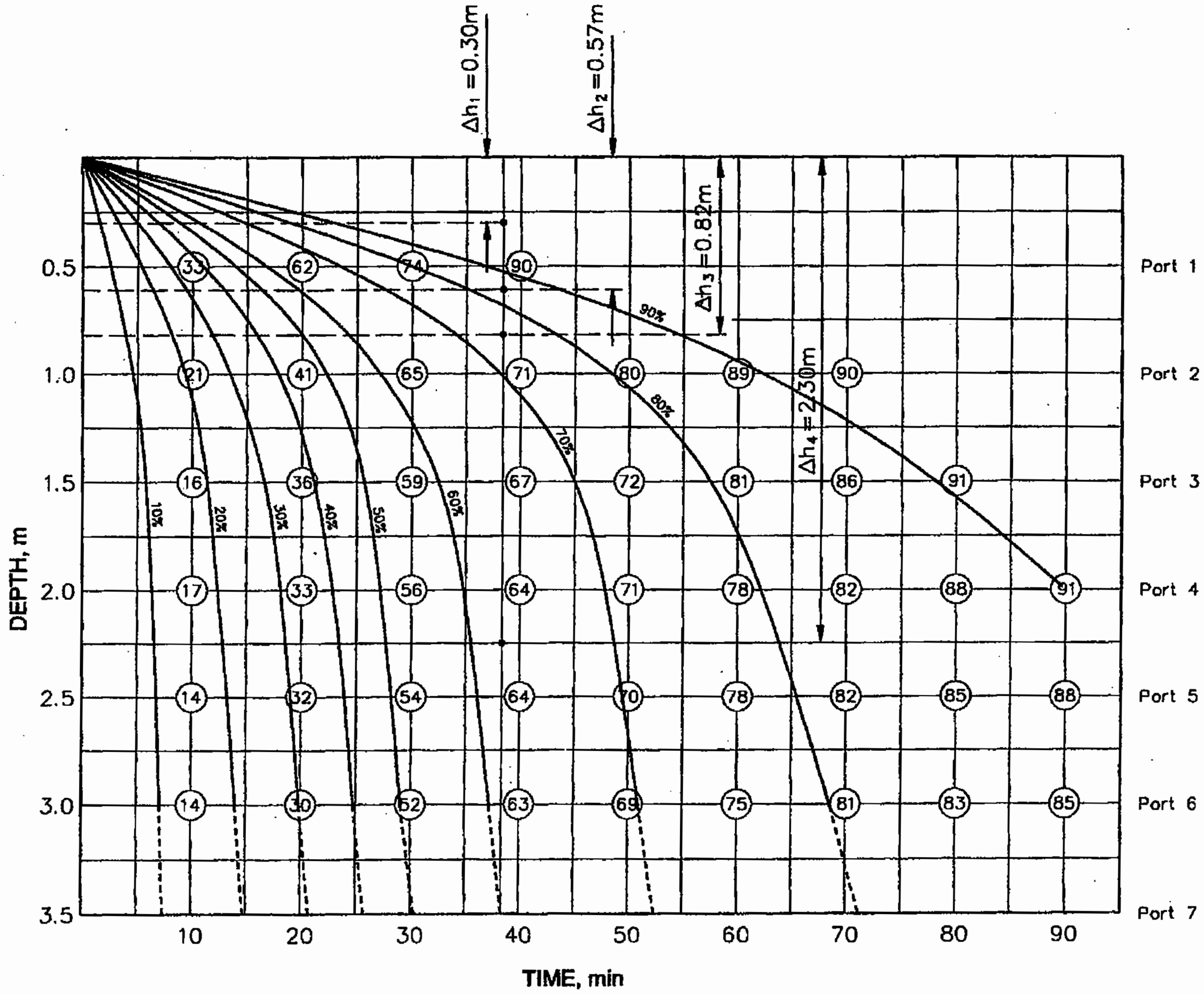


Figure 12-21 Plotting and Reducing Batch Settling Test Data.

12-2 A 30-m-diameter sedimentation basin has an average water depth of 3.0 m. It is treating 0.3 m^3/s wastewater flow. Compute overflow rate and detention time.

12-3 A sedimentation basin has an overflow rate of 80 $m^3/m^2 \cdot d$. What fraction of the particles that have a velocity of 0.02 m/min will be removed in this tank?

12-4 Design a circular clarifier for the Design Example. The design data are given in Sec. 12-8-1. The effluent launder is 0.5 m wide and is installed around the circumference of the basin, 1 m away from the concrete wall. The weir notches are provided on both sides of the launder. The effluent box is 1 m × 1 m, and the depth of flow in the effluent box at peak design flow is 1 m. The invert of the effluent launder is 0.46 m above the invert of the effluent box.

12-5 Develop the hydraulic profile through the primary sedimentation basin in the Design Example at average design flow when both basins are in operation.

12-6 A primary sedimentation basin is designed for an average flow of 0.3 m^3/s. The TSS concentration in the influent is 240 mg/L. The average solids removal efficiency of the basin is 60 percent. The sludge has an average solids concentration of 4 percent and a specific gravity of 1.025. Calculate (a) the quantity and volume of sludge produced, (b) the effluent flow rate, and (c) the pump cycle time if the pumping rate is 570 L/min.

12-7 A primary sedimentation basin is designed for an overflow rate of 30 $m^3/m^2 \cdot d$. Calculate different liquid depths if the basin is designed for the following detention times: 1.0, 1.5, 2.0, 2.5, and 3.0 hours. Assume a flow rate of 0.5 m^3/s. Prepare a curve between detention times and overflow rates at different calculated depths.

12-8 A primary sedimentation facility was designed to treat an average flow of 0.6 m^3/s. The design overflow rate and detention time are 45 $m^3/m^2{\cdot}d$ and 2.5 h, respectively. The length to width ratio of the rectangular basin is 4.3:1. Calculate the dimensions of the basin if two, three, or four basins are provided. Also compute the weir loading rate in each case if one weir trough is provided along the width in each basin as shown in Figure 12-11(b-i). The weir trough has weirs on both sides, and the outlet channel is 1 m wide.

12-9 What are the major differences in three types of clarifiers: horizontal flow, solids contact, and inclined surface? Write the advantages and disadvantages and major applications of each type.

12-10 Four primary sedimentation basins are designed for total average flow of 1.2 m^3/s. The TSS concentration in the primary treated effluent is 150 mg/L, and TSS removal is 63 percent. The sludge has an average solids concentration of 6 percent and specific gravity of 1.045. What is the capacity of the sludge pump in m^3/min per basin if the pump is used on a 15-min pumping cycle per h?

12-11 70,000 kg/d wet sludge (liquid and solids) are produced from a primary sedimentation basin. The sludge pump has a capacity of 0.2 m^3/min. If the sludge is pumped 50 times daily, how would you operate the pump? Assume that the specific gravity of wet sludge is 1.02.

12-12 Study the computational scheme of determining the water surface profile in a flume using Eqs. (11-12)–(11-16). A solution procedure is given in Chapter 11. Perform similar steps to determine the depth of water in the effluent launder of the primary sedimentation facility given in the Design Example. Use a uniform increment of 1.0 m. Compare your result with that given in Sec. 12-8-2, Step C, 7.

12-13 Study the effluent structure of the sedimentation facility given in Figure 12-2(b). How does this differ from that given in Figure 12-18? Prepare a computational scheme similar to that given in Sec. 11-10-2, Step E, 6 for designing the effluent trough and central channel in Figure 12-2(b) if the control conditions at the exit point of the central channel are known.

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13

Biological Waste Treatment

13-1 INTRODUCTION

The major goal of primary treatment is to remove those pollutants that can settle or float. The purpose of secondary treatment is to remove the soluble and colloidal organics that escape the primary treatment and to provide further removal of suspended solids. These removals are typically achieved by using biological treatment processes. The biological treatment processes provide similar biological reactions that would occur in the receiving waters if the aquatic environment had adequate capacity to assimilate the wastes. Although conventional secondary treatment may remove more than 85 percent of the BOD_5 and suspended solids, it does not remove significant amounts of nitrogen, phosphorus, heavy metals, nonbiodegradable organics, bacteria, and viruses. These pollutants may require further removal where receiving waters are especially sensitive. Recent advancements in biological waste treatment technology are capable of providing enhanced nutrient removal by process modification. These are called biological nutrient removal (BNR) processes.

Secondary levels of treatment can also be achieved by physical-chemical or natural systems. These are add-on treatment systems to the existing biological treatment plants to upgrade the effluent quality or to achieve advanced wastewater treatment. These treatment systems are presented in Chapter 24. The purpose of this chapter is to present the theory and design of biological waste treatment processes. The major treatment processes discussed in this chapter include aerobic, anaerobic, and BNR. The BNR processes use a combination of anaerobic, anoxic, and aerobic reactors. Since they achieve enhanced nitrogen and phosphorus removal in addition to BOD_5 and total suspended solids (TSS) removal, they have gained a great deal of interest in recent years for designing medium and large wastewater treatment plants. Therefore, a step-by-step procedure for designing such a facility is presented in the Design Example.

13-2 FUNDAMENTALS OF BIOLOGICAL WASTE TREATMENT

13-2-1 Growth and Substrate Utilization

The basic objective of biological waste treatment is to feed the substrate to a mixed microbial culture to remove it from the solution. Substrate is the term used to denote organic matter, nutrients, and other substances that may be present in wastewater. Thus, microorganisms are used to consume organics, nitrify ammonia, denitrify nitrate, and release and uptake phosphorus. Some of the recent advancements in wastewater treatment technology are now used to design and build wastewater treatment facilities that achieve effectively and economically carbonaceous BOD and substantial amounts of nitrogen and phosphorus removal. In this section the basic concepts of microbiology, growth phases, and substrate utilization requirements are covered.

Microbiology. The stabilization of wastes is accomplished by a variety of microorganisms. In a mixed culture the number of species and their populations depend upon the characteristics of wastewater and the environmental conditions. Classification of microorganisms and basic concepts are covered in Sec. 3-4-4.[1-4]

Basic Requirements. Biological waste treatment involves bringing the active microbial growth into contact with wastewater so that they can consume the impurities as food. A great variety of microorganisms come into play that include bacteria, protozoa, rotifers, nematodes, fungi, algae, and so forth. A general overview of important microorganisms in biological waste treatment is given in Chapter 3. These organisms, in the presence of oxygen, convert the biodegradable organics into carbon dioxide, water, more cell material, and other inert products. The basic ingredients needed for secondary biological treatment are the availability of (1) mixed populations of active microorganisms, (2) good contact between the microorganism and waste material, (3) availability of oxygen, (4) availability of nutrients, and (5) maintenance of other favorable environmental conditions, such as temperature, pH, sufficient contact time, and so on.

Growth Phases. The bacteria reproduce by *binary fission.* The time required for each fission (division of original cell into two new cells) is called generation time. This can vary from less than 20 min to several days. The growth of microorganisms in a batch reactor may be expressed in terms of time versus viable bacterial number or viable biomass. Initially, the number or mass of microorganisms is small. As soon as food is brought into contact with microorganisms, growth starts. Initially, the growth is slow because the microorganisms are becoming adjusted to their new environment. This is called the *lag growth* phase. The lag growth phase for bacterial mass is not as long as the corresponding lag growth phase for bacterial number because mass begins to increase before cell division takes place. After the initial phase there is a rapid growth in number or mass. This phase is called the *log growth* phase and is typical of microorganisms when there is an excess of food. In this phase, the rate of growth of the cells and their subsequent division is limited by the ability of the microorganisms to process the substrate. During the log growth phase, the microorganisms grow at their maximum rate and consequently remove organic matter from solution at a maximum rate. As the food concentration becomes limited, a *declining growth rate* develops. A further decrease in food concentration inhibits microbial metabolism that results in a *stationary phase,* followed by *death phase.* In this phase the bacterial death rate exceeds the production of new cells. During this phase the microorganisms are forced to metabolize their own protoplasm

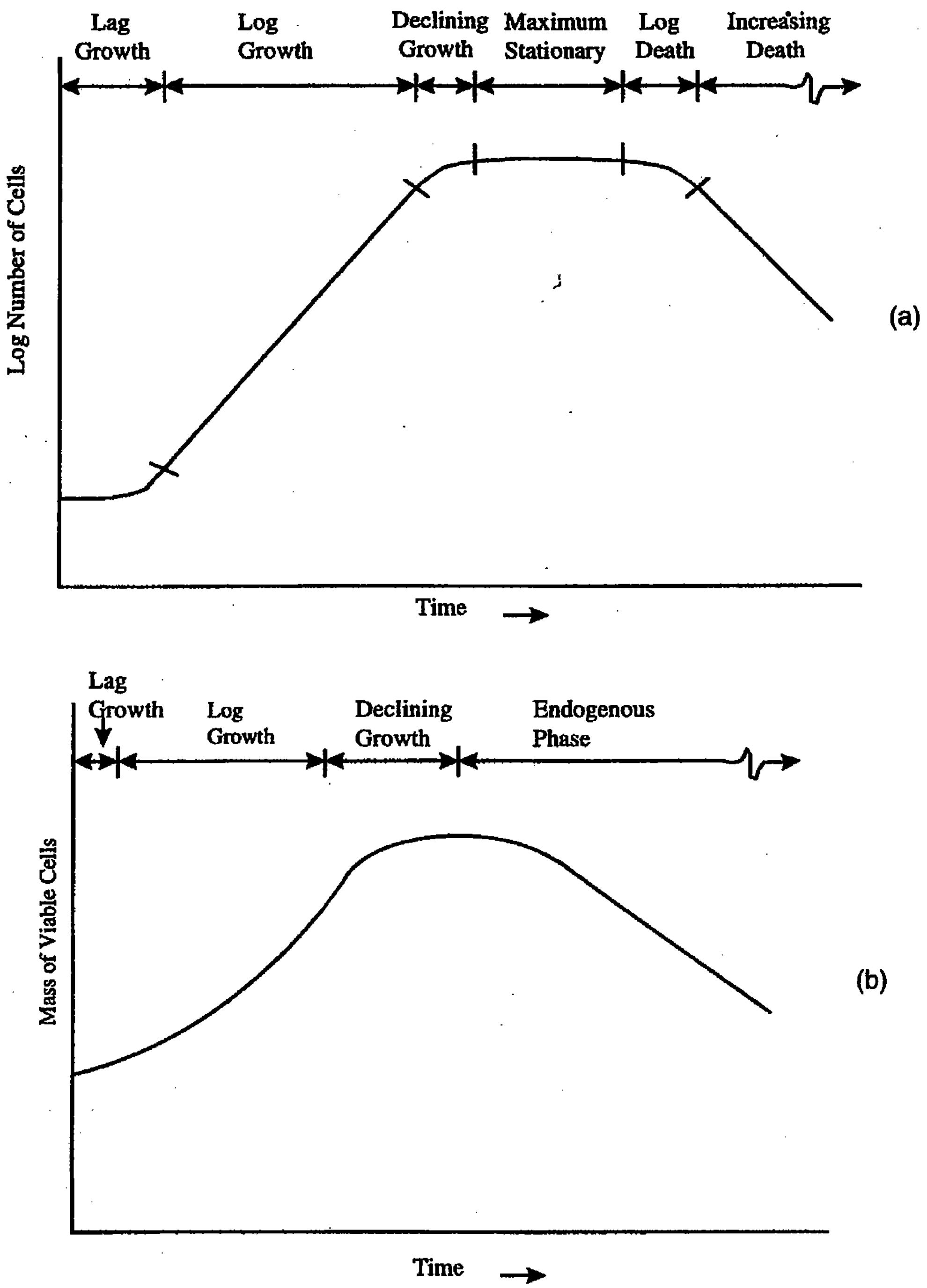

Figure 13-1 Typical Bacterial Growth Curves: (a) viable numbers and (b) mass of viable cells.

without replacement, resulting in a decrease in the biological mass. This is also known as the *endogenous phase*. The typical biological curves in terms of numbers and cell mass are shown in Figure 13-1.[4-8]

Role of Enzymes. Microorganisms grow and obtain their energy from substrates utilizing very complex and intricate biochemical reactions. Several enzymes are involved in a series of reactions forming a sequence of enzyme–substrate complexes, which are then converted to a product and the original enzyme. The enzymes are proteins that act as catalysts. Enzymes are also specific to each substrate and have a high degree of efficiency in converting the substrate to the end products. The enzymes are *extracellular* and *intracellular.* Extracellular enzymes convert the substrate to a form that can diffuse into the cell. The intracellular enzymes bring about oxidation, synthesis, and energy reactions within the cell. The enzyme activity, however, is substantially affected by pH, temperature, and substrate concentration.

When substrates are oxidized within the cell, energy is released. The energy thus released is stored by the phosphate enzyme system. Inorganic phosphate is added to adenosine diphosphate (ADP) to form adenosine triphosphate (ATP). In this way the energy is stored in the ATP rather than lost as heat. As the microorganisms require the energy, the ATP is reduced back to ADP with a transfer of energy to the chemical reactions needing it or for growth and cell activity. The biochemistry of the metabolic pathways is very complex and beyond the scope of this book. Readers may consult Refs. 1–5 for more on this subject.

Nutritional and Environmental Requirements. For growth and survival of microorganisms, there must be (1) a source of energy, (2) carbon for the synthesis of new cells, and (3) inorganic elements or nutrients. The energy needed for cell synthesis may be supplied by light or by a chemical oxidation reaction. The most common sources of cell carbon are organic matter and carbon dioxide. The classification of microorganisms by sources of energy and cell carbon is presented in Chapter 3 (Sec. 3-4-4). The principal inorganic nutrients needed by microorganisms are N, P, S, K, Mg, Ca, Fe, Na, and Cl. Minor nutrients of importance include Zn, Mn, Mo, Co, Cu, Ni, V, and W. In addition to inorganic nutrients, organic nutrients or growth factors are also needed by some microorganisms. The major growth factors are (1) amino acids, (2) purines and pyrimidines, and (3) vitamins.

The principal environmental factors affecting microbial activity are (1) temperature, (2) chemicals (salt content), (3) presence or absence of oxygen, and (4) mixing or contact between food and microorganisms. The temperature also affects the microbial activity. At an average, a temperature increase of 10°C approximately doubles the activity. The chemicals affect the pH, buffering capacity, oxidation and reduction, and osmotic pressure of the aqueous solutions. Microorganisms can function within a narrow range of these factors that are easily influenced by chemicals. Heavy metal ions are toxic in relatively low concentrations. The toxicity in general increases with an increase in the atomic weight. The heavy metals that may be encountered in wastewaters are arsenic, barium, cadmium, chromium, copper, lead, mercury, nickel, silver, and zinc. Certain organic chemicals are also quite toxic to microorganisms.

13-2-2 Biological Kinetic Equations

In the past, the designs of biological wastewater treatment systems were based on empirical parameters developed by experience. Many of these empirical parameters included organic loading, hydraulic loading, reaction period, and others. Today, however, the design utilizes empirical, as well as rational parameters based on biological kinetic equations. These equations express growth of biological solids, substrate utilization rates in terms of biological kinetic coefficients, food-to-microorganism ratio, the mean cell residence time, and more. Using these equations, design parameters such as reactor volume, biomass growth, substrate utilization, and effluent quality can be calculated. The biological kinetic notations and equations are adapted from Ref. 4. Readers should refer to this source for a more detailed discussion on this subject.

Cell Growth Rate in Excess Substrate. The growth rate of bacterial cells when excess substrate is present is expressed in Eq. (13-1)

$$r_g = \mu X \tag{13-1}$$

where

r_g = rate of microbial growth, g/m^3·d
μ = specific growth rate, d^{-1}
X = concentration of microorganisms or total volatile suspended solids (TVSS), g/m^3

Cell Growth Rate in Limited Substrate. Under substrate- or nutrient-limited situations, the growth of microorganisms is limited. The growth is defined by an expression proposed by the Monod Eq. (13-2):

$$\mu = \mu_{\max} \frac{S}{K_s + S} \tag{13-2}$$

where

$\mu_{\max}$ = maximum specific growth rate, d^{-1}
K_s = half-velocity constant or the substrate concentration at one-half the maximum specific growth rate, g/m^3
S = concentration of growth-limiting substrate in solution, g/m^3

Substituting the value of μ from Eq. (13-2) into Eq. (13-1), the growth of microorganisms under a limited substrate environment is obtained. This is expressed by Eq. (13-3):

$$r_g = \mu_{\max} \frac{XS}{K_s + S} \tag{13-3}$$

Substrate Utilization Rate. In a reactor, as microorganisms grow, a portion of substrate is converted into new cells, and a portion of substrate is oxidized for energy. The relationship between microorganism growth and substrate utilization is expressed by Eq. (13-4):

$$Y = -\frac{r_g}{r_{su}} \tag{13-4}$$

where

Y = maximum cell yield coefficient, g of cell mass/g of substrate utilized over a finite period of growth phase
r_{su} = rate of substrate utilization, g/m^3·d

Eqs. (13-3) and (13-4) are combined to express the substrate utilization rate. This relationship is expressed by Eq. (13-5):

$$r_{su} = -\frac{\mu_{max} XS}{Y(K_s + S)} \tag{13-5}$$

In Eq. (13-5), the term μ_{max}/Y is substituted from Eq. (13-6) to yield a more usable expression [Eq. (13-7)]:

$$k = \frac{\mu_{max}}{Y} \tag{13-6}$$

$$r_{su} = -\frac{kXS}{(K_s + S)} \tag{13-7}$$

where

k = maximum rate of substrate utilization per unit mass of microorganisms, d^{-1}

Endogenous Metabolism. In a mixed culture biomass growth is also accompanied by death, predation, cell lysing, and decay. It is assumed that the decrease in cell mass is proportional to the concentration of biomass present. This decrease is called endogenous decay. The net growth of biomass and net specific growth rates are therefore expressed by Eqs. (13-8)–(13-10).

$$r_g' = \mu_{max} \frac{XS}{K_s + S} - k_d X \tag{13-8}$$

$$r_g' = -Yr_{su} - k_d X \tag{13-9}$$

$$\mu' = \mu_{max} \frac{S}{K_s + S} - k_d \tag{13-10}$$

where

r_g' = net growth rate of biomass, g/m^3·d
k_d = endogenous decay coefficient, d^{-1}
μ' = net specific growth rate, d^{-1}

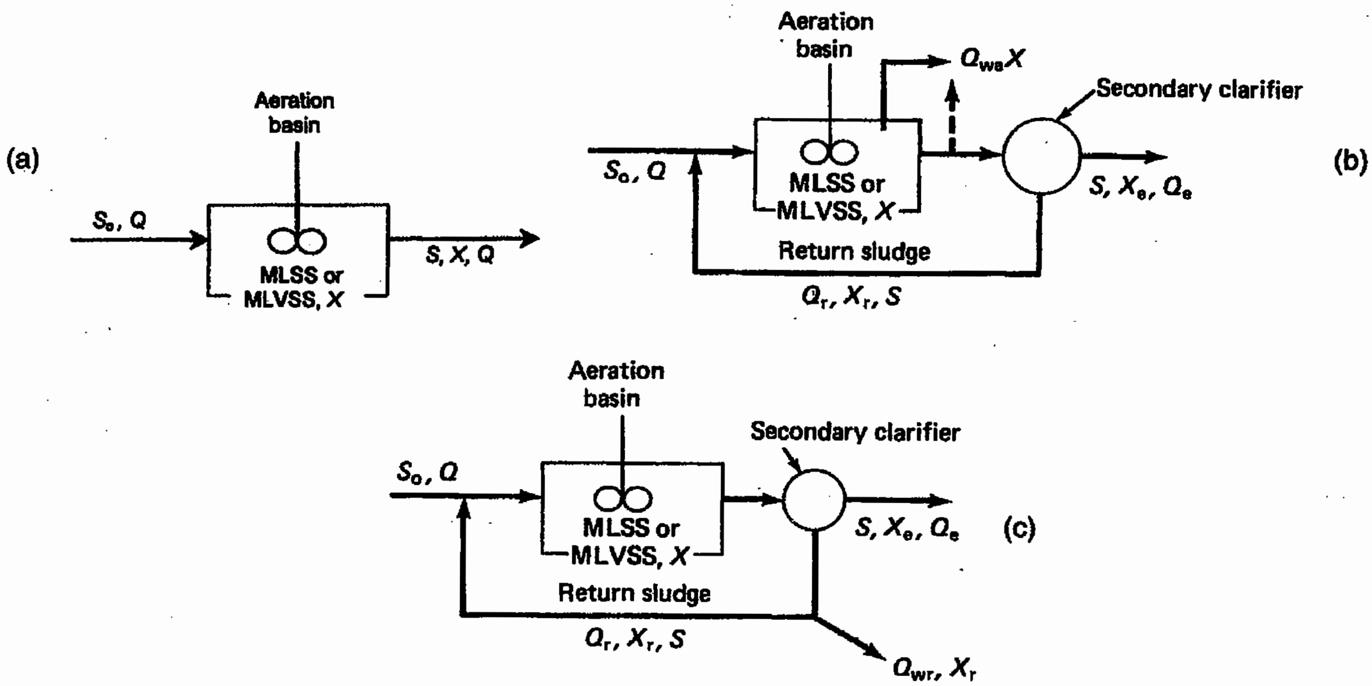

Figure 13-2 Complete-Mix Biological Reactor: (a) no solids return, (b) solids return and wasting from aeration basin, and (c) solids return and wasting from return sludge line.

The endogenous respiration has an effect on net cell yield. Therefore, the observed yield (Y_{obs}) is expressed by Eq. (13-11):

$$Y_{obs} = -\frac{r_g'}{r_{su}} \tag{13-11}$$

Process Design Relationships. In a continuous substrate feed system with no biosolids input [Figure 13-2(a)], the steady-state condition is reached when concentration of biosolids in the reactor is constant. Under this condition, the loss of biosolids from the system is equal to the net growth, and θ is also equal to solids retention time. This relationship for no solids return system is expressed by Eqs. (13-12)–(13-14):

$$QX = V(-Yr_{su} - k_d X) \tag{13-12}$$

$$\frac{1}{\theta} = -Y\frac{r_{su}}{X} - k_d \tag{13-13}$$

$$\frac{1}{\theta} = YU - k_d \tag{13-14}$$

where

Q = flow to the reactor, m^3/d
V = Volume of the reactor, m^3

θ = hydraulic retention time (V/Q), d
U = specific substrate utilization rate ($-r_{su}/X$), d^{-1}

The specific substrate utilization rate U is also expressed in Eqs. (13-15)–(13-17).

$$U = \frac{Q(S_o - S)}{VX} = \frac{(S_o - S)}{\theta X} \tag{13-15}$$

$$U = \frac{kS}{K + S} \tag{13-16}$$

or

$$\frac{1}{U} = \frac{K_s}{kS} + \frac{1}{k} \tag{13-17}$$

where

S_o = substrate concentration in the influent (BOD_5 or COD), g/m^3
S = substrate concentration in the effluent (BOD_5 or COD), g/m^3

Continuous Flow Reactor with Solids Recycle. In a biological reactor, if solids are recycled [Figure 13-2(b) and (c)], the solids retention time (SRT), or θ_c in the system is obtained from the mass of solids maintained in the reactor divided by mass of solids produced or removed per day from the system. Many related relationships are also expressed by Eqs. (13-18)–(13-22).

$$\theta_c = \frac{VX}{Y\theta(S_o - S)} = \frac{VX}{Q_{wa}X + Q_eX_e} = \frac{VX}{Q_{wr}X_r + Q_eX_e} = \frac{\theta X}{P_x} \tag{13-18}$$

$$\frac{1}{\theta_c} = YU - k_d \tag{13-19}$$

$$Y_{\text{obs}} = \frac{YU - k_d}{U} = \frac{Y}{1 + \theta_c k_d} \tag{13-20}$$

$$P_x = Y_{\text{obs}}\, Q(S_o - S) \tag{13-21}$$

$$\frac{P_x}{Q} = p_x = Y_{\text{obs}}(S_o - S) \tag{13-22}$$

where

θ_c = mean cell residence time or solids retention time (SRT), d
Q_{wa} = sludge wasted from the reactor, m^3/d
Q_e = effluent discharge, m^3/d
Q_{wr} = sludge wasted from the sludge return line, m^3/d

X_r = concentration of MLVSS in the waste sludge line, g/m^3
X_e = concentration of VSS in the effluent, g/m^3
P_x = solids growth (VSS), g/d
p_x = sludge growth rate per unit flow, g/m^3

There is a minimum mean cell residence time ($\theta_c^{\min}$) below which substrate utilization and cell growth do not occur. The value of $\theta_c^{\min}$ is determined from Eqs. (13-6), (13-16), and (13-19). In wastewater treatment practice normally, S_o is much greater than K_s. As a result, $(K_s + S_o) \approx S_o$, and $\theta_c^{\min}$ can be approximately computed from Eq. (13-23):

$$\theta_c^{\min} \approx \frac{1}{\mu_{\max} - k_d} \tag{13-23}$$

Many other useful process design equations can also be developed from the preceding discussion and are expressed by Eqs. (13-24)–(13-27). Readers should refer to Refs. 4–7 for in-depth coverage on this topic.

$$F/M = \frac{QS_o}{VX} = \frac{1}{\theta}\frac{S_o}{X} \tag{13-24}$$

$$U = \frac{(F/M)\,E}{100} \text{ and } E = \frac{(S_o - S)}{S_o} \times 100 \tag{13-25}$$

$$X = \frac{\theta_c Y(S_o - S)}{\theta\,(1 + \theta_c k_d)} = \frac{\theta_c Y_{\text{obs}}\,(S_o - S)}{\theta} \tag{13-26}$$

$$S = \frac{K_s\,(1 + \theta_c k_d)}{\theta_c\,(Yk - k_d) - 1} \tag{13-27}$$

where

F/M = food-to-microorganism ratio, d^{-1}
E = process efficiency, percent

Determination of Kinetic Coefficients. The values of kinetic coefficients Y, k, k_d, and K_S greatly influence the design of a biological treatment process. These values depend on the characteristics of the wastewater and therefore must be determined for every waste stream (especially if it contains industrial wastes) from bench or pilot plant studies. The procedure is to operate an experimental continuous flow reactor at different mixed liquor volatile suspended solids (MLVSS) concentrations. One such reactor is shown in Figure 2-4(a). Using the data collected under the steady-state condition at each concentration of MLVSS, the mean values of Q, Q_{wa}, S_o, S, and X_e are determined. From these results $(Q_{wa}X + Q_eX_e)$ is obtained, and the values of U and θ_c are calculated from Eqs. (13-15) and (13-18), respectively.[4,8-10]

A plot of $1/\theta_c$ versus U gives a straight line [Eq. (13-19)]. The slope of the straight line is the value of Y and the intercept is k_d. Similarly, a plot of $1/U$ versus $1/S$ [Eq.

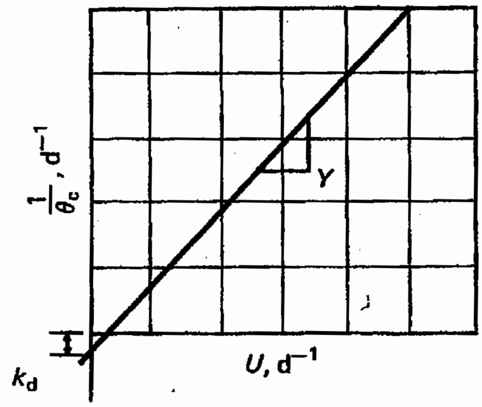

Figure 13-3 Procedure to Determine Y *and* k_d.

(13-17)] gives a straight line. The slope of the straight line is K_s/k and the intercept is $1/k$. These plots are shown in Figures 13-3 and 13-4. The range and typical values of these kinetic coefficients for municipal wastewater are summarized in Table 13-1.[4-7]

13-2-3 Types of Biological Treatment Processes

The common biological waste treatment processes fall into three major groups. These are aerobic, anaerobic, and biological nutrient removal processes. Further division is made on the basis of suspended growth, attached growth, and a combination of these. Brief descriptions of these major groups of the treatment processes and their applications are presented in Table 13-2. Discussion on these processes is divided into the following categories: (a) aerobic suspended growth; (b) aerobic attached growth and combinations of these; (c) anaerobic suspended growth; (d) anaerobic attached growth and combinations of these; and (e) biological nutrient removal systems that include combinations of anaerobic, anoxic,[a] and aerobic processes. Each of these processes is presented below.

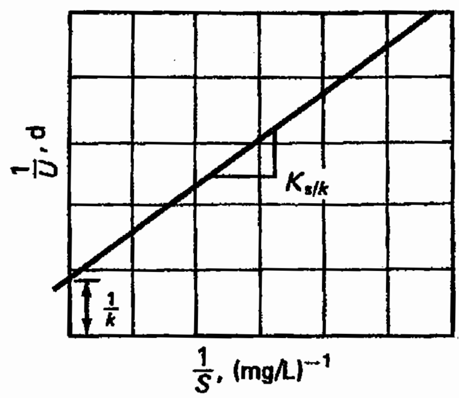

Figure 13-4 Procedure to Determine k *and* K_s.

[a]Anoxic condition occurs in the presence of nitrate nitrogen, and the absence of oxygen. Nitrate nitrogen is biologically converted into nitrogen gas. This process is also known as anoxic denitrification.

TABLE 13-1 Typical Values of Kinetic Coefficients for Activated Sludge Process

Coefficient	Basis	Values: Range	Values: Typical
k	d^{-1}	2–8	4
k_d	d^{-1}	0.03–0.07	0.05
K_s	mg/L, BOD_5	40–120	80
	mg/L, COD	20–80	40
Y	VSS/BOD_5	0.3–0.7	0.5
	VSS/COD	0.2–0.5	0.4

13-3 AEROBIC SUSPENDED GROWTH REACTORS

Suspended growth, aerobic treatment systems are those in which the microorganisms remain in suspension, and aerobic condition prevails. Common suspended growth processes used for secondary treatment are (1) activated sludge and modifications and (2) other applications. Following is a general discussion and design considerations on many aerobic suspended growth system.

TABLE 13-2 Major Group and Subgroups of Biological Treatment Processes and Their Applications

Major Group	Description	Major Subgroup	Applications
Aerobic processes	Treatment occurs in presence of oxygen.	Suspended growth, attached growth, and combined suspended and attached growth	Removal of carbonaceous BOD and nitrification
Anaerobic processes	Treatment occurs in absence of oxygen and nitrate.	Suspended growth and attached growth	Removal of carbonaceous BOD
Biological nutrient removal	Treatment occurs in a single or multiple reactors. Anaerobic, anoxic, and aerobic conditions are created. Denitrification, phosphorus release, and uptake occur in presence of suitable carbon source.	Suspended growth and combination of suspended and attached growth	Removal of carbonaceous BOD, nitrification, denitrification, and phosphorus release and uptake

13-3-1 Activated Sludge and Modifications

In the activated sludge process, microorganisms (MO) are mixed thoroughly with the organics so that they can grow and stabilize the organics. As the microorganisms are mixed with incoming wastewater and oxygen is supplied by aeration, the individual organisms clump together (flocculate) to form an active mass of microbial floc called *activated sludge*. The mixture of the activated sludge and wastewater in the aeration basin is called *mixed liquor*. The mixed liquor flows from the aeration basin to a secondary clarifier where the activated sludge is settled. A portion of the settled sludge is returned to the aeration basin to maintain the proper food-to-MO ratio to permit rapid breakdown of the organic matter. Because more activated sludge is produced than can be used in the process, some of it is wasted from the aeration basin or from the returned sludge line to the sludge-handling systems for treatment and disposal. Air is introduced into the aeration basin either by diffuser or by mechanical mixers. There are many modifications of the activated sludge process. These modifications differ in mixing and flow pattern in the aeration basin and in the manner in which the microorganisms are mixed with the incoming wastewater. The basic design principles of the activated sludge processes are discussed below.

Type of Reactors. The principal types of biological reactors (aeration basins) are plug-flow, complete-mix, and arbitrary flow. In a plug-flow reactor the particles pass through the tank and are discharged in the same sequence in which they enter. This type of flow is achieved in a long, narrow basin. In a complete-mix reactor, the entering particles are dispersed immediately throughout the entire basin. Complete-mix is achieved in circular or square basins. Arbitrary-flow reactors exhibit partial mixing somewhere between the plug-flow and complete-mix reactors. A more detailed discussion on this subject may be found in Chapter 4.

Process Modifications. The major process modifications of activated sludge processes are (1) conventional, (2) tapered aeration, (3) step aeration, (4) complete-mix, (5) modified aeration, (6)high-rate aeration, (7) extended aeration, (8) single-stage nitrification, (9) separate stage nitrification, (10) deep-shaft reactor, (11) sequencing batch reactor, (12) contact stabilization, (13) Kraus process, and (14) high-purity oxygen system.[4,11-17] These processes differ from each other in the manner in which the influent is applied, microorganisms are utilized, and hardware is assembled. These process modifications and principal design parameters are summarized in Table 13-3. Flow diagrams of different process modifications are illustrated in Figure 13-5.

Aeration System. Two major types of aeration systems are used in the activated sludge process. These are (1) diffused aeration and (2) mechanical aeration. Each of these systems is discussed briefly below.

Diffused Aeration. In diffused aeration, air is supplied through porous diffusers or through air nozzles near the bottom of the tank. Various components of the diffused aeration system include (1) diffuser or air nozzles, (2) pipings, and (3) blower or com-

Text continued on page 399.

TABLE 13-3 *Description and Design Parameters of Various Activated Sludge Process Modifications*

Process Modification	Brief Description	Flow Regime	Sludge Retention Time, θ_c (d)	Food to MO Ratio[a] (d^{-1})	Aerator Loading[b] (kg/m^3d)	MLSS[c] (mg/L)	Aeration Period (h)	Recirculation Ratio Q_r/Q
Conventional	The influent and returned sludge enter the tank at the head end of the basin and are mixed by the aeration system [Figure 13-5(a)].	Plug	5–15	0.2–0.4	0.3–0.6	1500–3000	4–8	0.25–0.5
Tapered aeration	The tapered aeration system is similar to the conventional activated sludge process. The major difference is in arrangement of the diffusers. The diffusers are close together at the influent end where more oxygen is needed. The spacing of diffusers is increased toward the other end of the aeration basin.	Plug	5–15	0.2–0.4	0.3–0.6	1500–3000	4–8	0.25–0.5
Step-feed aeration	The influent is applied at several points in the aeration basin. Generally, the tank is subdivided into three or more parallel channels with around-the-end baffles, and the influent is applied at separate channels or steps. The oxygen demand is uniformly distributed [Figure 13-5(b)].	Plug	5–15	0.2–0.4	0.6–1.0	2000–3500	3–5	0.25–0.75

Continued

TABLE 13-3 Description and Design Parameters of Various Activated Sludge Process Modifications—cont'd

Process Modification	Brief Description	Flow Regime	Sludge Retention Time, θ_c (d)	Food to MO Ratio[a] (d^{-1})	Aerator Loading[b] (kg/m^3d)	MLSS[c] (mg/L)	Aeration Period (h)	Recirculation Ratio Q_r/Q
Complete-mix aeration	The influent and the returned sludge are mixed and applied at several points along the length and width of the basin. The contents are mixed, and the MLSS flows across the tank to the effluent channel. The oxygen demand and organic loading are uniform along the entire length of the basin [Figure 13-4(c)].	Complete mix	5–15	0.2–0.6	0.8–2.0	3000–6000	3–5	0.25–1.00
Modified aeration	This process is used as an intermediate treatment to reduce the organic loading in subsequent process. It is similar to conventional treatment. Shorter aeration period, low MLSS and high *F/M* ratio are utilized.	Plug	0.2–0.5	1.5–5.0	1.2–2.4	200–1000	1.5–3	0.05–0.25

High-rate aeration	The process is similar to a conventional treatment process. High MLSS concentration and high volumetric loading are applied. This way low mean cell residence time and a high *F/M* ratio are achieved. Aeration period is relatively short. Aeration and mixing is achieved by mechanical mixers.	Plug	3–10	0.4–1.5	2.0–15	3000–6000	2–4	0.5–2.0
Extended aeration (oxidation ditch)	The extended aeration process utilizes a large aeration basin where high population of MO is maintained. It is used for small flows from subdivisions, schools, etc. Prefabricated package plants utilize this process extensively. Oxidation ditch is a variation of extended aeration process. It has a channel in shape of a race track. Rotors are used to supply oxygen and maintain circulation [Figure 13-5(d)].	Complete mix or plug	20–30	0.05–0.15	0.1–0.4	3000–6000	18–36	0.5–2.0
Single-stage nitrification	Carbon oxidation and nitrification are carried out into one aeration basin. High mean cell residence time and low *F/M* ratio are utilized. Aeration period is relatively high.	Plug	10–20	0.05–0.2	0.08–0.3	1500–3500	6–15	0.5–1.5

Continued

TABLE 13-3 Description and Design Parameters of Various Activated Sludge Process Modifications—cont'd

Process Modification	Brief Description	Flow Regime	Sludge Retention Time, θ_c (d)	Food to MO Ratio[a] (d^{-1})	Aerator Loading[b] (kg/m^3d)	MLSS[c] (mg/L)	Aeration Period (h)	Recirculation Ratio Q_r/Q
Separate-stage nitrification	Two-stage aeration is applied. In the first stage BOD is reduced. The reactor is similar to high-rate aeration. In the second reactor nitrification is achieved. The process may commonly be an add-on to an existing activated sludge plant to enhance nitrification [Figure 13-5(e)].	Plug	2–5 (8–15)[d]	0.5–1.5 (0.1–0.3)[d]	1.2–2.4 (0.3–0.8)[d]	2000–3000 (2000–4000)	1–3 (2–4)	0.2–0.5 (0.2–0.5)
Deep shaft	It is an aerobic biological sub-surface wastewater treatment process. A vertical shaft about 100–150 m (330–500 ft) deep is drilled and lined with steel shell. A concentric downcomer brings MLSS, which is aerated and passes into the riser section. The shaft replaces the primary clarifier and aeration basin. The solids/liquid separation is achieved in a flotation tank [Figure 13-5(f)].	Plug	2–5	0.5–1.2	0.5–1.5	4000–7000	0.5–2	0.2–0.5

Sequencing batch reactor	This is a fill-and-draw-type reactor that acts as aeration basin and final clarifier. MLSS remains in the reactor. Primary clarification is generally not needed. Some nitrogen and phosphorus removal is also achieved [Figure 13-5(g)].	Complete-mix	Not applicable	0.05–0.30	0.20–0.70	1500–5000[e]	4–9	Not applicable
Contact stabilization	The activated sludge is mixed with influent in the contact tank in which the organics are absorbed by MO. The MLSS is settled in the clarifier. The returned sludge is aerated in the reaeration basin to stabilize the organics. The process requires approximately 50 percent less tank volume [Figure 13-5(h)].	Plug	5–15	0.2–0.6	1.0–1.2	1000–4000[f] 4000–10000[g]	0.5–1.0[f] 3.0–6.0[g]	0.5–1.0
Kraus process	This is a variation of step aeration process. Used commonly for industrial wastes low in nitrogen. In a separate aeration basin the digester supernatant is mixed with the return sludge to provide needed nutrients. This returned sludge is added into the aeration basin at desired locations.	Plug	5–15	0.3–1.0	0.5–1.5	2000–3000	4–8	0.5–1.0

Continued

TABLE 13-3 Description and Design Parameters of Various Activated Sludge Process Modifications—cont'd

Process Modification	Brief Description	Flow Regime	Sludge Retention Time, θ_c (d)	Food to MO Ratio[a] (d^{-1})	Aerator Loading[b] (kg/m^3d)	MLSS[c] (mg/L)	Aeration Period (h)	Recirculation Ratio Q_r/Q
High-purity oxygen	Oxygen is diffused into covered aeration tanks. A portion of gas is wasted from the tank to reduce the concentration of CO_2. The process is suitable for high-strength wastes where space may be limited. Special equipment for generation of oxygen is needed [Figure 13-5(i)].	Complete mix	8–20	0.25–1.0	1.6–3.3	6000–8000	2–5	0.25–0.5

[a]Food-to-microorganism ratio (*F/M*) is kg BOD_5 applied per day per kg of MLVSS in the aeration basin.
[b]Aerator loading is kg of BOD_5 applied per day per cubic meter of aeration capacity.
[c]Generally, the ratio of MLVSS to MLSS is 0.75–0.85.
[d]Values for nitrification tank are based on influent BOD_5 = 50 mg/L.
[e]MLSS varies during fill-and-draw cycles.
[f]Contact tank.
[g]Reaeration or stabilization tank.
Source: Adapted in part from Refs. 4–12.

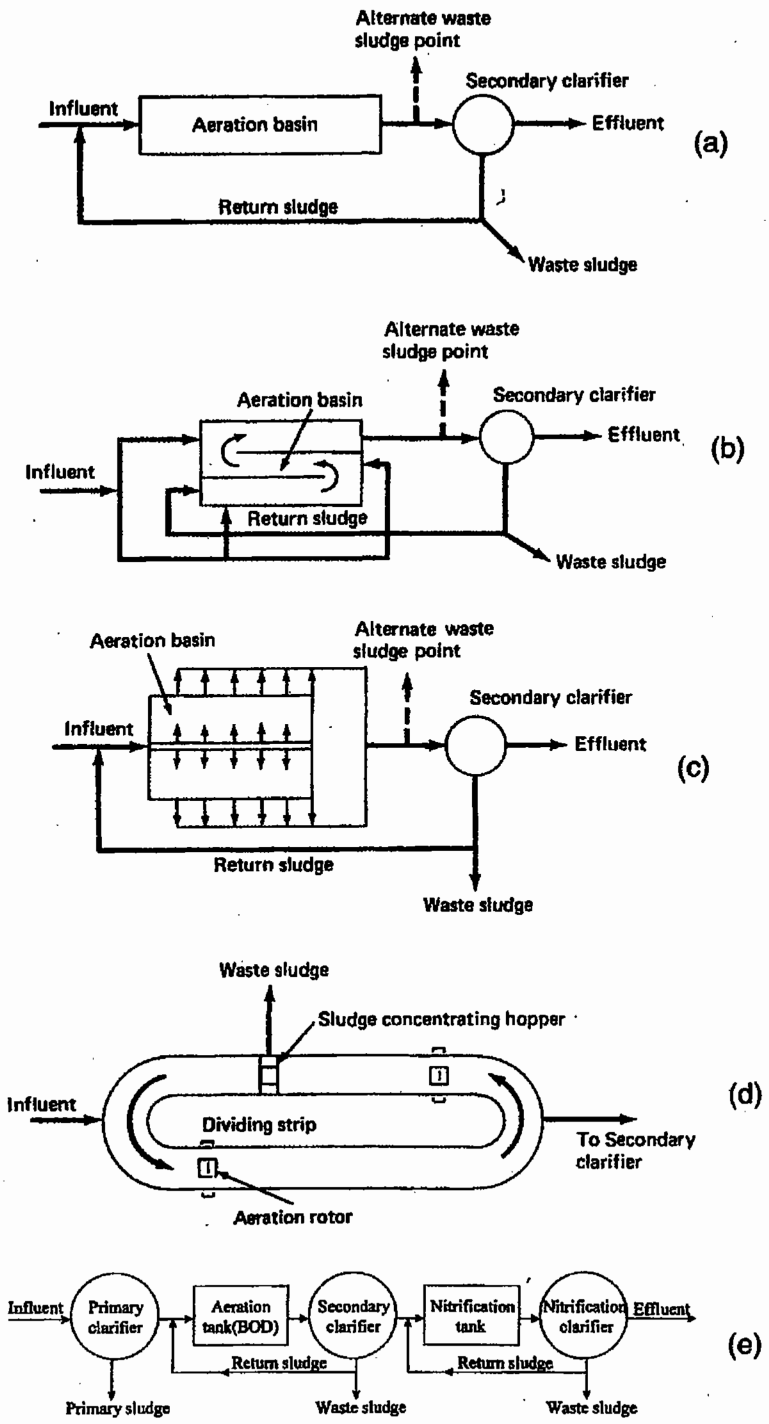

Figure 13-5 Modifications of Activated Sludge Process (from Refs. 4–12): (a) conventional, (b) step aeration, (c) complete mix, (d) oxidation ditch, (e) carbon oxidation and nitrification in separate stage.

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(f)

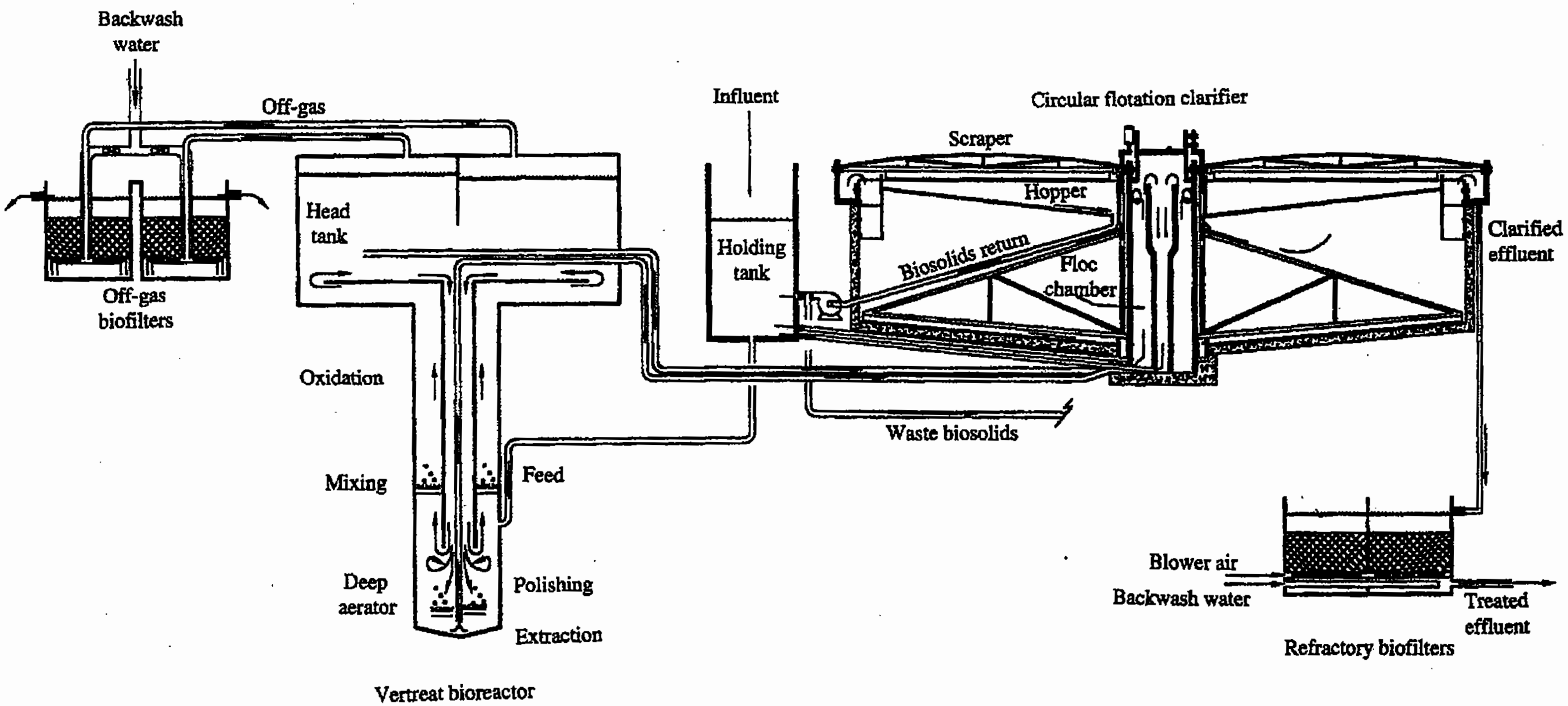

Figure 13-5—cont'd (f) deep-shaft biological treatment system (courtesy Deep Shaft Technology, Inc.).

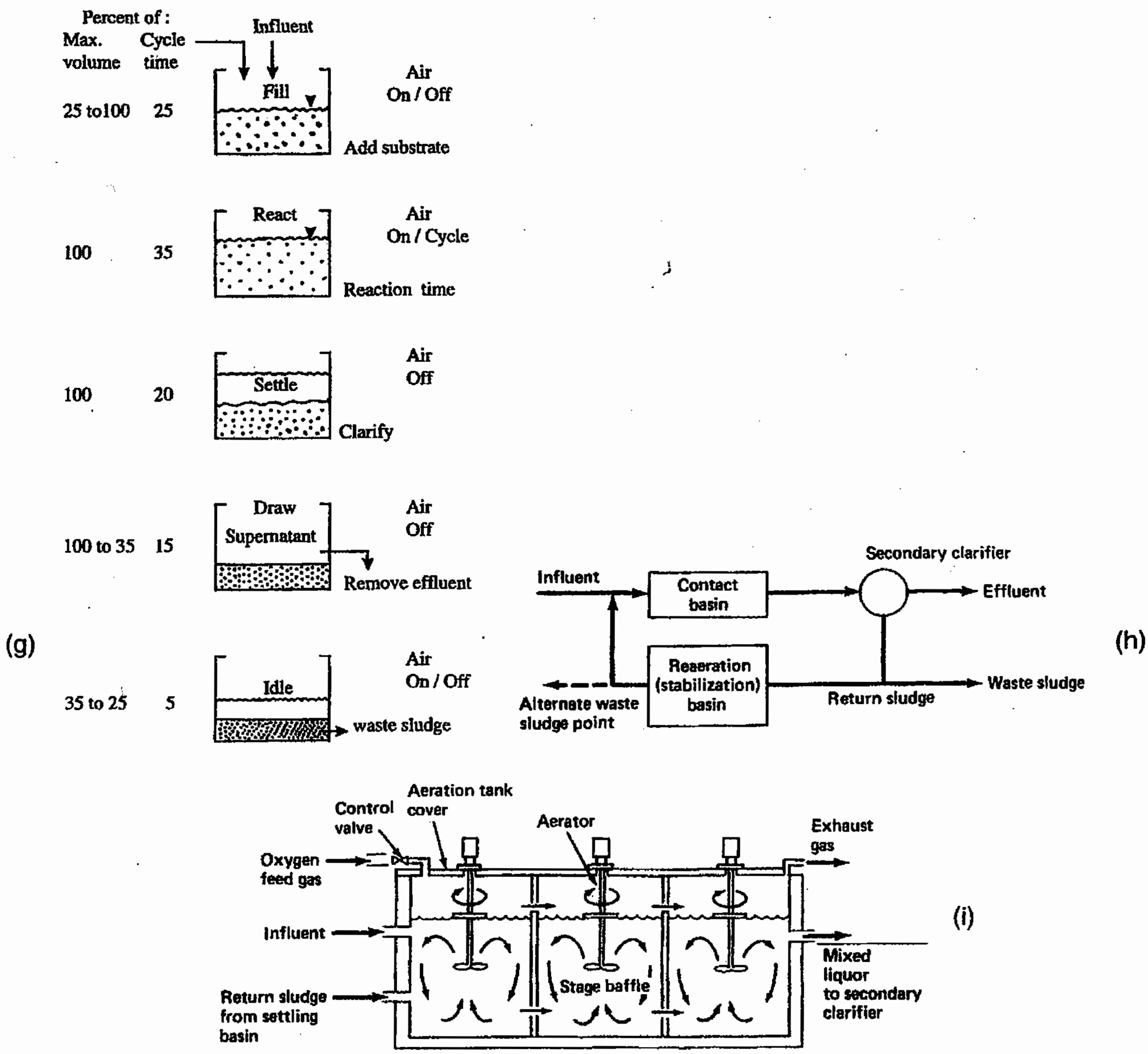

Figure 13-5—cont'd (g) operational stages of sequencing batch reactor, (h) contact stabilization, and (i) multistage pure oxygen system.

pressor. The factors affecting the oxygen transfer are bubble size, diffuser air rate, diffuser placement, and velocity of the surrounding medium.

The air diffusers are of various types. Common types include the bubble diffuser, tubular diffuser, jet diffuser, aspirator device, and U-tube.[18,19] Brief descriptions and performance of many types of air diffusers are given in Table 13-4. Typical diffused air aeration devices are illustrated in Figure 13-6. Discussion on diffused aeration equipment and pure oxygen systems may be obtained in Refs. 4, 11, and 18–21.

The air piping consists of header pipe, mains, valves, meters, and other fittings that transport compressed air from the blower to the diffusers. Different types of diffuser arrangements and pipings are shown in Figure 13-7. The basic design considerations for piping systems are listed below:

1. Piping is sized such that the head losses in the piping system are small in comparison to those in the diffusers.

Text continued on page 406.

TABLE 13-4 *Characteristics of Typical Diffused Aeration Devices*

Aeration System	Description	Advantages	Disadvantages	Transfer Efficiency (percent)	Transfer Rate (standard $kgO_2/kW\cdot h$)[a]
Diffused air system	Air is introduced near the bottom of the tank through porous or nonporous diffusers. Oxygen transfer and mixing occur as air bubbles rise to the surface.				
Fine bubble	Porous plates, dome, disc, or tubes made of ceramic media such as bonded grains of fused crystalline aluminum oxide, vitreous-silicate-bonded grains, or resin-bonded grains of pure silica [Figure 13-6 (a)–(d)]	Good mixing, varying air flow provides good operational flexibility, and good oxygen transfer	High initial and maintenance costs, air filter needed	10–30	1.2–2.0
Medium bubble	Made of a perforated stainless steel tube and coated with spiral winding of saran cord, or covering with woven fabric sock or sleeve to form a tube 7.5 cm diameter and 61 cm long [Figure 13-6(e)]	Good mixing, lower maintenance cost as wrappers or sock could be changed, used to produce spiral flow	High initial cost, air filter may be needed.	6–15	1.0–1.6

Coarse bubble	Various nozzles or orifices with check-valve feature; sparger air escapes from periphery of a flexible disc that may lift over its seat under air pressure; slot orifice injector [Figure 13-6(f),(g)]	Nonclogging; low maintenance; air filter not needed, used to produce spiral flow	High initial cost; low oxygen transfer, high power cost	4–8	0.6–1.2
Tubular system or static tube	In tubular system air flows upward through a tortuous pathway within a tube that may be of different height (0.5 to 1.25 m). Alternately placed deflection plates increase air–wastewater contact. Mixing and oxygen transfer is accomplished because the tube aerator acts as air-lift pump [Figure 13-6(h)]. Static tubes are stationary vertical tubes mounted on basin floor. A series of diffuser membranes may be arranged. Static tubes also function like an air-lift pump [Figure 13-6(i)].	Low initial cost, low maintenance, high transfer efficiency	Low mixing	7–10	1.2–1.6
Jet	Compressed air and liquid are mixed and discharged horizontally. The rising plume of fine bubbles produces mixing and oxygen transfer [Figure 13-6(j)].	Moderate cost, suited for deep tanks, high transfer efficiency	Requires blower and pumping equipment, nozzle clogging	10–25	1.2–2.4

Continued

TABLE 13-4 Characteristics of Typical Diffused Aeration Devices—cont'd

Aeration System	Description	Advantages	Disadvantages	Transfer Efficiency (percent)	Transfer Rate (standard $kgO_2/kW\cdot h$)[a]
Aspirator Jet	Consist of motor-driven propeller aspirator. The propeller rotation draws air through hollow tubes and injects it underwater. Air velocity and propeller action create turbulence and fine bubbles near the tip [Figure 13-6(l)].	High efficiency	Requires propeller and pumping equipment; needs mounting on fixed structure or on pontoons; high maintenance	20–30	1.5–2.5
U-tube	The aeration device consists of outer draft pipe and a central downcomer. The air bubbles rise into the draft tube. High static pressure and air–water mixture cause greater solubility of oxygen. The system acts as an air-lift pump [Figure 13-6(m)].	High oxygen transfer efficiency. Suited for very deep tanks. Can also act as air pump. Low maintenance as all moving parts are above liquid.	Requires high pressure blower	15–20	1.3–2.4

[a]Standard conditions: tap water, 20°C, at 101.325 kN/m^2(1 atm), and initial dissolved oxygen = 0 mg/L.
$kg/kW\cdot h \times 1.644 = lb/hp\cdot h$.
$kN/m^2 \times 0.145 = lb/in^2$.
Source: Adapted in part from Refs. 4, 5, 7, 11, and 12.

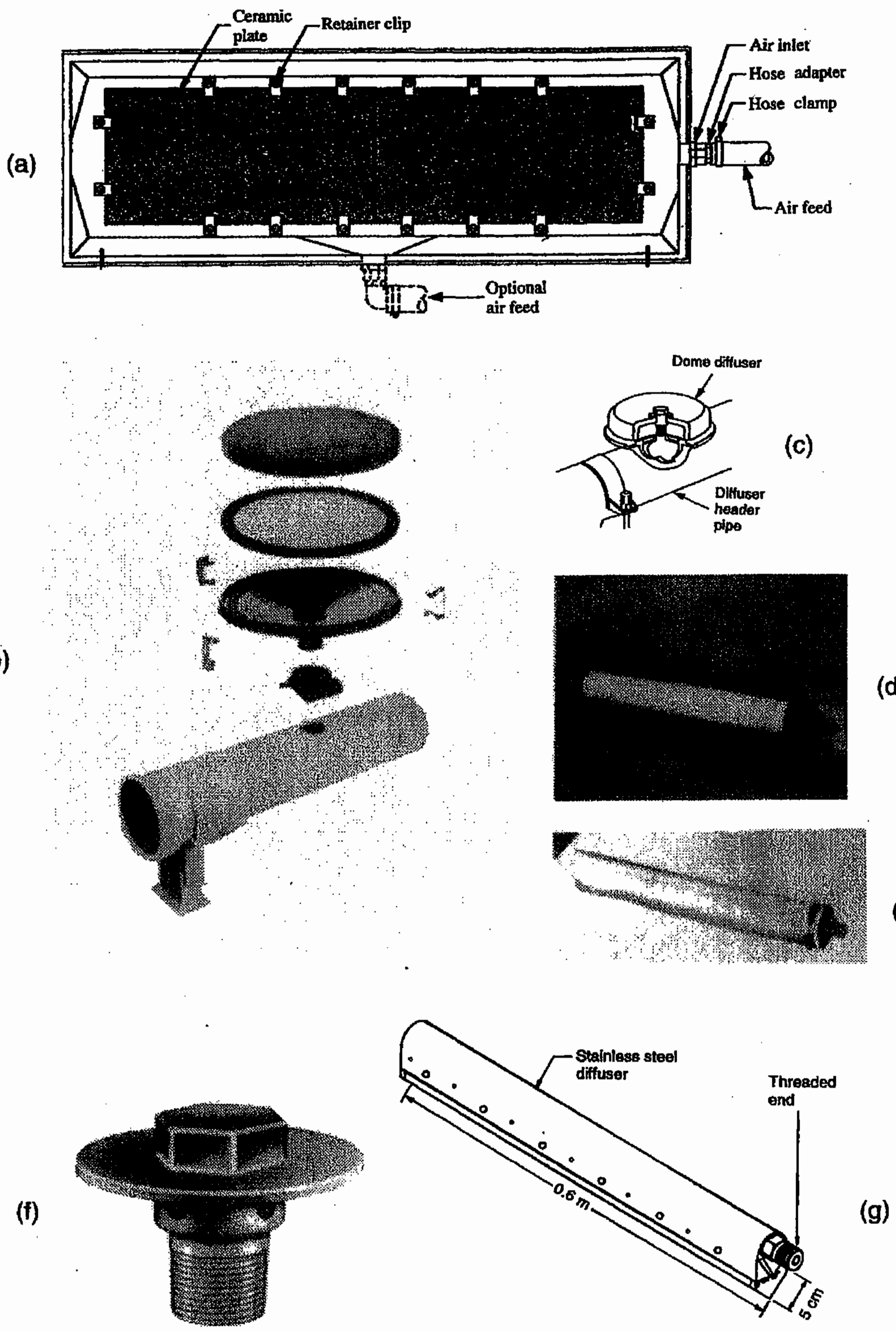

Figure 13-6 Various Types of Diffused Aeration Devices (from Refs. 4, 11, and 18): (a) ceramic plate, (b) ceramic disc (courtesy Envirex, U.S. Filter), (c) dome, (d) ceramic tube (courtesy Envirex, U.S. Filter), (e) tube with fine sheath and holding clamp (courtesy FMC, U.S. Filter), (f) orifice (courtesy FMC, U.S. Filter), (g) stainless steel tube.

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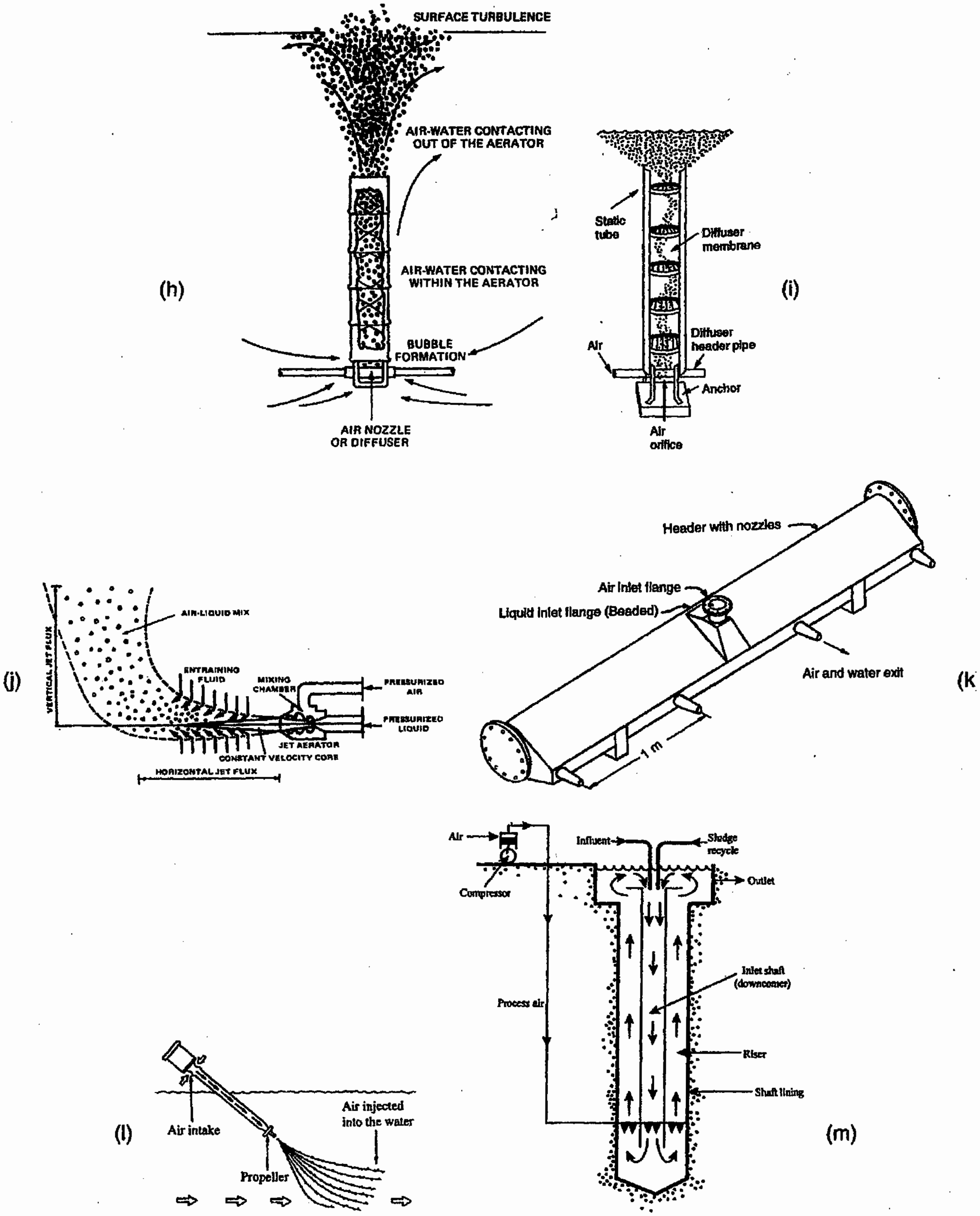

Figure 13-6—cont'd (h) tubular (from Ref. 20, used with permission of WEF and ASCE), (i) static tube, (j) jet aerator (from Ref. 20, used with permission of WEF and ASCE), (k) jet aerator with header and nozzles, (l) aspirator, and (m) U-tube aerator.

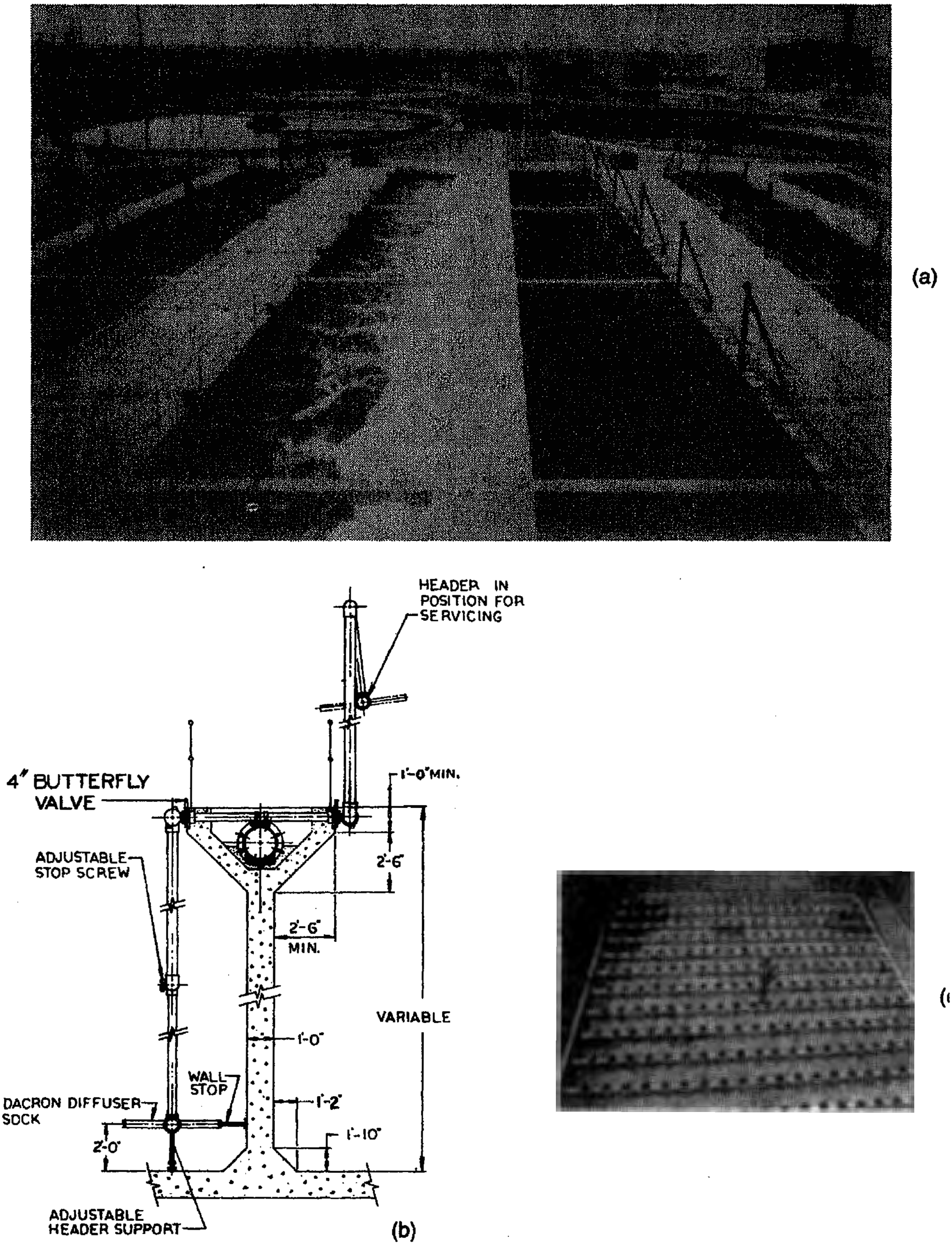

Figure 13-7 Diffused Aeration System Installation: (a) diffused aeration system assembly (courtesy FMC, U.S. Filter); (b) rotary lift diffuser showing header in position for servicing (courtesy Envirex, U.S. Filter); and (c) ceramic grid aeration system (courtesy Sanitaire Water Pollution Control Corp.).

2. Piping must be of corrosion-resistant material (stainless steel, galvanized steel, fiberglass, plastic, etc.).
3. Valves should be provided for flow regulation.
4. Piping losses should be calculated for maximum summer temperatures, taking into account the theoretical adiabatic temperature rise during compression.
5. Friction loss in the piping is calculated using the Darcy-Weisbach or Hazen-Williams equations.
6. With porous diffusers producing fine- to medium-sized bubbles, it is important to have swing-lift piping to permit maintenance without dewatering the tank.

A procedure for designing the air-piping system is covered in the Design Example. The blower or compressor is designed to supply the required amount of air at the design pressure. Two types of blowers are in common use: (1) centrifugal and (2) rotary positive displacement. A procedure for determining the power requirements and blower selection is presented in the Design Example.

Mechanical Aeration. The mechanical aerators consist of submerged or partly submerged impellers that are attached to motors mounted on a float or on fixed structures. The oxygen is entrained from the atmosphere. The mechanical aerators fall into two major groups: aerators with vertical axis and aerators with horizontal axis. Both groups are further divided into surface and submerged aerators. Brief descriptions and performance of various types of mechanical aerators are provided in Table 13-5.

Aeration Basin. Aeration basins are generally rectangular tanks constructed of reinforced concrete. Important design factors of aeration basins are given below:

1. The depth of the aeration basin is 3–5 m (10–16 ft), with 0.3–0.6 m (1–2 ft) freeboard.
2. For spiral flow mixing the width-to-depth ratio is 1.0:1 to 2.2:1. This limits the width of a tank by 3–11 m (10–36 ft).
3. If the aeration tank volume exceeds 140 m^3 (5000 ft^3), two or more units should be provided. Each unit should be capable of independent operation.
4. Common-wall construction should be used for multiple basins.
5. Exceptionally long tanks should utilize multiple channels using around-the-end-flow baffles.
6. Avoid dead spots by providing baffles and fillets in the corners.
7. The foundation should be designed to prevent settlement and prevent flotation when tank is empty.
8. The inlet and outlet structures should be designed to permit removal of an individual tank from service for routine maintenance.
9. Suitable arrangement for draining the aeration basin should be made.
10. Froth control system should be provided by installing an effluent spray nozzle along the length on the opposite side of the diffuser. Provision for adding an antifoaming agent into the spray water is often made.

Text continued on page 412.

TABLE 13-5 Characteristics of Mechanical Aeration Devices

Aeration System	Description	Advantage	Disadvantage	Transfer Efficiency (percent)	Transfer Rate (standard $kgO_2/kW{\cdot}h$)[a]
Vertical axis	The shaft is vertical, and blades are attached in the shaft. The motor sits on top of a fixed or floating platform.				
Surface aerator	The impeller is submerged or partly submerged. The radial flow aerator is low speed (20–100 rpm) and has a gear box to reduce speed. The motor is mounted on the float or on a fixed structure. The motor action induces updraft or downdraft flow. Draft tube may be placed below the impeller to induce circulation [Figure 13-7(a)(i–iii)].	Flexibility in tank shape and size; good mixing	Initial cost high, icing in cold climate; gear reducer may cause maintenance problem.	—	1.2–2.7
	The high-speed surface aerators (axial flow) have speeds of 300 to 1200 rpm and are mostly mounted on floats [Figure 13-7(b)] used in aerated lagoons.	Low initial cost, can be adjusted to varying water level, flexible operation	Icing in cold climate, poor accessibility for maintenance, mixing inadequate		1.2–2.7

Continued

TABLE 13-5 Characteristics of Mechanical Aeration Devices—cont'd

Aeration System	Description	Advantage	Disadvantage	Transfer Efficiency (percent)	Transfer Rate (standard $kgO_2/kW \cdot h$)[a]
Submerged or turbine aerator	The impeller is submerged and piped air or oxygen is delivered to a point below the impeller. The impeller disperses the air into fine bubbles and mixes the contents of the tank. Draft tube may also be used to increase circulation. Air flow may vary from 4 to 8 L/s [Figure 13-7(c)(i–iii)].	Good mixing, suitable for deep tank, operational flexibility, no icing or splash	High initial cost, require both gear reducer and blower, high power requirement	15–35	1.2–2.0
Horizontal axis	The aerator has a horizontal axis. A cylinder or drum either exposed or submerged provides aeration and forward movement of the liquid. Commonly used in oxidation ditch.				

Surface aerator or brush aerator	Consists of cylinder or drum with bristles of steel protruding from the perimeter into wastewater; provide aeration and move the liquid forward [Figure 13-7(d)(i)]	Provide aeration and circulation; moderate initial cost, good maintenance accessibility	Tank geometry is limited, gear reducer, low efficiency	—	1.—2.0
Submerged disc aerator	Consists of discs that are submerged in the liquid approximately one-eighth to three-eighth of the diameter. The recesses in the disks introduce entrapped air into the submerged section. The disc spacing and submergence can vary depending upon the oxygen requirement [Figure 13-7(d)(ii)].	Same as surface aerator	Tank geometry is limited, gear reducer	—	1.2–2.4

[a]Standard conditions: tap water, 20°C, at 101.325 kN/m^2 (1 atm), and initial dissolved oxygen = 0 mg/L.
kg/kW·h × 1.644 = lb/HP·h.
kN/m^2 × 0.145 = lb/in^2.
Source: Adapted in part from Refs. 4, 7, 11, and 12.

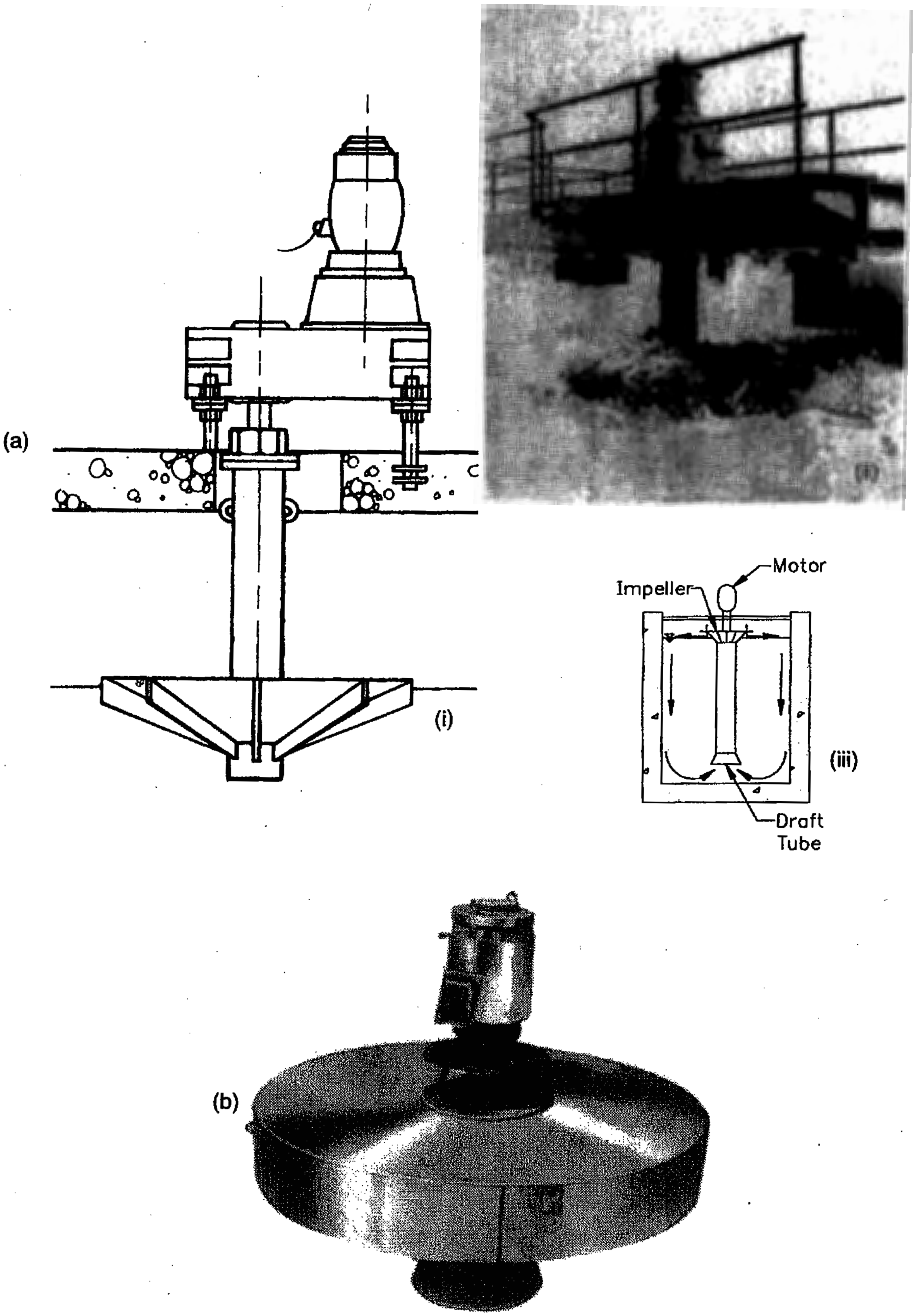

Figure 13-8 Mechanical Aerators: (a) vertical axis, low-speed surface aerator: (i) impeller and motor arrangement, (ii) in operation (courtesy EIMCO Process Equipment Co.), (iii) surface aerator with draft tube; (b) vertical axis floating aerator (courtesy Aqua-Aerobic Systems, Inc.).

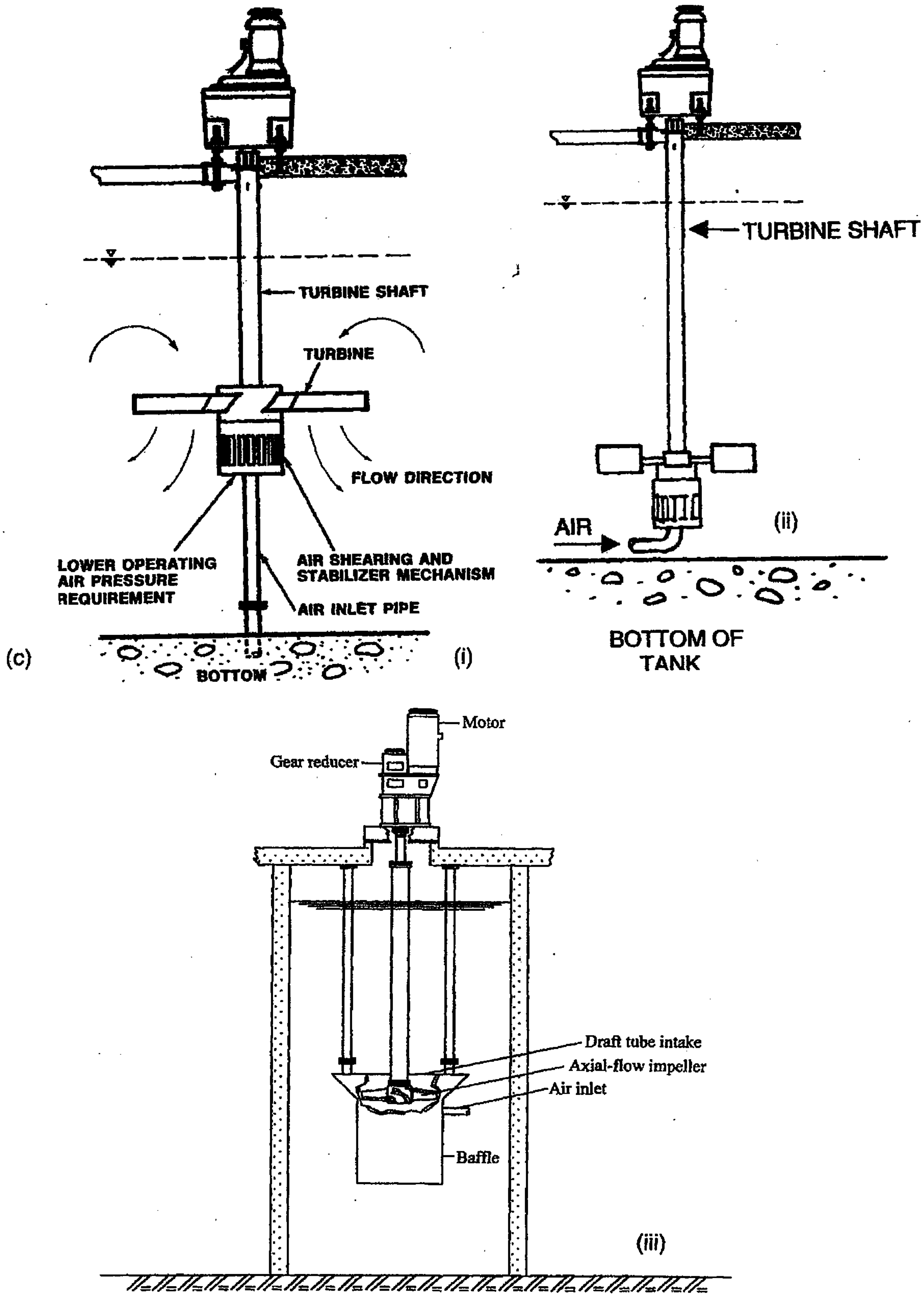

Figure 13-8—cont'd (c) vertical axis submerged turbine aerator: (i) axial flow (courtesy EIMCO Process Equipment Co.), (ii) radial flow, (iii) submerged aerator with draft tube; (d) horizontal shaft aerator: (i) brush type (courtesy Smith & Loveless, Inc.), (ii) disc type (courtesy Envirex, U.S. Filter).

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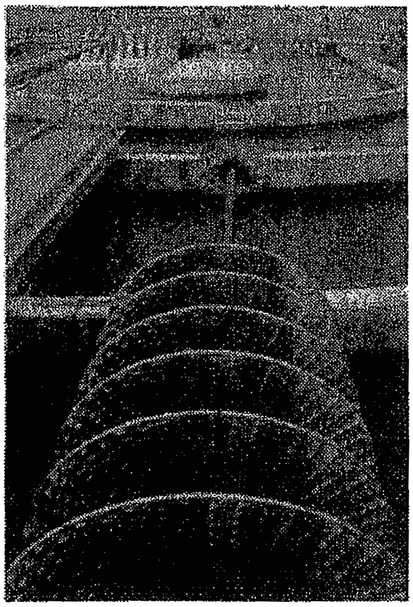
(d) (i)



(ii)

Figure 13-8—cont'd (d) horizontal shaft aerator: (i) brush type (courtesy Smith & Loveless, Inc.), (ii) disc type (courtesy Envirex, U.S. Filter).

Solids Removal System. The mixed-liquor suspended solids must be settled in a sedimentation basin to produce well-clarified effluent. The design criteria and design procedure for solids removal systems have been presented in Chapter 12. The secondary clarifier in general must perform two functions: (a) provide clarification to produce high-quality effluent and (b) provide thickening of settled solids. The secondary clarifiers are also used in conjunction with suspended and attached growth reactors systems of aerobic, anaerobic, and biological nutrient removal processes. Therefore, in-depth design information on secondary clarifiers is provided after completing the discussion on biological treatment processes and reactor design. Readers may refer to Sec. 13-11-8 for design information on secondary clarifiers.

Return Sludge System. The settled sludge is returned from the clarifier to the aeration basin to maintain the desired food-to-microorganism ratio. The return sludge system is designed for a total capacity of 50–150 percent of the average flow. The most common operational range is 20–50 percent. The return flow requirement is determined from some simple tests in the field or in the laboratory. One most common method is to measure the depth of sludge blanket in the basin with a *sludge judge.* Another method is to determine the volume of settled sludge in 30 min in a 1-L graduated cylinder filled with MLSS. The return sludge ratio is calculated from Eq. (13-28):

$$Q_r/Q = \frac{[\text{vol. of settled sludge, mL}]}{[1000\text{ mL} - \text{vol. of settled sludge, mL}]} \tag{13-28}$$

The most accurate method of determining the return sludge is based on determination of *sludge volume index* (SVI), which is defined as the volume occupied in milliliters by 1 g of settled sludge. It is also defined by the ratio of percent volume occupied by settled sludge in 30 min and MLSS concentration in percent. The SVI test has been used traditionally to determine the settling behavior of MLSS. An SVI below 100 is an indication of excellent settling sludge. Poor settling is characterized by high SVI. At SVI values above 200, the sludge from a conventional aeration basin does not settle well and fills up the clarifier. The return sludge ratio is calculated from Eq. (13-29):

$$Q_rQ = \left[\frac{100}{\text{MLSS concentration }\% \times \text{SVI}} - 1\right]^{-1} \tag{13-29}$$

Waste-Activated Sludge. The excess sludge is wasted either from the effluent line of the aeration basin or from the return sludge line. The waste sludge from the aeration basin is quite thin (0.2–0.6 percent solids). Thickening may be achieved by returning it to the primary sedimentation basin or by installing the proper thickening device. The waste sludge from secondary clarifier is considerably thicker (0.5–1.2 percent solids). The waste-activated sludge may be thickened separately or may be mixed with the primary sludge, and the combined sludge may be thickened. Various types of sludge thickening devices are discussed in Chapter 16.

Filamentous Growth and Sludge Bulking. Although sludge bulking is a part of an operational problem, this topic is briefly covered here because solids separation is an integral part of process design. Sludge bulking is the rising of sludge, poor settling, or foaming. The main causes of sludge bulking are (1) characteristics of wastewater, (2) design limitations, and (3) plant operation. Fluctuations in flow and strength, pH, temperature, nutrients, and nature of wastes are related to wastewater characteristics. The design limitations may include an insufficient capacity of aeration, mixing, and return sludge. The plant operation factors constitute low DO, nutrient limitations, low *F/M* ratio, excessive aeration, and *Nocardia* growth. Sludge bulking, troubleshooting and control measures are covered in the operation and maintenance section of this chapter (Sec. 13-12).

Kinetic Equations for Design of Activated Sludge Process. The basic design considerations for an activated sludge process involve (a) selection of reactor type, (b) organic loading, (c) aeration requirements, (d) sludge growth and wasting, (e) return sludge, and (f) effluent characteristics. Many generalized kinetic equations have been presented in earlier sections that express some of these design factors. Quite often, these kinetic equations are modified for field conditions so that they may be used for reactor sizing and performance evaluation. The application of many of these equations with proper modifications, units of expression, and range and typical values of various parameters for designing a biological reactors are presented in the Design Example.

13-3-2 Other Applications

Two other applications of a suspended growth biological treatment process include aerated lagoon and stabilization ponds. Both these applications utilize earthen or lined basins for treatment of relatively small flows. A brief discussion on both applications is given below.

Aerated Lagoon. The aerated lagoons are suspended growth reactors in earthen basins with no sludge recycle. Mechanical aerators are normally used for mixing and supplying oxygen demand. Since the aerated lagoons have a large detention period (2–6 days), a certain amount of nitrification is achieved. Higher temperatures and suspended solids and lower organic loadings generally encourage nitrification.

Design of aerated lagoons is similar to an activated sludge process with no recycle. The design equations for an activated sludge system derived from soluble substrate removal kinetics have been discussed in the preceding sections. The procedure for designing an aerated lagoon may be obtained in Refs. 11, 22, and 23.

In the absence of a clarifier, the concentration of suspended solids in the effluent from an aerated lagoon is high. Although the aerated lagoons are designed as completely mixed reactors, a certain amount of settling does occur in different parts of the basin. Aerated lagoons produce effluents that have suspended solids concentrations in the range of 80–250 mg/L. To meet the secondary effluent standards, settling basins have been added to the existing aerated lagoons. A clarification facility using the concept of intrachannel clarifier can be economically provided within an existing aerated lagoon to effectively remove TSS from the effluent. Two types of clarification facilities using this concept are presented in Chapter 12 [Figures 12-6(c) and (d)]. Chemical coagulation and clarification of effluent from aerated lagoons will produce well-nitrified effluent that is also low in phosphorus. Design details of aerated lagoons are given in Figure 13-9.

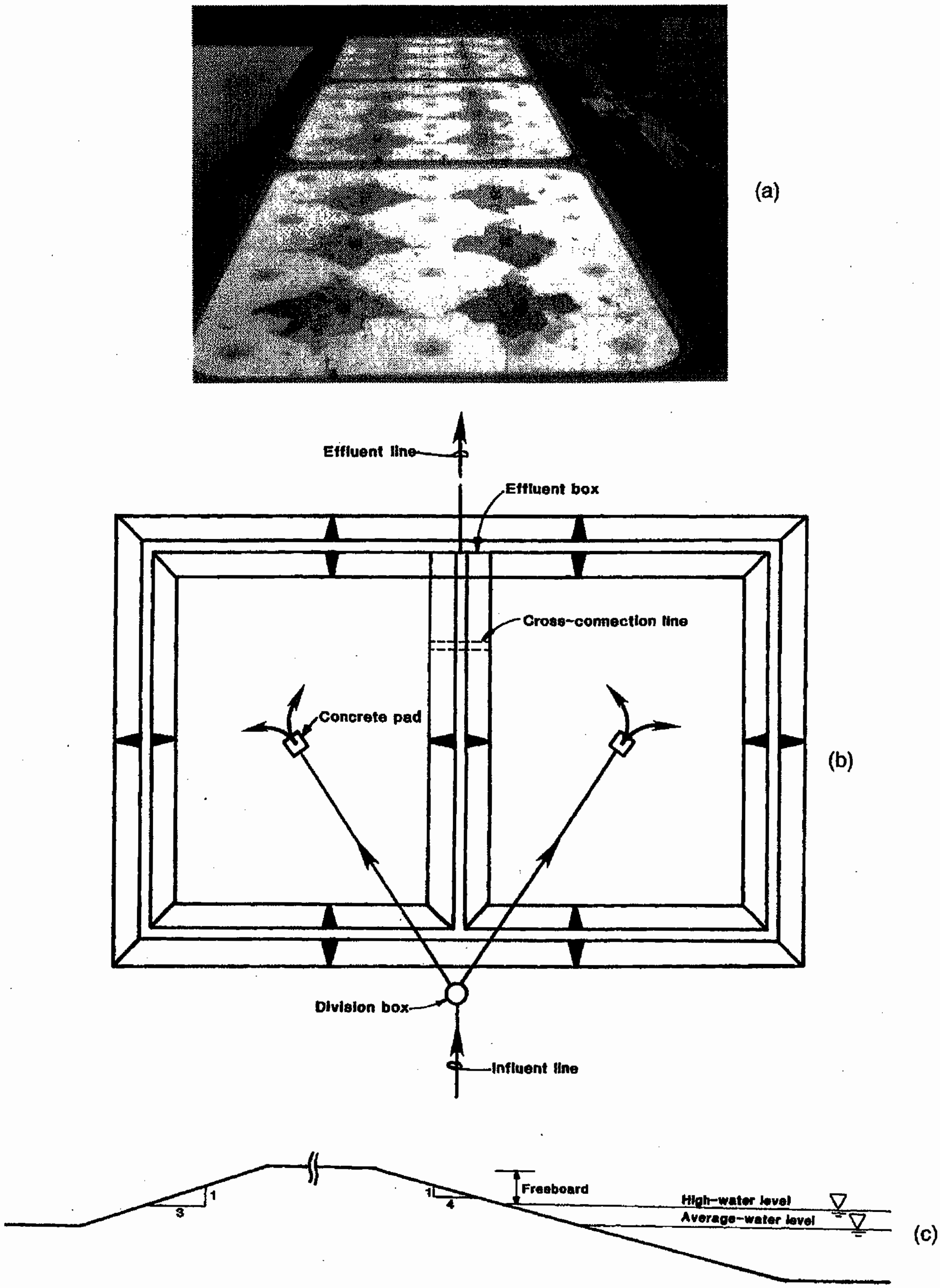


Figure 13-9 Design Details of Aerated Lagoon and Stabilization Ponds: (a) aerated lagoons (courtesy Aqua-Aerobic Systems, Inc.), (b) plan of stabilization pond, and (c) typical cross section of levees of stabilization pond.

Stabilization Pond.[b] A stabilization pond is a relatively shallow body of water contained in an earthen basin of certain shape that is designed to treat wastewater. These ponds have become a popular means of wastewater treatment for small communities and industries that produce organic waste streams. The stabilization ponds have the advantage of low construction and operation costs. The major disadvantages are large land area required, ordor and insect problems, possible groundwater contamination, and poor effluent quality.

Process Description. In a stabilization pond solids settle to the bottom. A wide variety of microscopic plants and animals find the environment a suitable habitat. Organic matter is metabolized by bacteria and protozoa as primary feeders. Secondary feeders include protozoa and higher animals such as rotifers and crustaceans. The nutrients released are utilized by algae and other aquatic plants. The main sources of oxygen are natural reaeration and photosynthesis. In the bottom layer, the accumulated solids are actively decomposed by anaerobic bacteria.

Types of Stabilization Ponds and Design Considerations. The stabilization ponds are usually classified as aerobic, facultative, and anaerobic. This classification is based on the nature of the biological activity taking place. Design factors such as depth, detention time, organic loading, and effluent quality also vary greatly for the three types of lagoons. Brief descriptions of the three types of lagoons, their design factors, and their effluent qualities are given in Table 13-6.

Effluent Quality. The effluent quality from a stabilization pond is poor and does not meet the EPA secondary treatment criteria. Although the effluent is low in soluble BOD_5, it is high in total suspended solids and total BOD_5. High values of total suspended solids and total BOD_5 are attributed to algae. If some type of solids removal system is used in conjunction with lagoon treatment, a high quality of effluent that is also low in nutrients can be obtained. Some solids removal techniques that have been used or are under investigation for removal of algae from the pond effluent are as follows: (1) coagulation and clarification, (2) dissolved air flotation, (3) microscreening, (4) sand filtration, (5) rock filters, and (6) specialized algae-harvesting devices. A coagulation, flocculation, and sedimentation facility can be integrated within a stabilization basin using the concept of a DensaDeg® clarifier [Figure 12-6(a)]. A discussion of all these methods of algae removal from lagoon effluent and design details may be found in Refs. 4, 11, and 24–27. Design details of stabilization ponds are shown in Figure 13-9.

13-4 AEROBIC ATTACHED GROWTH AND COMBINED ATTACHED AND SUSPENDED GROWTH BIOLOGICAL TREATMENT

In attached growth biological treatment processes the population of active microorganisms is developed over a solid media (rock or plastic). The attached growths of microorganisms stabilize the organic matter as the wastewater passes over them. There are two

[b]The terms *oxidation pond* and *lagoon* are also used for stabilization pond.

TABLE 13-6 Process Description and Design Parameters for Aerobic, Facultative, and Anaerobic Stabilization Ponds

Parameter	Aerobic (high rate)	Aerobic-Anaerobic (facultative)	Anaerobic
Process description	Aerobic condition prevails throughout the entire depth. Bacteria and algae remain in suspension. Principal sources of oxygen are natural reaeration and photosynthesis, which is governed by solar energy. Design is based on organic loading, first-order reaction kinetics, or by equating the oxygen resources of the pond to the applied organic loading. The BOD_5 removal rate constant is dependent on temperature, mixing, solar radiation, and type of wastewater treated.	The upper layer is aerobic zone (maintained by algae and natural reaeration). Lower layers are facultative. Bottom layer of solids undergoes anaerobic decomposition. The design of facultative pond is based on organic loading and reaction kinetics. Accumulation of bottom sludge is also given consideration in oxygen utilization.	Anaerobic conditions prevail throughout the pond. Design is based on the principles of anaerobic digesters with no mixing. Design details of anaerobic digesters are provided in Chapter 17. The ponds are associated with undesirable odors. Anaerobic ponds are normally used where concentrated industrial waste streams are treated. Examples are slaughterhouses, dairies, and canning and meat-processing plants.
Detention time, days	5–20	10–30	20–50
Water depth, m	0.3–1[a]	1–2	2.5–5
BOD_5 loading kg/ha·d	40–120[b]	15–120	200–500
Soluble BOD_5 removal, percent	90–97	85–95	80–95
Overall BOD_5 removal, percent	40–80[c]	70–90	60–90
Algae concentration, mg/L	100–200	20–80	0–5
Effluent TSS, mg/L	100–250	40–100	70–120

[a] 1 m = 3.28 ft.
[b] 1 kg/ha·d = 0.8922 lb/acre·d.
[c] Overall BOD_5 removal is small because of high concentration of algae in the effluent.
Source: Adapted in part from Refs. 4, 11, and 24–27.

major types of attached growth processes: (1) trickling filters and (2) rotating biological contactors. Both of these processes are discussed below.

13-4-1 Trickling Filter

The trickling filter consists of a shallow bed filled with crushed stones or synthetic media. Wastewater is applied on the surface by means of a self-propelled or mechanical rotary distribution system. The organics are removed by the attached layer of microorganism (slime layer) that develops over the media. The underdrain system collects the trickled liquid that also contains the biological solids detached from the media. The air circulates through the pores because of a natural draft caused by thermal gradient. The trickled liquid and detached biological solids are settled in a clarifier. A portion of the flow is recycled to maintain a uniform hydraulic loading and to dilute the influent. The details of trickling filter and typical recirculation patterns are shown in Figure 13-10.

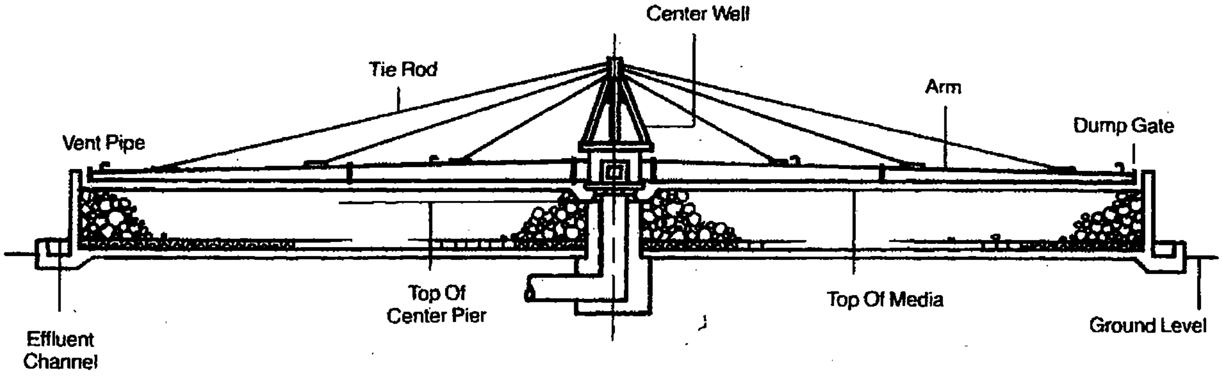
Types of Trickling Filters. Based on the organic and hydraulic loading, the trickling filters are classified into low-rate, intermediate-rate, high-rate, and super-rate (roughing filters). Often, two-stage trickling filters (two trickling filters in series) are used for treating high-strength wastes. Typical design information for different types of trickling filters are summarized in Table 13-7.

Design Methods. Many design equations and procedures for designing trickling filters have been proposed over the years. Some of these equations were developed by the National Research Council (NRC),[28] Velz,[29] Eckenfelder,[30] Galler and Gotaas,[31] Schulze,[32] and Logan et al.[33,34] Discussions on these equations, design procedures, and design examples may be obtained from several sources.[4-8,11,12,35-37]

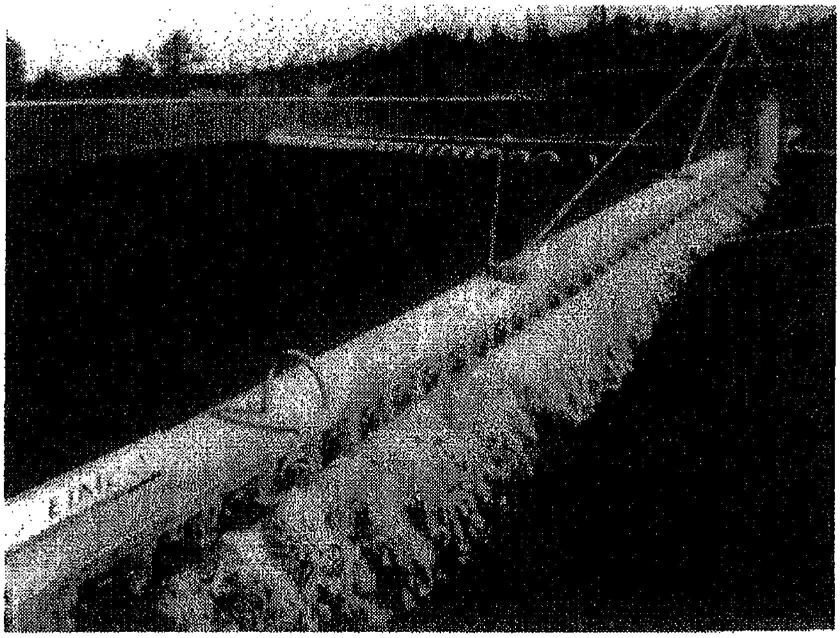
13-4-2 Rotating Biological Contactor (RBC)

A rotating biological contactor (also called bio-disc process) consists of a series of circular plastic plates (discs) mounted over a shaft that rotates slowly. These discs remain approximately 40 percent immersed in a contoured bottom tank. The discs are spaced so that wastewater and air can enter the space. The biological growth that develops over the discs receives alternating exposures to organics and the air. The excess growth of microorganisms becomes detached, and therefore, the effluent requires clarification. The rotating biological contactor has a low power demand, greater process stability, higher organic loadings, and a smaller quantity of waste sludge than the waste activated sludge.

The RBC may be designed for BOD removal, combined nitrification, and separate nitrification. The staging configuration of the RBC system is an integral part of the overall design process. Staging is a compartmentalization of the RBC units to form a series of cell operation. The hydraulic and organic loading may vary, respectively, in the range of 0.03–0.16 $m^3/m^2{\cdot}d$ (0.75–4.0 $gal/ft^2{\cdot}d$) and 0.001–0.03 kg soluble $BOD_5/m^2{\cdot}d$ (0.2–6 $lb/ft^2{\cdot}d$). The RBC design features are shown in Figure 13-11. Technical discussion and design information may be obtained from Refs. 4, 11, 22, and 37–40.



(a



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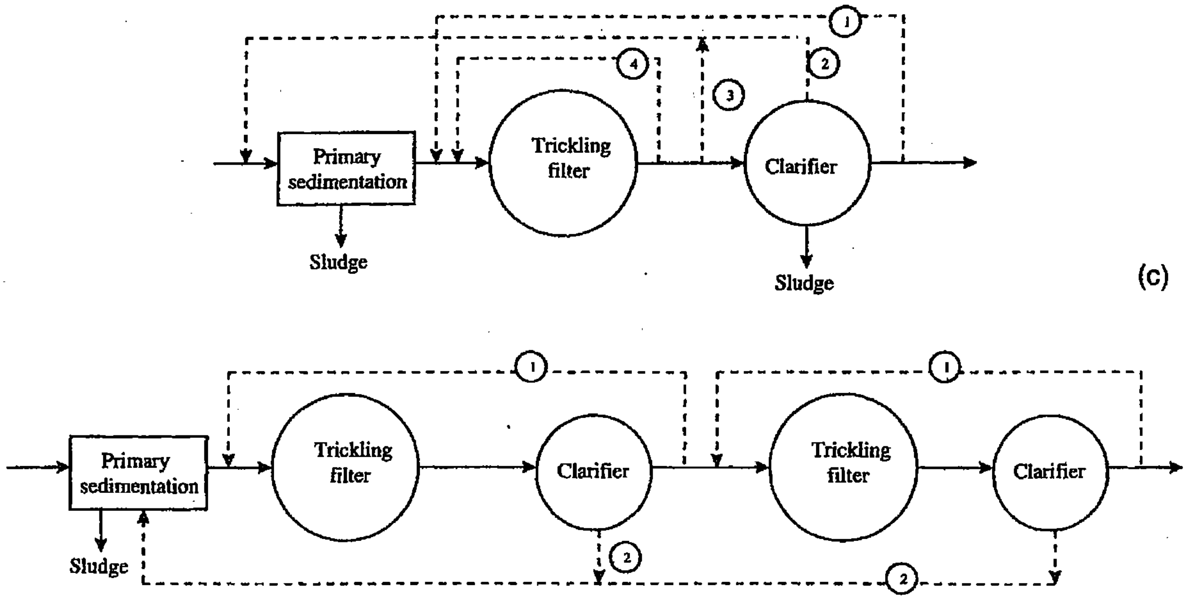


(c)

Figure 13-10 Trickling Filter: (a) trickling filter assembly (courtesy EIMCO Process Equipment Company); (b) trickling filter installation (courtesy EIMCO Process Equipment Company); and (c) trickling filter recirculation patterns: (i) intermediate and high rate filter, (ii) two-stage filters. Various recirculation options are numbered.

TABLE 13-7 *Typical Design Information for Different Types of Trickling Filters*

Item	Low Rate	Intermediate Rate	High Rate	Super Rate or Roughing	Two-Stage
Description	Shallow depth, simple design and dependable operation; consistent effluent quality. Design tank is small.	Shallow depth; recirculation maintains uniform hydraulic loading. Dependable operation; consistent effluent quality. Many recirculation patterns are used.	Shallow depth, but high hydraulic loading. Effluent quality is consistent and dependable. Many recirculation patterns are used.	Deep bed; operation similar to packed bed; high organic and hydraulic loading. Many recirculation patterns are used.	Two fillers with intermediate clarifier are often used. Suitable for high-strength wastewater.
Operation	Intermittent	Continuous	Continuous	Continuous	Continuous
Recirculation ratio	0	0–1.0	1.0–2.5	1.0–4.0	0.5–3.0
Depth, m	1.5–3.0	1.25–2.5	1.0–2.0	4.5–12	2.0–3.0
Hydraulic loading, $m^3/m^2{\cdot}d$	1–4	4–10	10–40	40–200	10–40
BOD_5 loading, $kg/m^3{\cdot}d$	0.08–0.32	0.24–0.48	0.32–1.0	0.8–6.0	1.0–2.0
Sloughing	Intermittent	Intermittent	Continuous	Continuous	Continuous
Media	Rock, slag	Rock, slag	Rock, slag, synthetic	Synthetic	Rock, slag, synthetic
Filter flies (*Psychode*)	Many	Medium	Small	None	None
Power $kW/10^3m^3$	2–4	2–8	6–10	10–20	6–10
BOD_5 removal efficiency, percent	74–80	80–85	80–85	60–80	85–95
Effluent	Well-nitrified	Well-nitrified	Little nitrification	Little nitrification	Well-nitrified

Source: Refs. 4, 11, 22, and 30–37.

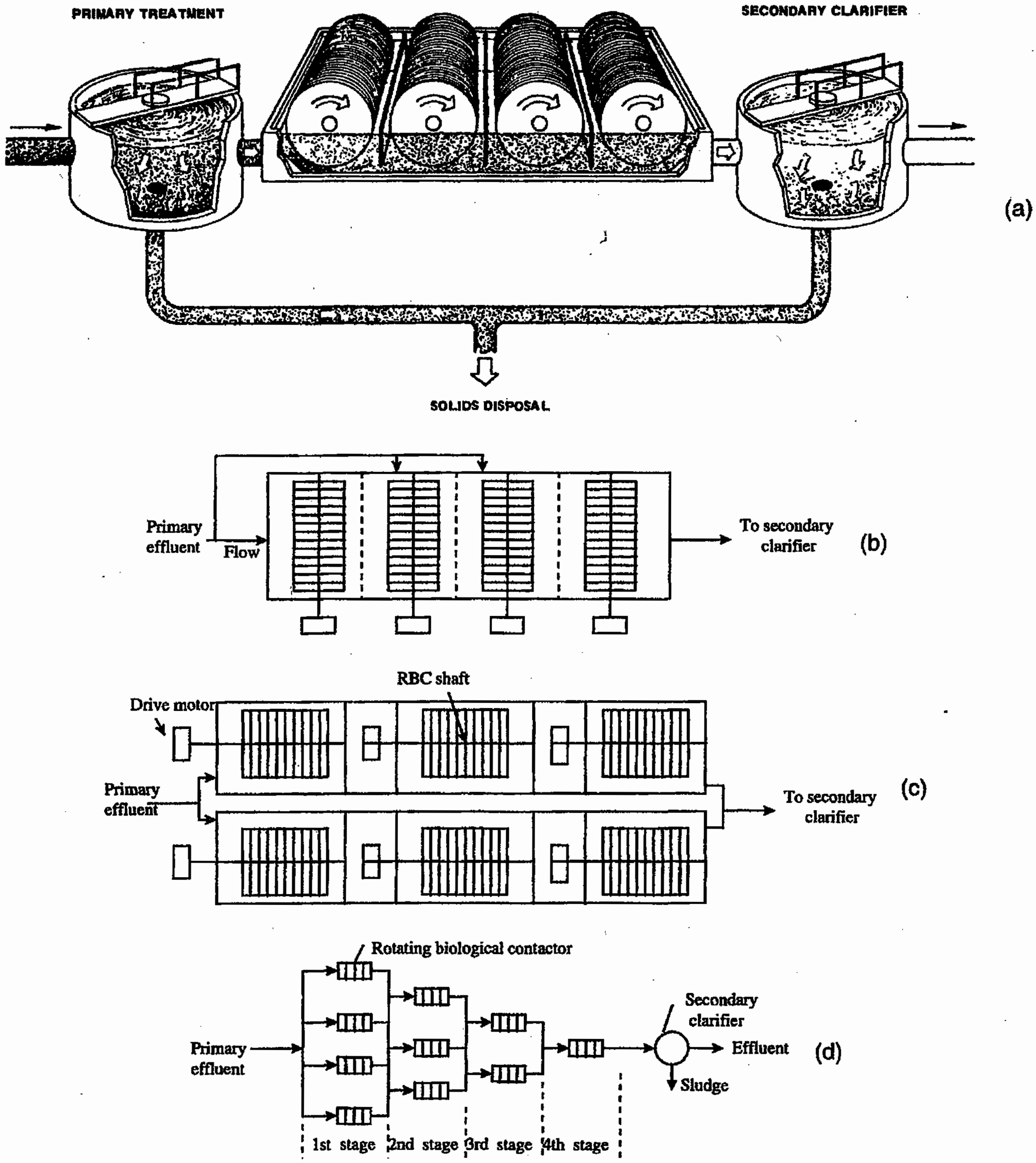


Figure 13-11 Rotating Biological Contactor: (a) system configuration, (b) flow perpendicular to shaft with step feed, (c) flow parallel to shaft, and (d) tapered stages.

13-4-3 Packed-Bed Reactor

Packed-bed reactors are deep vessels filled with a plastic media. The reactor serves as an attached growth reactor on which microorganisms become attached. Wastewater is introduced from the bottom. Air or pure oxygen is also supplied from the bottom. Carbonaceous BOD is stabilized, and nitrification can take place. The upper portion of the reactor vessel unfilled with the media may serve as a clarifier. High organic loading can be applied.[4,11]

13-4-4 Combined Attached and Suspended Growth (Dual) Processes

A combination of attached and suspended growth treatment is achieved in deep trickling filters 9–12 m (30–40 ft). These filters are circular or square and utilize plastic media. Air is supplied at the bottom of the bed. The effluent from the filter is clarified, and a portion of the sludge is returned to the filter to maintain hydraulic loading and to supply biological solids. The system differs from a conventional trickling filter because much of the active microbial growth is suspended in a manner similar to that of an activated sludge process. In many instances series arrangements of trickling filter and activated sludge processes are also used.[4,11,22,41-44] Arrangements are also called dual biological treatment processes and have been given names in accordance to the reactor's arrangement. A schematic process diagram of a dual biological process with different components is shown in Figure 13-12(a). A brief description and design information on many other process combinations are presented in Table 13-8 and shown in Figure 13-12(b)–(e).

13-5 ANAEROBIC, SUSPENDED GROWTH BIOLOGICAL TREATMENT

Anaerobic sludge digestion is one of the oldest processes used at municipal wastewater treatment plants. It is a typical example of an anaerobic, suspended growth biological treatment process. Because of the importance of anaerobic processes in stabilization of high-strength organic wastes and biological solids, the capabilities and fundamentals of anaerobic treatment processes are covered in this section. A discussion on suspended growth anaerobic processes is also included.

13-5-1 Capabilities of Anaerobic Treatment Processes

Anaerobic decomposition of organic matter occurs in the absence of oxygen. Today, an anaerobic process is looked upon as a viable alternative to aerobic system for treatment of medium to high strength wastes. Recent research has shown that the old concept that anaerobic process is "very slow" as compared to aerobic process is not always true. A comparison of anaerobic and aerobic treatment processes clearly show that anaerobic processes offer the following advantages:

- Volumetric organic loading rate is five to ten times higher, thus reduction of installation space requirements.
- Biomass synthesis is five to twenty times lesser, thus reduction of waste biomass to be managed.
- Nutrient requirements are five to twenty times lesser.
- There is no energy requirement for aeration.
- Energy is produced in the form of methane.
- Off-gases, volatile organic compounds (VOCs), and odorous compounds causing air pollution are eliminated.
- Refractory biomass is ten times less.
- Anaerobic biomass can be preserved for a long time without feed, thus better seasonal treatment.

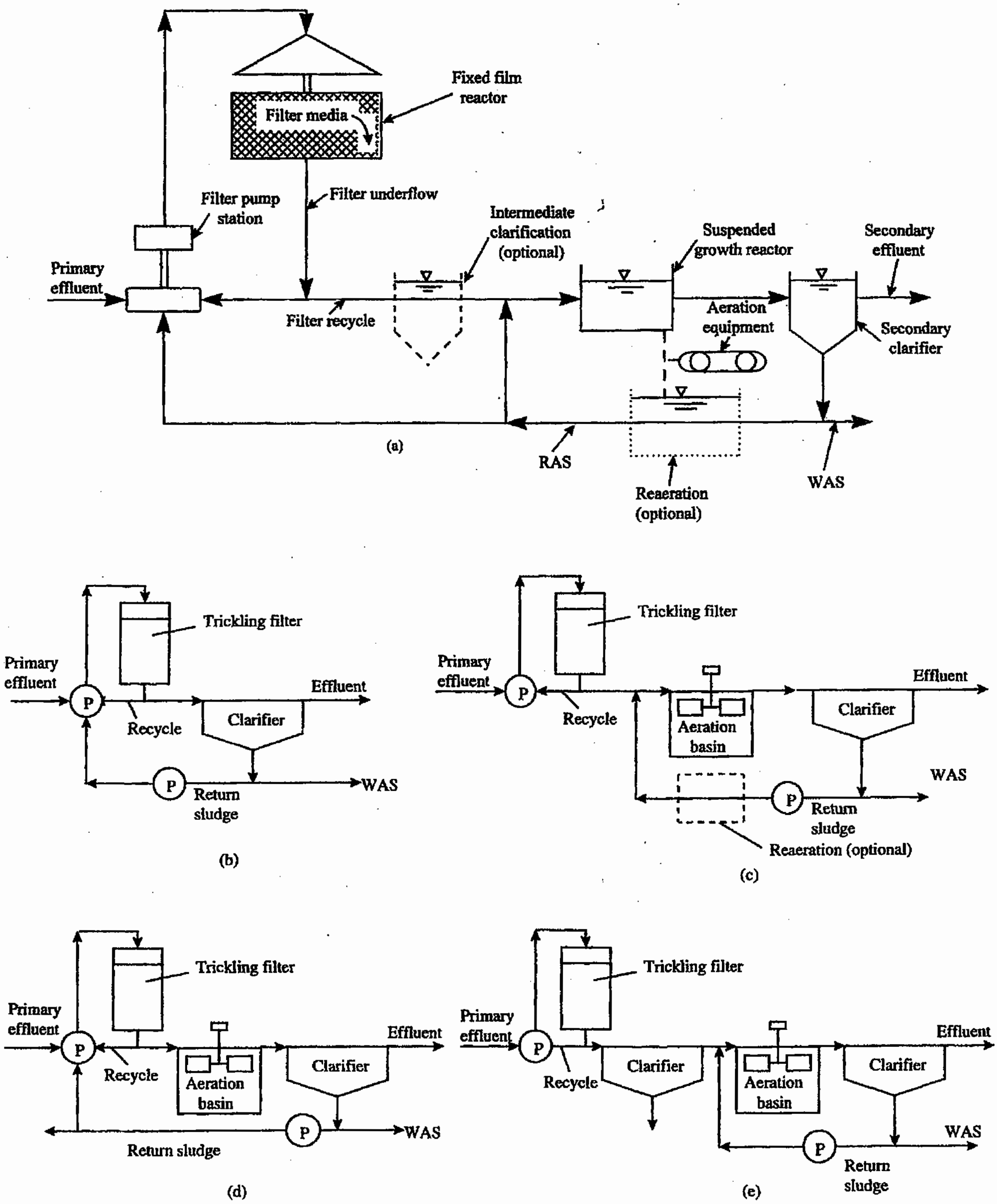


Figure 13-12 Dual Biological Treatment Processes: (a) schematic process diagram with different components; (b) activated biofilter; (c) trickling filter/solids contact; (d) roughing filter, biofilter, or trickling filter/activated sludge; and (e) series trickling filter/activated sludge.

TABLE 13-8 Dual Processes, Description, and Design Criteria

Combined Processes	Description and Design Information	References
Activated biofilter	Fixed film reactor uses high-rate plastic or redwood media. Used for treating medium- to high-strength wastes. BOD loading = 0.2–1.2 kg/m^3·d; hydraulic loading = 2.0–12.2 m^3/m^2·h; MLSS = 1500–3000 mg/L.	2,11,37,41,42, Figure 13-12(b)
Trickling filter/ solids contact	The process includes a fixed film reactor followed by a contact channel 10–15% of the size of an aeration basin of a conventional activated sludge plant. BOD loading = 0.4–1.2 kg/m^2·d; hydraulic loading = 0.2–5.0 m^3/m^2·h; MLSS = 1500 to 3000 mg/L; hydraulic residence time = 0.5–2.0 h (based on influent flow); mean cell residence time = 0.5–2.0 d.	2,11,37,43, Figure 13-12(c)
Roughing filter, biofilter, or trickling filter/activated sludge	The combinations are roughing filter-activated sludge, biofilter-activated sludge, or trickling filter-activated sludge. These combinations are used to upgrade existing activated sludge plants. The roughing filter is 12–20% of high-rate trickling filter. BOD loading = 1.2–3.2 kg/m^3·d; hydraulic loading = 2.0–12.2 m^3/m^2·d; MLSS = 1500–4000 mg/L; hydraulic residence time 2.0–4.0 h; mean cell residence time = 2–8 d; *F/M* = 0.5–1.2 d^{-1}	2,11,37,42–44 Figure 13-12(d)
Series trickling filter and activated sludge	The trickling filter is designed as a roughing filter with high organic loading. The activated sludge receives lower organic loading and provides well-nitrified effluent that is also low in organics.	2,11,42,44, Figure 13-12(e)

13-5-2 Fundamentals of Anaerobic Process

The anaerobic breakdown of organic matter is carried out in an airtight reactor. A multitude of microbial species executes a complex process in a series of interdependent steps: (1) The complex organic compounds (protein, carbohydrates, lipids) are hydrolyzed to simpler organics (amino acids, sugars, peptides). (2) These organics are fermented to volatile acids by *acidogenesis;* the most common acid of anaerobic decomposition is the acetic acid. The group of microorganisms that bring about these conversions

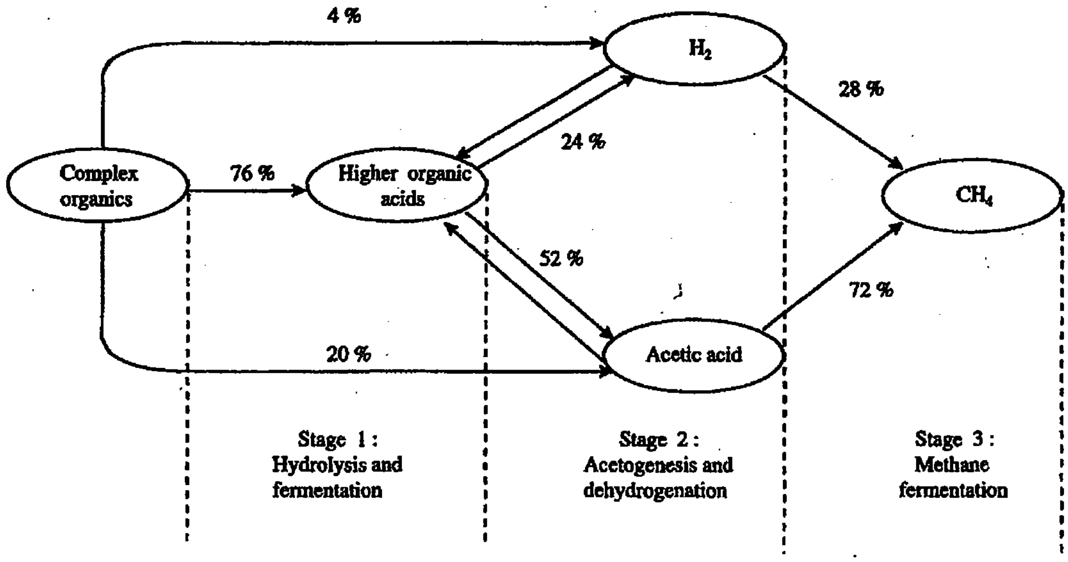


Figure 13-13 Simplified Schematic, Basic Steps and Energy Flow in the Anaerobic Decomposition Process.

is facultative and obligate anaerobic bacteria, collectively called *acidogens* or *acid formers.* Little change occurs in the total amount of organic material although some lowering of pH results. (3) Finally, the third step involves gasification or conversion of acetic acid and hydrogen into methane and carbon dioxide. The microorganisms responsible for this conversion are strict anaerobes and are called methanogens. A simplified schematic of the overall mechanisms of anaerobic decomposition is shown in Figure 13-13.

13-5-3 Process Analysis

The anaerobic process is complex, and many rate-limiting reactions occur. Environmental conditions satisfactory to both acid formers and methane bacteria are essential. The important environmental conditions are listed below:

1. The dissolved oxygen must be zero to maintain strictly anaerobic conditions all the time.
2. Good contact between the microorganisms and the influent must be maintained.
3. The pH of the reactor must range from 6.6–7.8. This is an essential requirement because acid formers tend to lower the pH, while methane formers are sensitive to pH. If pH drops below 6.2, methane formation essentially ceases and more acid accumulates, thus bringing the digestion process to a standstill.
4. The alkalinity of the digester fluid should range from 1000 to 5000 mg/L, and volatile fatty acid should remain below 250 mg/L.
5. The optimum temperature in the mesophilic range should be 30–38°C (85–100°F) and in the thermophilic range should be 49–57°C (120–135°F).
6. Nutrients such as nitrogen and phosphorus should be present in sufficient amounts to ensure proper growth of microorganisms.

7. The anaerobic process has a relatively slow growth rate. The cellular growth is low, resulting in a small quantity of solids production. The typical biological kinetic coefficients for anaerobic digestion are summarized in Table 13-9.

13-5-4 Types of Anaerobic Suspended Growth Reactors

There are three major types of suspended growth, anaerobic treatment processes: (1) anaerobic digestion, (2) anaerobic contact process, and (3) upflow anaerobic sludge blanket (UASB) process. The process description and design information are briefly presented below. The Design Example of an anaerobic digester is provided in Chapter 17.

Anaerobic Digestion. Anaerobic digestion is used for sludge stabilization. The digester contents are heated and mixed. Solids are liquified and digested, and a mixture of methane and carbon dioxide is collected as an energy source. Based on organic and hydraulic loading rates and design features, the anaerobic digesters are divided into standard rate, high rate, and a combination of high rate and standard rate called "two-stage" digesters. In-depth discussion on these digesters and a design of a high-rate digester is presented in Chapter 17.

Anaerobic Contact Process. The anaerobic contact process has an anaerobic reactor and a sedimentation basin. The settled solids are returned to the reactor to maintain a high population of microorganisms. The reactor is airtight, mixed, and heated. The clarifier may have gravity settling or vacuum flotation. The supernatant is discharged or further treated. The flow diagram of an anaerobic contact process is shown in Figure 13-14(a). An anaerobic contact process, like all other anaerobic processes, has low synthesis. As a result, a high population of organisms cannot be maintained in low-strength wastewaters. The influent waste must have a chemical oxygen demand (COD) of 1500

TABLE 13-9 Typical Kinetic Coefficients for the Anaerobic Digestion of Various Substrates[a]

	Coefficient	Basis	Value[b] Range	Typical
Domestic sludge	Y	mg VSS/mg BOD_5	0.040–0.100	0.06
	k_d	d^{-1}	0.020–0.040	0.03
Fatty acid	Y		0.040–0.070	0.050
	k_d	d^{-1}	0.030–0.050	0.040
Carbohydrate	Y		0.020–0.040	0.024
	k_d	d^{-1}	0.025–0.035	0.03
Protein	Y		0.050–0.090	0.075
	k_d	d^{-1}	0.010–0.020	0.014

[a]Derived in part from Refs. 2, 11, 45–47.
[b]Values reported are for 20°C (68°F).

mg/L or higher, generally encountered in food industries (dairy, meat-packing, canning, etc.). The recommended organic loading and hydraulic detention times are 0.5–2.4 kg $COD/m^3 \cdot d$ (0.03–0.15 $lb/ft^3 \cdot d$) and 2–10 h, respectively. The COD removal efficiency is 75–90 percent.[2,47]

Upflow Anaerobic Sludge Blanket Process (UASB). The UASB process was developed in the Netherlands to treat many high-strength industrial wastes.[48] The influent is introduced at the bottom of the reactor. The flow moves upward through a sludge blanket composed of biologically formed granules. Dense granules in the UASB (similar to packing media) allow very high organic loadings (4–12 kg $COD/m^3 \cdot d$). In addition, the biomass concentration in the range of 30,000–80,000 mg/L is achieved. The solid–

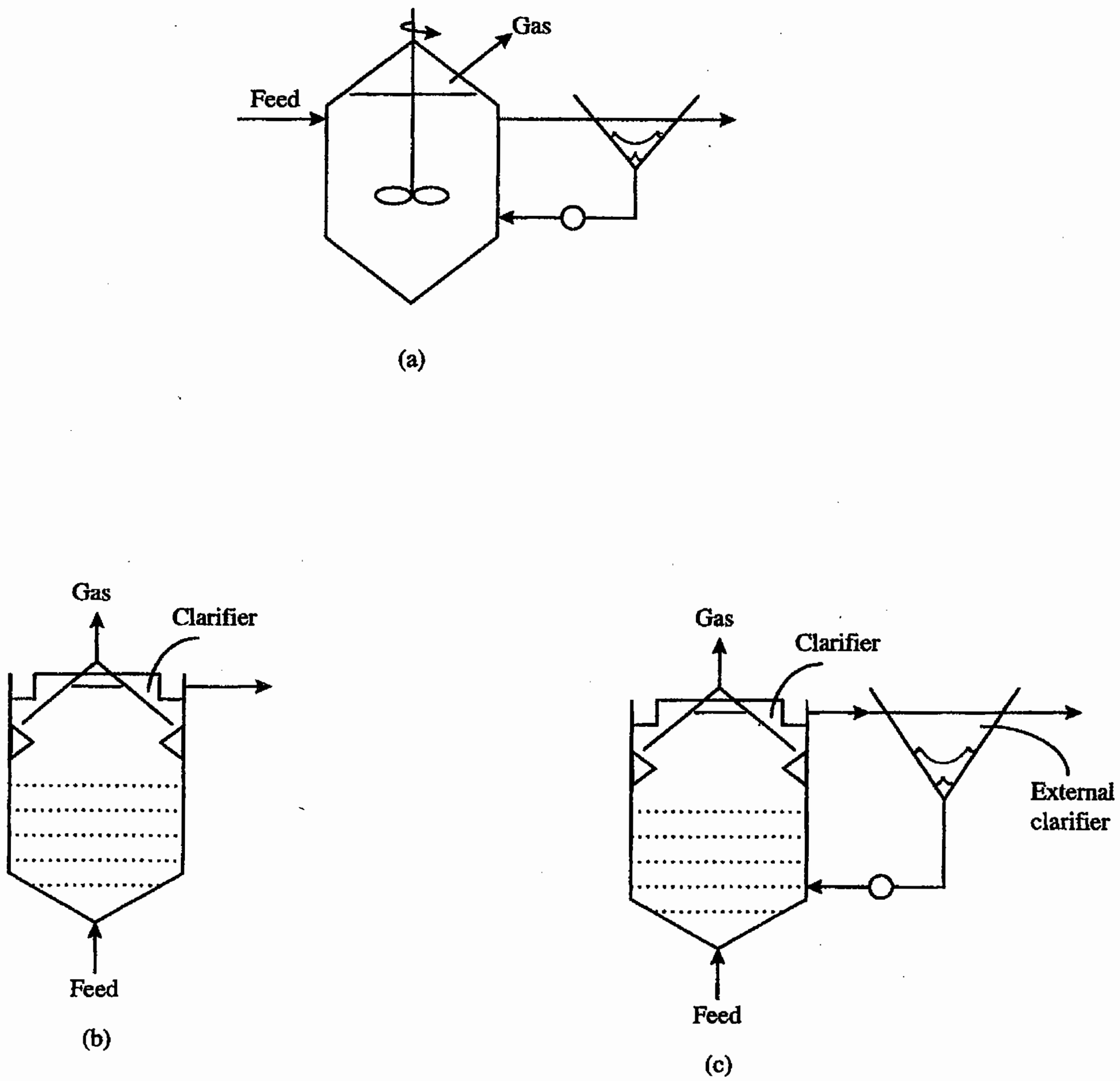


Figure 13-14 Suspended Growth Anaerobic Treatment Processes: (a) anaerobic contact process, (b) upflow anaerobic sludge blanket (UASB) process, and (c) upflow anaerobic sludge blanket (UASB) process with external clarifier.

liquid separation is excellent. The appearance of the granules is dark brown and may be smooth, flocculent, or granular. The hydraulic detention time is 4–12 h. Influent with a COD of 5000–15,000 mg/L may achieve 75–90 percent COD removal. The design features of a UASB system are shown in Figure 13-14(b). The treatment starts as the influent comes in contact with the granules. The gases produced cause internal recirculation and upflow velocity in the range of 0.6–0.9 m/h (2–3 ft/h), which keeps the granules in suspension. The granules attached with gases rise to the top and strike the degassing baffle and drop back to the blanket. The gas is collected. The liquid settles in the sedimentation zone that is provided in the upper portion of the reactor. The solids and granules settle and fall back into the reactor. Often, an external sedimentation basin with sludge return is also provided [Figure 13-14(c)].

Anaerobic Sequencing Batch Reactors (ASBRs). ASBR is a batch variation of the UASB. A single reactor is used for filling, reacting, settling, and decanting.[45] The required volume is larger than that for a continuous flow system, but no separate clarifier and external recycle are required. The volume and the time required for different reaction steps depend on (1) influent characteristics, (2) biomass concentration, (3) effluent quality required, and (4) temperature of the influent. Spreece reported that ASBRs are capable of achieving efficient treatment of relatively dilute wastewaters and at lower temperatures than previously thought possible.[45] The greatest design concern, however, is the entry of air during the decanting step.

13-6 ANAEROBIC, ATTACHED GROWTH BIOLOGICAL TREATMENT

The anaerobic, attached growth biological process utilizes a support medium for microorganisms to attach. Two most common attached growth treatment processes are the anaerobic filter and the expanded bed process. Both processes are described below.

13-6-1 Anaerobic Filters

Anaerobic filters utilize a packed column in which influent flows upward. Biological solids remain attached to the media. A mean-cell residence time as high as 100 days has been achieved. With such a high microbiological population, low-strength wastes at ambient temperature have been effectively treated in a relatively short hydraulic detention time. The reported hydraulic detention time and organic loadings reported for high-strength waste (COD 10,000–20,000 mg/L) are 24–48 h and 1–5 $kg/m^3 \cdot d$, respectively. The COD removal efficiency was 75–85 percent. Other variations of anaerobic filters are downflow packed-bed and series anaerobic filter beds. The anaerobic filters are shown in Figure 13-15(a).

13-6-2 Expanded-Bed Process

The expanded-bed process is an upflow filter in which the media is fluidized. The media may be of sand, expanded aggregate, ion exchange media, diatomaceous earth, plastic, coal, and polyurethane foam on which biofilm develops. A high population of microor-

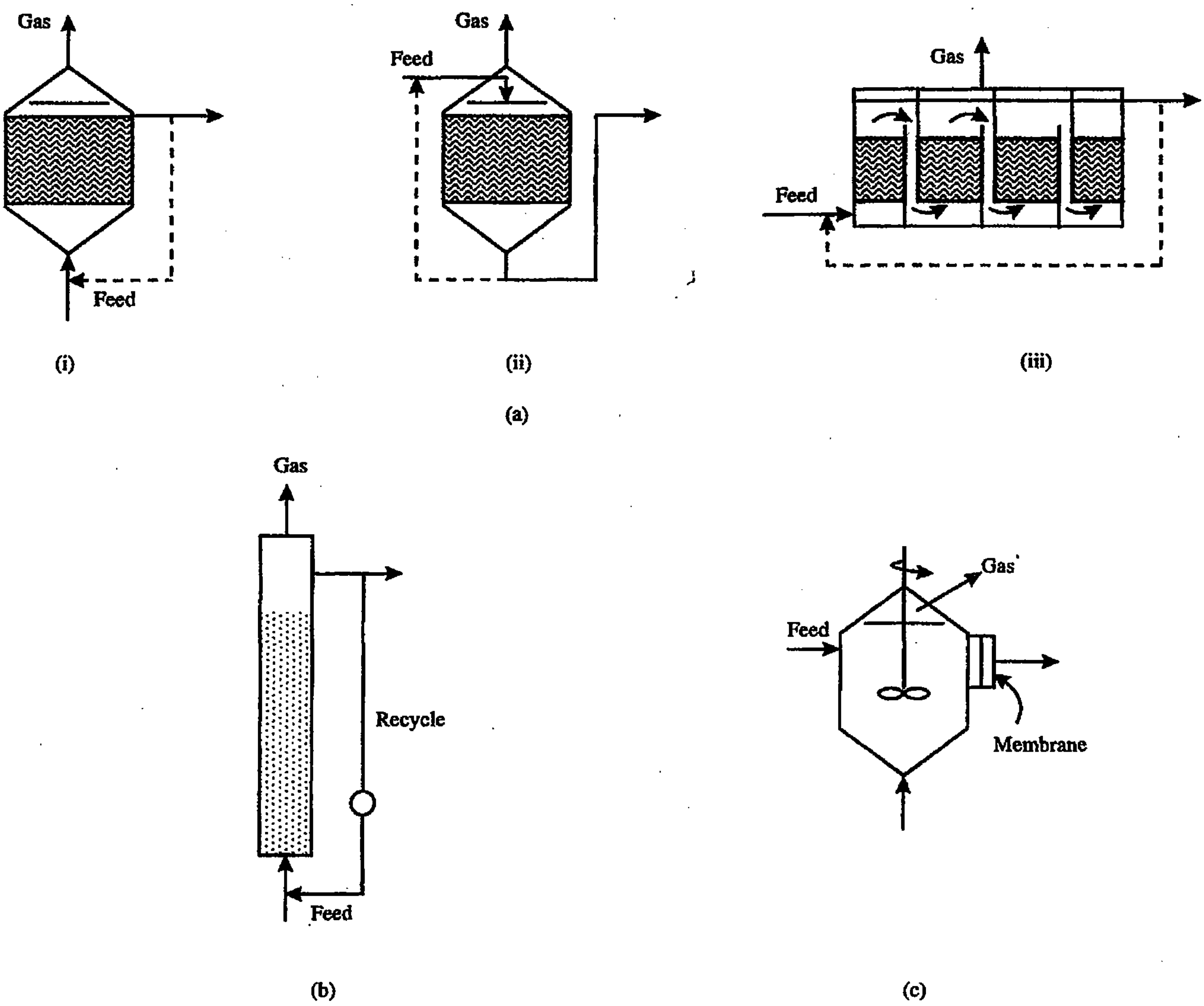


Figure 13-15 Anaerobic Attached Growth Biological Reactors: (a) anaerobic filters: (i) upflow, (ii) downflow, (iii) upflow filters in series; (b) expanded-bed reactor; and (c) anaerobic reactor with membrane solids separation.

ganisms is maintained in the system. As a result, high-strength industrial, as well as municipal, wastewater has been successfully treated. The effluent is recycled to dilute the incoming waste and to maintain bed expansion even under low flow conditions. The hydraulic and solids retention periods are 5–10 h and 0.5–5 days, respectively. The COD loading is 1–10 kg/m^3·d. Reported COD removal efficiency is 80–90 percent. The process configuration of an expanded-bed filter is shown in Figure 13-15(b).

To prevent washout of biosolids from an anaerobic reactor, various types of devices have been used. Among these are internal/external gravity sedimentation, tube settlers, floating plastic media, screen, nylon mesh bags, and ultrafiltration membranes. The solid recovery is essential because anaerobic processes generate a very small quantity of biosolids. Loss of solids in the effluent is sufficient to cause a washout effect, particularly with influent containing a low concentration of organic matter. A membrane solids separation system is shown in Figure 13-15(c).

13-6-3 Anaerobic Rotating Biological Contactor (RBC)

These are biodiscs similar to those discussed in Sec. 13-4-2. To maintain an anaerobic condition, the discs are submerged. The RBC unit may have a fixed cover, and flow may be under slight pressure. The biological growth developed on the surface of the discs, which rotate slowly within the wastewater. The anaerobic condition is maintained within the system. Methane and carbon dioxide produced under anaerobic conditions are collected. The effluent from the RBC unit is settled for clarification. The settled solids are either returned in the RBC unit or wasted.[49,50]

13-7 BIOLOGICAL NUTRIENT REMOVAL (BNR) PROCESSES

Over the last decade there has been an increased emphasis on limiting nutrients in municipal wastewater effluents. Many existing facilities have been upgraded in recent years to include nitrogen and phosphorus removal. Many physical and chemical methods, in conjunction with conventional biological treatment processes, have been utilized to remove these nutrients. In addition to these, biological nutrient removal methods (without any chemical addition) have received great attention. The basic benefits of biological nutrient removal include (1) relatively low cost for removing nitrogen and phosphorus, (2) monetary savings through reduced aeration capacity, (3) less sludge quantity, (4) the obviated expense for chemical treatment, (5) enhanced removal of BOD_5 and TSS, and (6) added process stability and reliability. The integrated physical, chemical, and biological treatment processes are discussed in Chapter 24 for upgrading secondary wastewater treatment facilities. In this section only those biological treatment processes are presented that can *enhance* nutrient removal in conjunction with BOD and TSS removal.[4,11,51,52]

13-7-1 Biological Nitrogen Removal

Both the theoretical and practical aspects of biological nitrogen removal are well understood. Nitrification is the first step, followed by denitrification. Both steps are discussed below.

Nitrification. Nitrification is conversion of ammonia to nitrate. This conversion process generally involves two oxidation reactions: (1) oxidation of ammonia to nitrite and (2) oxidation of nitrite to nitrite. These reactions, biomass synthesis and overall reactions are expressed by Eqs. (13-30)–(13-34). The kinetics of cell growth and substrate utilization, design equations, and design examples of a nitrification process may be found in Refs. 2, 4–6, 10, 11, 13, and 51.
Oxidation of ammonia to nitrite:

$$NH_4^+ + \frac{3}{2} O_2 \xrightarrow{\textit{Nitrosomonas}} NO_2^- + 2H^+ + H_2O \tag{13-30}$$

Oxidation of nitrite to nitrate:

$$NO_2^- + \frac{1}{2} O_2 \xrightarrow{\textit{Nitrobacter}} NO_3 \tag{13-31}$$

Overall ammonia oxidation reaction:

$$NH_4^+ + 2O_2 = \longrightarrow NO_3^- + 2H^+ + H_2O \tag{13-32}$$

Biomass synthesis from ammonia:

$$4CO_2 + HCO_3^- + NH_4^+ + H_2O \longrightarrow C_5H_7O_2N + 5O_2 \tag{13-33}$$

Overall oxidation and biomass synthesis from nitrification:

$$\begin{aligned} NH_4^+ + 1.83O_2 + 1.98HCO_3^- \longrightarrow\ & 0.021C_5H_7O_2N + 0.98NO_3^- \\ & + 1.041H_2O + 1.88H_2CO_3 \end{aligned} \tag{13-34}$$

The basic features of a nitrification process are briefly discussed below:

1. In the nitrification process, the oxidation reactions are carried out by autotrophic organisms. These organisms are generally called *nitrifiers*. The nitrifiers consist of two distinctive genera of organisms: *Nitrosomonas* and *Nitrobacter*. In the first step, the oxidation of ammonia to nitrite occurs and, *Nitrosomonas* is involved. Subsequently, the oxidation of nitrite to nitrate is carried out by *Nitrobacter*. The activity of nitrifiers decreases significantly at lower temperatures, and nitrification is adversely affected.
2. Nitrification can be accomplished in conjunction with a typical carbonaceous BOD removal process using suspended growth, attached growth, or a combination of suspended and attached growth reactors. Additional oxygen, however, is required to oxidize ammonia to nitrate. The stoichiometric oxygen requirement for oxidation of ammonia to nitrate is 4.57 g O_2/g NH_4^+-N.[c] The actual oxygen consumption is lower than the stoichiometric value when synthesis of cellular tissues is considered. The biochemical reactions involving biomass synthesis from ammonia is given by Eq. (13-33).[51] The overall reaction for oxidation and synthesis during nitrification can be developed and expressed by Eq. (13-34). In this overall expression, the actual oxygen requirement is 4.2 g O_2/g NH_4^+-N[d] rather than the stoichiometric value of 4.57 g O_2/g

[c]From Eqs. (13-30) and (13-31), 2 moles of O_2 are theoretically needed per mole of NH_4^+-N or

$$\frac{2 \times 32}{14} = 4.57 \text{ g } O_2/\text{g } NH_4^+\text{-N}$$

[d]From Eq. (13-34), g O_2/g NH_4^+-N $= \dfrac{1.83 \times 32}{14} = 4.2$ g O_2/g NH_4^+-N

$$\text{Overall biomass increase} = \frac{0.021 \times 113 \text{ (mol. wt of } C_5H_7O_2N)}{14 \text{ (atomic wt of N)}} = 0.17 \text{ g/g } NH_4^+\text{-N}$$

NH_4^+-N for only ammonia oxidation. The overall biomass increase is 0.17 g VSS per gram NH_4^+-N oxidized.

3. The concentration of total kjeld nitrogen (TKN) in the influent is significantly lower than that of organic substances, and the cell mass produced due to ammonia oxidation is also smaller than that due to BOD_5 removal. As a result, the fraction of nitrifiers in the total population in the mixed liquor is typically in the range of 2–5 percent of MLVSS.[4] The activity of nitrifiers in a single-stage reactor depends upon the ratio of BOD_5 to TKN in the influent. At a ratio of 5 or higher, the activity of nitrifiers is limited.[5] For a combined carbon oxidation and nitrification process, the overall reactor design therefore is based on either BOD_5 or nitrification removal kinetics, whichever governs. For a separate stage of carbon oxidation and nitrification system, the reactors are designed separately using corresponding reaction kinetics.
4. The concentration of dissolved oxygen (DO) in an aerobic zone may also affect the nitrification process. The growth rate of nitrifiers at a DO level in the range of 0.3–0.5 mg/L may be insufficient, and nitrification may not occur. The nitrification rate increases with an increase in DO in the range of 1.0–3.0 mg/L. A minimum DO level of 2.0 mg/L is recommended for process design.
5. The effect of low pH on nitrification reaction is significant. The optimum pH for nitrification is in the range of 7.2–8.6. Nitrification practically stops at a pH below 6.3. The pH correction on the nitrification rate is discussed in the Design Example (Sec. 13-11-5, Step A, 3). Alkalinity is consumed during the nitrification process [see Eqs. (13-33) and (13-34)]. Approximately 7.1[e] g of alkalinity (as $CaCO_3$) is required per gram of NH_4^+-N oxidized.
6. Lower temperatures have an adverse effect upon nitrification. The temperature correction for nitrification rate constant is therefore applied in the design.
7. Many organic and inorganic compounds at different concentrations and under different environmental conditions may have inhibitory effects upon the activity of nitrifiers. Discussion on this topic may be found in Refs. 51–53.

Denitrification. Denitrification is a conversion of nitrate to nitrogen gas. This conversion process is carried out by microorganisms through a sequence of reduction reactions. Two simplified conceptual reactions, biomass synthesis and overall reactions, are expressed by Eqs. (13-35)–(13-39). These reactions are developed using methanol (CH_3OH) as an organic carbon source. The kinetics of cell growth and substrate utilization, design equations, and design examples of a denitrification process may be found in Refs. 2, 4–6, 10, 11, 13, and 51.

Reduction of nitrate (energy reaction):

$$6NO_3^- + 2CH_3OH \longrightarrow 6NO_2^- + 2CO_2 + 4H_2O \qquad (13\text{-}35)$$

[e]From Eq. (13-34) gram of alkalinity as $CaCO_3$ destroyed per g of NH_4^+-N $= \dfrac{1.98 \times 50 \text{ g } CaCO_3}{14 \text{ g } NH_4^+\text{-N}} = 7.1$

Reduction of nitrite (energy reaction):

$$6NO_2^- + 3CH_3OH \longrightarrow 3N_2 + 3CO_2 + 3H_2O + 6OH^- \tag{13-36}$$

Overall reduction of nitrate (energy reaction):

$$6NO_3^- + 5CH_3OH \longrightarrow 5CO_2 + 3N_2 + 7H_2O + 6OH^- \tag{13-37}$$

Biomass synthesis from denitrification:

$$3NO_3^- + 14CH_3OH + CO_2 + 3H^+ \longrightarrow 3C_5H_7O_2N + 19\,H_2O \tag{13-38}$$

Overall energy reaction and synthesis:

$$NO_3^- + 1.08CH_3OH + H^+ \longrightarrow 0.065C_5H_7O_2N + 0.47N_2 + 0.76CO_2 + 2.44H_2O \tag{13-39}$$

The basic features of a denitrification process are briefly discussed below:

1. Denitrification is an anoxic process that requires (a) the absence of oxygen and (b) the presence of a suitable organic carbon source as electron donor. This process is usually carried out by facultative heterotrophic bacteria termed *denitrifiers.*
2. The organic carbon source is required for the growth of denitrifiers. Based on the organic carbon sources, denitrification can be classified as (a) combined carbon oxidation nitrification/denitrification systems (single-sludge systems) using incoming raw wastewater as the sole organic carbon source and (b) separate-stage denitrification systems (separate-sludge systems) using a suitable external organic carbon source such as methanol. The denitrification reactions can be achieved in either a suspended growth or attached growth reactor. The biochemical reactions for biomass synthesis from nitrate reduction and methanol as the carbon source are expressed by Eq. (13-38). The overall expression for reduction of nitrate and synthesis of cellular tissues is also developed [Eq. (13-39)]. The cell yield from this expression is 0.52 g VSS per g NO_3^--N reduced.[f] The organic carbon source consumption is approximately 3.7 g BOD_L per g NO_3^--N for reduction of nitrate to nitrogen gas.[g] Although it is stoichiometrically developed for a separate-sludge system, this ratio can also be used to estimate the approximate consumption of BOD_5 in the anoxic zone when a single-

[f]Molar weight of $C_5H_7O_2N$ = 113, cell growth $= \dfrac{(0.065 \times 113)}{14} = 0.52$ g VSS/g NO_3^--N

[g]$CH_3OH + 1.5\ O_2 \rightarrow CO_2 + 2H_2O$. Assume methanol is completely biodegradable, 1 mole CH_3OH = 1.5 mole BOD_L.

From Eq. (13-39), the consumption of $BOD_L = \dfrac{(1.08 \times 32)\text{ g } CH_3OH}{14\text{ g } NO_3^-\text{-N}} \times \dfrac{1.5\text{ mole } BOD_L}{\text{mole } CH_3OH} = \dfrac{3.7\text{ g } BOD_L}{\text{g } NO_3^-\text{-N}}$

sludge system is chosen. Additionally, 1.3 g BOD_L is utilized to deoxygenate 1 g DO in the return flows.[4,13]

3. In a BNR system, a large population of heterotrophic organisms would utilize nitrate as the final electron acceptor under an anoxic condition. However, because of a single-sludge system with internal recycle, the heterotrophic population is alternated between anoxic and aerobic environments. As a result, the overall denitrification rate in a single-sludge system would be lower than that in a separate-sludge system. The lower denitrification rate in a single-sludge system may be adjusted by assuming that a smaller population of total heterotrophic organisms actually are able to carry out denitrification. Additional information on this topic is provided in the Design Example (Sec. 13-11-5, Step B, 3).
4. The denitrification rate is very sensitive to the DO level in the anoxic zone. It has been reported that the denitrification process might cease entirely at a DO concentration of 0.1–0.2 mg/L. A maximum DO level of 0.1 mg/L in the anoxic zone, therefore, is generally assumed for the design of the dentrification process.
5. The effect of pH is not significant if denitrification is carried out within the desired range of pH 6.5–8. A portion of alkalinity is recovered by the heterotrophic denitrification reactions. The recovery of alkalinity (as $CaCO_3$) is approximately 3.57 gram per gram of NO_3^--N reduced.[h] This is approximately one-half the amount of alkalinity destroyed by nitrification.
6. Temperature can significantly affect denitrification. The growth rate of denitrifiers decreases with a decrease in temperature. Therefore, proper temperature correction must be made in the design of the anoxic zone.

Combined Nitrification-Denitrification. Combined nitrification-denitrification can be achieved in a single reactor (oxidation ditch) or a series of reactors that create aerobic and anoxic conditions. Raw wastewater is utilized as an external organic carbon source. In a single-sludge system with excess organic carbon source, the denitrification rate is from 75 to 115 g NO_3^--N/kg MLVSS·d. Under limited carbon source situations the denitrification rate is 17–48 g NO_3^--N/kg MLVSS·d. A number of processes have been developed to achieve combined nitrification-denitrification. These processes are briefly summarized in Table 13-10.

13-7-2 Biological Phosphorus Removal (BPR)

Phosphorus Release and Uptake. Phosphorus in wastewater exists in organic and inorganic forms. Orthophosphate (PO_4^{-3}) and polyphosphate (P_2O_7) constitute the major inorganic component and account for 70 percent of total phosphorus. In a secondary wastewater treatment plant phosphorus is used up for cell synthesis and energy transport. For this reason, phosphorus it taken up in an amount related to the stoichiometric requirement for biosynthesis. A typical phosphorus content of a microbiological cell is

[h]From Eq. (13-38), destruction of acidity or recovery of alkalinity = $\dfrac{1\ \text{g}\ H^+}{14\ \text{g}\ NO_3^-\text{-N}} \times \dfrac{50\ \text{g}\ CaCO_3}{1\ \text{g}\ H^+} =$ 3.57 $CaCO_3$/g NO_3^--N.

TABLE 13-10 Combined Nitrification-Denitrification Processes

Process	Description	Ref.
Oxidation ditch	Anoxic and aerobic condition is created in zones upstream and downstream of the rotor. Orbal, Carrousel, and Bio-denitro are the proprietary processes.	11,51,54
Bardenpho process	The proprietary process has four stages: anoxic, aerobic, anoxic, and aerobic zones.	2,11,51,55
Wuhrmanr process	The process uses an aerobic-anoxic reactor followed by a clarifier. Returned sludge and influent are added at the head of the anoxic basin.	11,56
Ludzack-Ettinger process	The process has an anoxic-aerobic process sequence followed by clarification and return sludge. Modification to this process uses internal recycle.	55,57
Sequencing batch reactor	Nitrification and denitrification occurs during aeration and filling and mixing cycles of a single basin.	58,59
Alternating aeration systems	Dual-sludge and triple-sludge systems are used for nitrification and denitrification.	11,60,61
Attached growth processes	Nitrification-denitrification is achieved in aerobic and anoxic, attached growth reactors. These are nitrification filter and RBCs, denitrification filters, fluidized-bed denitrification filters, and submerged RBCs.	11,62,63,64

1.5–2 percent on a dry weight basis (VSS), and typical phosphorus removal in a secondary wastewater treatment plant is 10–30 percent of influent phosphorus.[4,11]

A sequence of an anaerobic zone, followed by an aerobic zone, encourages growth of microorganisms that are capable of uptaking phosphorus beyond the stoichiometric requirement for growth. As a result, phosphorus levels of 4–12 percent of microbiological solids (VSS) may result, and phosphorus removal up to 70–95 percent of influent concentration may take place. The primary organism associated with enhanced phosphorus removal belongs to the genus *Acinetobacter.* Under anaerobic stress the biomass releases inorganic orthophosphate followed by extra phosphorus uptake under aerobic environment (Figure 13-16). The extent of phosphate release in the anaerobic or anoxic zone and subsequent uptake in the aerobic zone are related to the type and quantity of biodegradable organic carbon available. Research has indicated that for each milligram per liter of phosphorus removed in the total system, approximately 17, 24, and 28 mg/L of acetic, propionic, or butyric acid as COD are utilized, respectively.[51] The overall COD consumption for municipal wastewater is approximately 20–50 mg/L for each milligram per liter of phosphorus removed.[65-68] Other studies have shown that biological phosphorus removal is influenced by the availability of biodegradable carbon in a high enough concentration in the mixture of influent and return sludge flow. More than 90 percent to-

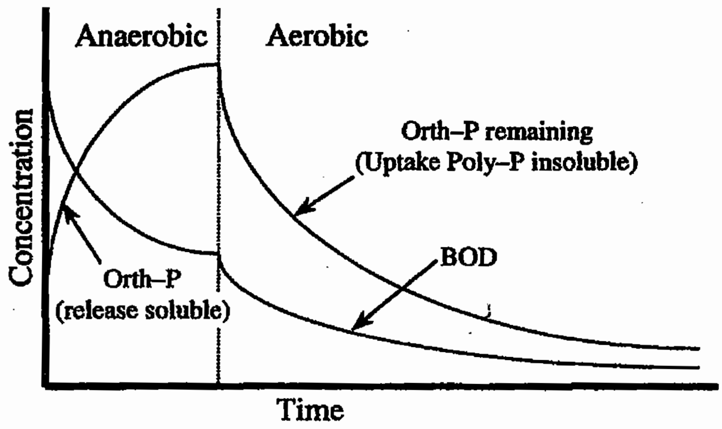


Figure 13-16 Phosphorus Release and Uptake under Anaerobic and Aerobic Conditions and BOD Utilization (from Ref. 75).

tal phosphorus removal can be achieved in the system when the soluble BOD_5 of the mixture is more than 120–125 mg/L.[69]

Design Considerations. The kinetics of biological phosphorus release and uptake are not yet fully developed. The design engineers have used empirical observations, bench-scale, or pilot plant data to design new or modify existing plants to achieve biological nutrient removal. The empirical design method suggested by Ekama and Marais and others is used extensively for designing the biological phosphorus removal (BPR) facility. A computer model developed by the International Association on Water Quality (IAWQ), formerly International Association on Water Pollution Research and Control (IAWPRC), is also used for design of BPR facilities.[51,52,70,71]

The basic principles and design considerations of BPR is simple and can be utilized in existing activated sludge plants. Some of the basic principles are listed below:

1. An anaerobic zone is placed ahead of an aerobic zone in a single-stage activated sludge system.
2. The return sludge or biomass is recycled through the anaerobic zone and mixed with the influent.
3. Sufficient biodegradable organics must be present in the anaerobic zone to ensure a true anaerobic condition. Organic matter will be utilized for (a) deoxygenation of DO contained in the return sludge, (b) deoxygenation of DO entering from the atmosphere, (c) satisfying the carbon source for denitrification of NO_3^--N in the return sludge, and (d) providing intercellular carbon storage for phosphorus release in the anaerobic zone. This carbon will subsequently be utilized in the aerobic zone during phosphorus uptake. Denitrification of return sludge may often be needed before recycling into the anaerobic zone.
4. Soluble *ortho*-P concentration after release in anaerobic zone may reach 20–40 mg/L.
5. At an average, 20–50 mg/L COD may be consumed per milligram per liter of phosphorus removed from the municipal wastewater.

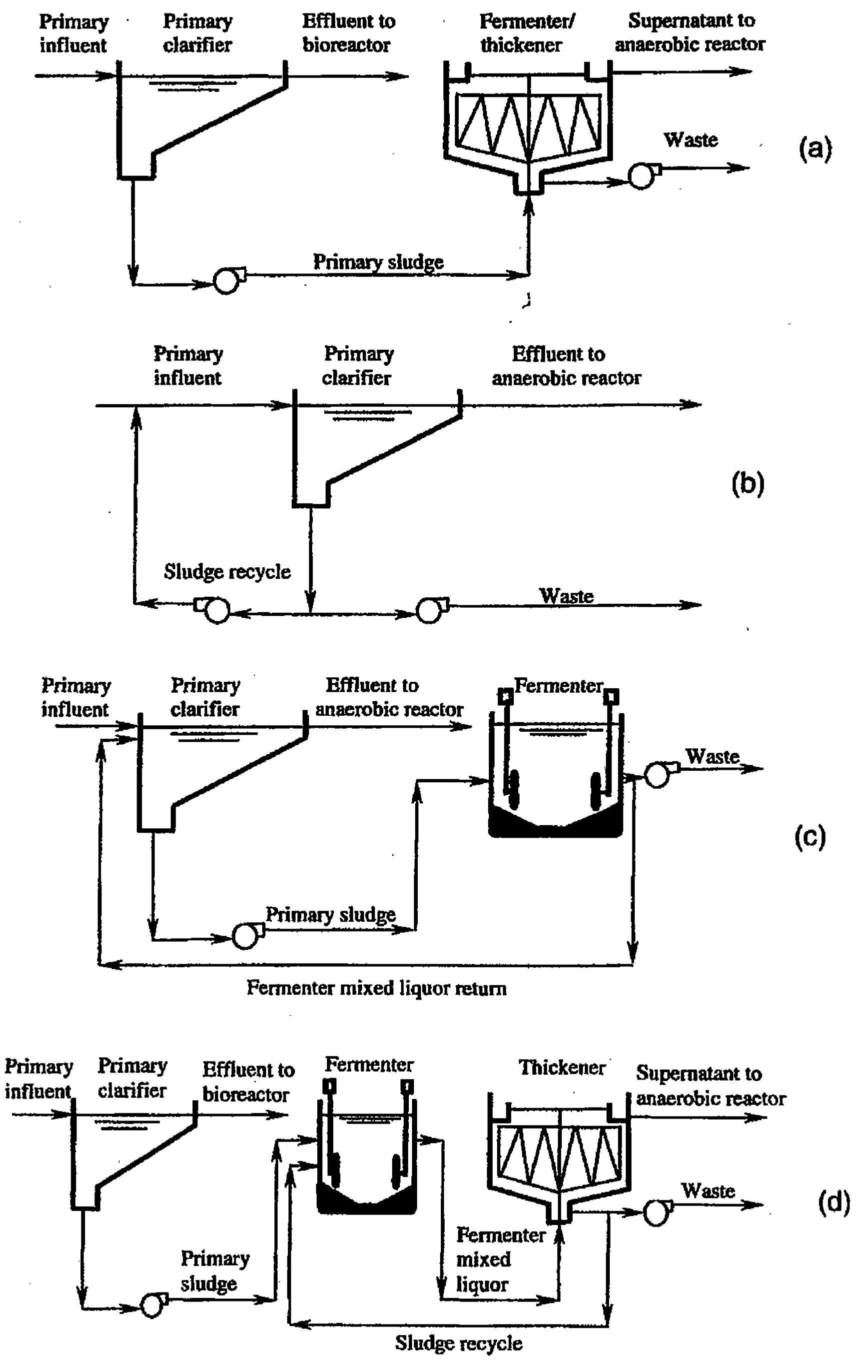


Figure 13-17 Primary and Thickener Fermenters: (a) static primary, (b) activated primary, (c) complete-mix primary, and (d) complete-mix fermenter thickener (from Ref. 75).

6. It is essential that the available organic compounds in the influent have undergone sufficient fermentation to generate the short-chain volatile fatty acids (SCVFAs) needed for poly-P bacteria to form stored organics such as poly-β-hydroxybutyrate (PHB) inside the cell. As a general guide, a COD to BOD_5 ratio below 2 is an indication of sufficient fermentation. Higher than a ratio of 2 may indicate insufficient fermentation, and a larger anaerobic zone may be necessary.[51]
7. If the influent biodegradable COD and total phosphorus (TP) ratio is considerably higher than 40, excellent phosphorus removal can be achieved in an anaerobic zone

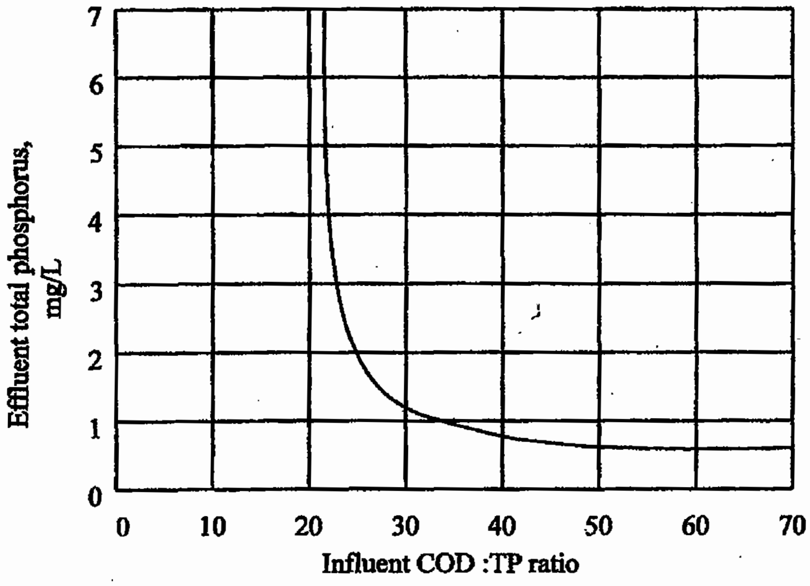


Figure 13-18 The Effect of TCOD: TP Ratio on the Effluent Soluble Phosphorus (from Ref. 51).

having hydraulic retention time (HRT) below 1.5 h although 1.5 h is considered the minimum for municipal wastewater treatment plants.[51] It may be noted that if sufficient SCVFAs are in the influent, then the anaerobic zone should only be large enough to store SCVFAs as PHB or similar polymers. This step is rapid, and required HRT for anaerobic zone is significantly less than 1 hour. At a ratio of COD : TP below 20, the wastewater is expected to have insufficient fermentation. A substantial increase in the anaerobic zone and/or an additional fermentation unit may be needed to accomplish the desired level of phosphorus removal. Often, on-site production of SCVFAs through primary sludge or thickener fermentation is used to increase its amount. Four principal fermenter configurations are utilized: static primary, activated primary, complete-mix fermenter for primary, and complete-mix fermenter for thickener. A schematic flow diagram for four fermenters is given in Figure 13-17. Readers may consult Refs.72–75 for process description and performance information on fermenters. Phosphorus released from the thickener fermenter is a concern and is briefly presented in item 13 of this section. Randall et al. have given other fermenter configurations and design considerations.[51]

A relationship between effluent total phosphorus and the ratio of COD and TP in the influent is given in Figure 13-18. It may be clearly noted that the effluent total phosphorus below 1 mg/L is achieved at a COD/TP ratio of 33 or higher. An equally important relationship between phosphorus removal (influent total P − effluent soluble P) mg/L versus HRT of anaerobic zone in hours is given in Figure 13-19. This linear relation is expressed by Eq. (13-40):

$$HRT = 0.81\,(TP_{inf} - SP_{eff}) - 2.11 \qquad (13\text{-}40)$$

where

HRT = hydraulic retention time of anaerobic zone, h
TP_{inf} = total phosphorus in influent, mg/L
SP_{eff} = soluble phosphorus in effluent, mg/L

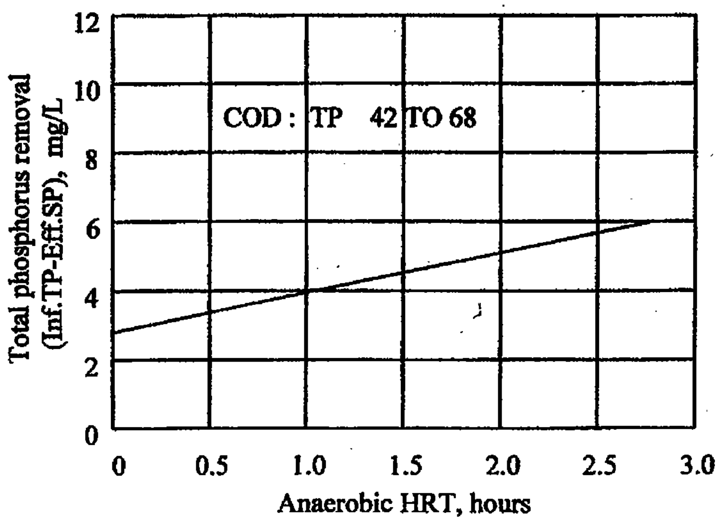


Figure 13-19 The Effect of HRT of Anaerobic Zone on the Phosphorus Removal in BPR Process (from Ref. 51).

8. The HRT of the anaerobic zone is often expressed as a fraction of total HRT in anaerobic, anoxic, and aerobic zones. Under normal conditions of COD/TP and COD/BOD_5 ratios, this fraction is 0.1 to 0.2 for needed fermentation and phosphorus release and subsequent uptake in the aerobic zone.
9. Both magnesium and potassium cations are essential for the BPR process. The minimum requirements are, respectively, 0.25 and 0.23 moles of magnesium and potassium for each mole of phosphorus removed. In municipal wastewater the concentrations of these cations are significantly higher than minimum requirements.
10. The optimum pH range for a BPR process is between 6.6 and 7.4. Lower pH may be encountered if the treatment facility is nitrifying without denitrification while treating low alkalinity wastewater. The BPR activity steadily declines below a pH of 6.5 and practically stops below pH 5.2. High nitrate concentration in the return sludge may interfere with phosphorus release.
11. Each mixer in the unaerated zone must provide sufficient mixing to keep the contents in suspension. Submersible or submerged turbine mixers are commonly used. Surface turbulence should be minimized. The minimum power requirement may be calculated from Eq. (13-41):

$$\frac{P}{V} = 0.00094\,(\mu)^{0.3}\,(\mathrm{MLSS})^{0.298} \tag{13-41}$$

where

MLSS = mixed liquor suspended solids, mg/L

μ = 1.0087 at 20°C

$\frac{P}{V}$ = power requirement for mixing, kW/m^3

12. Sufficient DO in the aerobic reactor is necessary for poly-P bacteria to completely metabolize the stored PHB under aerobic conditions. This provides energy for the

uptake of all available orthophosphate. Additionally, there should be sufficient DO in the sludge zone to prevent phosphorus release in the secondary clarifier. A DO level of 2.0–3.0 mg/L in the effluent from the aerobic reactor is considered satisfactory.

13. The sidestreams from sludge processing areas are generally returned before or after primary treatment facility. The sidestreams often contain high concentrations of nitrogen and phosphorus. As a result the combined flow reaching the biological waste treatment facility may often have a significantly lower TCOD:TP ratio. The waste-activated sludge from a BPR system, being rich in phosphorus (5–8%), may release *ortho*-P in the gravity thickener and anaerobic digester causing a buildup of phosphorus in the system. Naturally, the BPR system will not be able to produce effluent less than 1 mg/L total P at a low TCOD:TP ratio. Readers may note that in the Design Example (Sec. 13-11-5, Step D) a phosphorus stripper tank is provided to precipitate released phosphorus and chemically immobilize it before the combined sludge is thickened in the gravity thickener. This way, a buildup of phosphorus through return of sidestreams will not occur.

13-7-3 Combined Biological Nitrogen and Phosphorus Removal

Experience has shown that biological nutrient removal processes are relatively low-cost and are a reliable means of removing nitrogen and phosphorus along with BOD and TSS. As a result, a large number of wastewater treatment plants in North America and in many other countries have been constructed to achieve biological nutrient removal. Many of these are proprietary systems that use different combinations of anaerobic, anoxic and aerobic zones, and recirculation patterns. The overall process diagram may include three, four, or five zones and several recirculation patterns. The selection of reaction zones and recirculation will depend upon waste characteristics and effluent quantity required. In this section a discussion on capabilities of BNR systems, common proprietary systems available, and basic design factors are presented.

Capabilities of BNR Systems. The BNR systems are capable of removing significant amounts of total phosphorus, total nitrogen, BOD_5, and TSS. The degree of removal depends on the characteristics of wastewater, operating temperature, and many design and operational features of the facility. Under optimum conditions the BNR system with multiple zone is capable of producing effluent with total BOD_5, TSS, phosphorus, and ammonia concentrations below 5, 5, 0.5, and 0.1 mg/L, respectively. However, effluent from the BNR process will typically contain a soluble organic nitrogen concentration of 0.5–1.0 mg/L and an NO_3^--N concentration in the range of 5–8 mg/L. Further removal of organic nitrogen would require activated carbon treatment or other suitable methods.[51]

Nitrate plus nitrite[i] nitrogen concentration below 3 mg/L is technically possible, but high recycle and large denitrification unit would be required. A more economically achievable concentration of nitrate nitrogen in the effluent is around 5 mg/L. Thus, an

[i]Nitrite concentration in effluent is generally small.

achievable limit of total nitrogen, considering seasonal temperature variations and changes in the waste characteristics, is around 7 mg/L.

Several propriety BNR systems are currently available that achieve biological nitrogen and phosphorus removal. Many of these systems have been tested on full scale, and a considerable amount of design and operational information is available. The most common processes for combined nitrogen and phosphorus removal are (1) Bardenpho process, (2) A^2/O process, (3) UCT process, (4) VIP process, (5) sequencing batch reactor (SBR), (6) Phostrip process, (7) Orbal process, and (8) anoxic/anaerobic/aerobic process.

Bardenpho Process. The proprietary Bardenpho process for nitrogen removal was modified (called *Phoredox modification*) for combined nitrogen and phosphorus removal. The process has five stages of anaerobic, anoxic, and aerobic zones with sludge and internal recycling.[76] The process diagram is shown in Figure 13-20(a), and design features are summarized in Table 13-11.

A^2/O Process. The modification of the A/O process for nitrogen and phosphorus removal is the A^2/O process. The process diagram utilizes anaerobic, anoxic, and aerobic sequence with sludge and internal recycle.[77] The process has stability and produces effluent of high quality. The process diagram and design factors are given in Figure 13-20(b) and Table 13-11.

UCT Process. The modified University of Cape Town (UCT) process for nitrogen and phosphorus removal uses anaerobic, anoxic, and aerobic reactor sequence. Both the return activated sludge and the aeration tank contents are recycled to the anoxic zone, and the contents of the anoxic zone are then recycled to the anaerobic zone.[66] The process diagram and design information are provided in Figure 13-20(c) and Table 13-11.

VIP Process. The VIP (named for the Virginia Initiative Plant in Norfolk, Virginia) is similar to the A^2/O and UCT processes except for the method of sludge recycle.[78] The process diagram and design information is provided in Figure 13-20(d) and Table 13-11. A staged reactor configuration is provided by using at least two complete mix cells in series for each zone of the biological reactor.

Sequencing Batch Reactor (SBR). Enhanced nitrogen and phosphorus removal can be achieved in a sequencing batch reactor. Phosphorus release and some BOD uptake takes place during fill and anaerobic stir operation. Phosphorus uptake, BOD oxidation, and nitrification occurs under the aerobic cycle. Denitrification is achieved during anoxic stir and settling cycles. The cycle phases are shown in Figure 13-5(e). The cycle time is given in Table 13-11.[4-11]

PhoStrip Process. The proprietary PhoStrip process has a stripping tank in which a portion of the return sludge is diverted. The return sludge may be elutriated with primary effluent to give the necessary carbanaceous source for phosphorus release. Under anaerobic condition nitrogen is removed by denitrification, and phosphorus is released into the liquid. The biological solids are separated and returned to the process. The phosphorus-

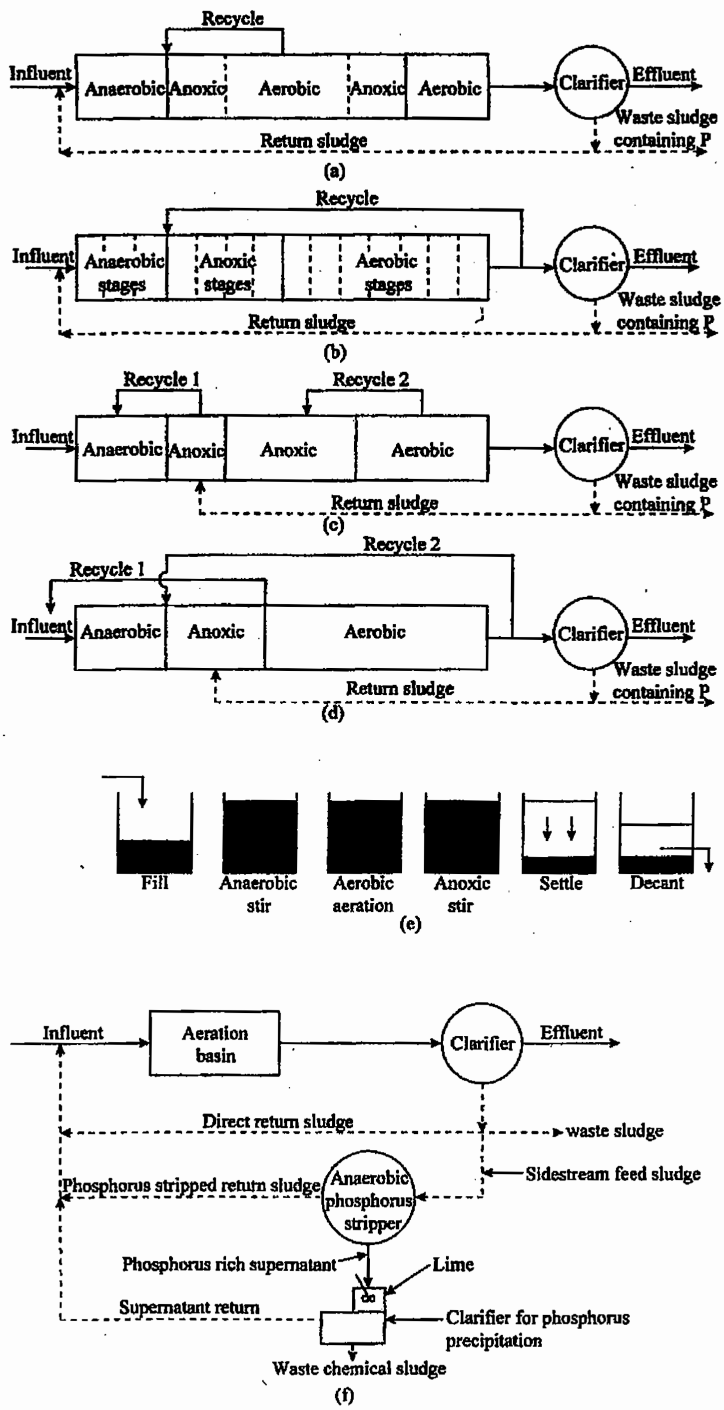


Figure 13-20 Combined Biological Nitrogen and Phosphorus Removal Processes: (a) modified Bardenpho; (b) A^2/O; (c) UCT; (d) VIP; (e) sequencing batch reactor; (f) Pho-Strip.

rich supernatant is coagulated to precipitate phosphorus.[79,80] The process diagram and design information are provided in Figure 13-20(f) and Table 13-11.

Orbal Process. The Orbal process typically consists of three concentric channels operating in series where the outside channel is maintained in an oxygen deficit condition, but a sizeable amount of oxygen is delivered, allowing simultaneous nitrification–

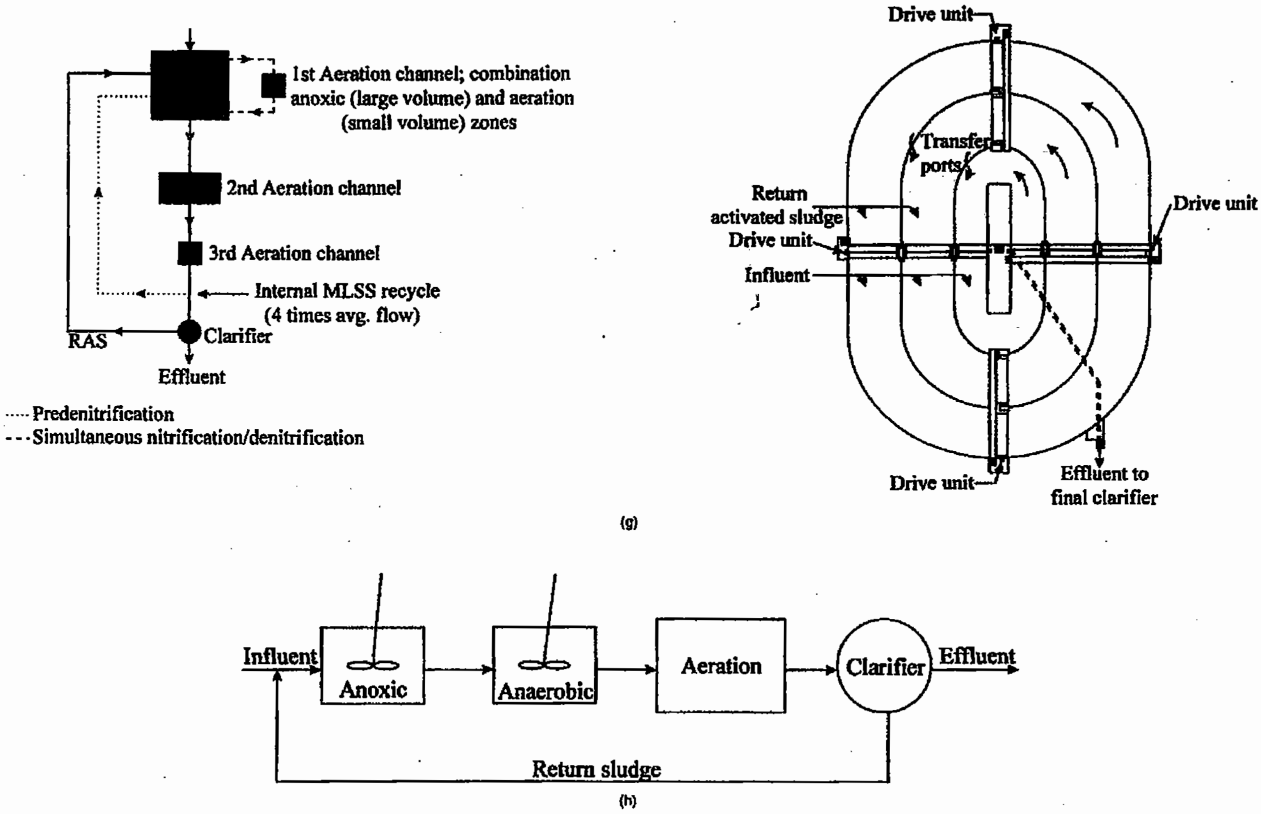


Figure 13-20—cont'd (g) Orbal (courtesy Envirex, U.S. Filter): (i) schematic, (ii) flow diagram; (h) anoxic/anaerobic/aerobic system.

denitrification. Orbals are designed for 80 percent total nitrogen removal; higher removal rates (95 percent and more) are accomplished by recycling MLSS from the third channel back to the first.[81-83] For biological phosphorus removal, a strong oxygen deficit condition is maintained in the first channel, and a minor deficit condition is kept in the second channel. The process configuration is shown in Figure 13-20(g). The design features are summarized in Table 13-11.

Anoxic/Anaerobic/Aerobic Process. The University of Texas at Arlington conducted bench-scale and pilot plant studies to enhance nutrient removal from an existing activated sludge plant. The requirements were to evaluate a simplified treatment regime that would require a minimum unit process addition and use a single recycle line common to most activated sludge designs. An anoxic/anaerobic/aerobic reactor sequence with recycle from the settling basin to anoxic basin was used. The process train and design factors are provided in Figure 13-20(h) and Table 13-11. The results of the bench-scale and pilot studies are reported in Refs. 69 and 84–86.

The results of these studies clearly show that significant amounts of total nitrogen (TN) and total phosphorus (TP) removal can be achieved in a BNR facility. Higher return sludge would increase nitrate recycle and enhanced nitrogen removal due to denitrification in the anoxic reactor. The total phosphorus release in the anoxic and anaerobic reactor, and subsequent uptake in the aerobic reactor, is greatly influenced by the availability of biodegradable carbon in high enough concentrations in the mixture of return

TABLE 13-11 Process Design Parameters for Biological Nutrient Removal Systems

Design Parameters	Unit	Process: Modified Bardenpho	A^2/O	UCT	VIP	SBR	PhoStrip	Orbal	Anoxic/ Anaerobic/ Aerobic
F/M ratio	kg BOD_5/ kg MLVSS·d	0.1–0.2	0.15–0.25	0.1–0.2	0.1–0.2	0.1–0.2	0.1–0.5	0.1–0.2	0.1–0.2
SRT	d	10–40	4–27	10–30	5–10	—	0.3–0.8	8–20	10–15
MLSS	mg/L	2000–5000	2000–4000	2000–5000	1500–3000	600–5000	2000–4000	2000–6000	3000–4000
HRT	h								
Anaerobic		1–2	0.5–1.5	1–2	1–2	0.0–3.0	8–12	3–6	0.7–1.2
Anoxic		2–4	0.5–1.0	2–4	1–2	0.0–1.6	—	4–8	1.0–3.0
Aerobic		4–12	3.5–6.0	4–12	2.5–4	0.5–1.0	4–10	6–12	4.0–6.0
Anoxic		2–4	—	2–4		0.0–0.3	—		—
Aerobic		0.5–1.0	—	—		0.0–0.3	—		—
Settle/Decant			—	—		1.5–2.0	—		—
Total		9.5–23.0	4.5–8.5	9–22	4.5–8	4.0–9.0	12–22.0	13–26	5.7–10.2
RAS Q_r	% of Q_o	80–100	20–50	80–100	50–100	—	20–50	75–100	20–50
Internal recycle	% of Q_o	400–600	100–300	100–600	200–400	—	—	0–400	0
Aerobic recycle				50–100	—	—	—	—	—
Effluent TP	mg/L	<2	<2	<2	<2	<2	<0.5	<5	<2
Effluent TN	mg/L	<6	<6	<6	<6	<6	<8	<5	<8

Note: All processes are capable of achieving soluble BOD_5 and TSS below 5 mg/L.

sludge and the influent. An effluent quality of 5/5/8/0.5/1 (BOD_5/TSS/NH_4^+-N/TN/TP) can be consistently met from primary settled effluent and at less than 50 percent return sludge and 100 percent MLSS recycle.

Design Considerations for BNR Systems. The purpose of a BNR system is to maximize the removal of organics, nitrogen, and phosphorus. Organics removal occurs because of oxidation and biomass synthesis. Nitrogen removal takes place by nitrification–denitrification and conversion into cellular mass. Phosphorus removal occurs by release and then uptake. In all steps organic matter is utilized for synthesis and as an energy source. The design and operation of BNR systems therefore require thorough understanding of influent characteristics, environmental factors, and design parameters such as SRT, number of reactors and configuration, and recirculation pattern. Many of these factors have been presented in detail in earlier sections. The following basic design steps are useful for understanding the design procedure:

1. Determine the influent waste characteristics reaching the BNR facility. A material mass balance analysis is required to develop such information. Give special consideration to mass balance since phosphorus may be released in the sludge processing areas and sidestreams may cause excessive buildup of phosphorus in the system.
2. Determine the ratio of TCOD:TP and expected SCVFAs. The desirable ranges of these parameter were presented previously. If the SCVFAs are insufficient, a fermentation facility may be incorporated in the process train.
3. Develop the kinetic coefficients for carbonaceous BOD removal, nitrification, and denitrification. These coefficients for municipal wastewater are well known. If the wastewater source is not a typical municipal system, the design engineer should develop these coefficients based on experience from a similar source or conduct bench-scale or pilot testing. The range and typical values of the kinetic coefficients for designing reactors for BOD removal, nitrification, and denitrification are provided in the Design Example.
4. Determine the overall reactor's size, which will include (a) anaerobic zone, (b) one or more anoxic zones, and (c) one or more aerobic (oxic) zones. Using kinetic equations and winter temperature conditions, determine HRT (θ) and SRT (θ_c) for different stages. As a rule of thumb, the anaerobic zone will utilize approximately 10–20 percent of the total volume, depending on the TCOD:TP ratio and availability of SCVFAs. A larger proportion would be needed if a fermentation step is necessary in the anaerobic zone. The total anoxic zone will occupy 10–30 percent of the total volume. The proportion of anoxic zone will be dependent upon the recycle ratio and the extent of denitrification desired for total nitrogen removal. The remaining volume in the aerobic zone will achieve stabilization of leftover carbonaceous BOD and nitrification.
5. Make a material balance of biodegradable organic matter through the entire process train. Approximately 1.0 mg/L acetate COD will be taken up by the *Acinetobacter* for each milligram per liter of phosphorus released. Generally, 20–40 mg/L of inorganic phosphorus concentration can be reached in the anaerobic zone. Another

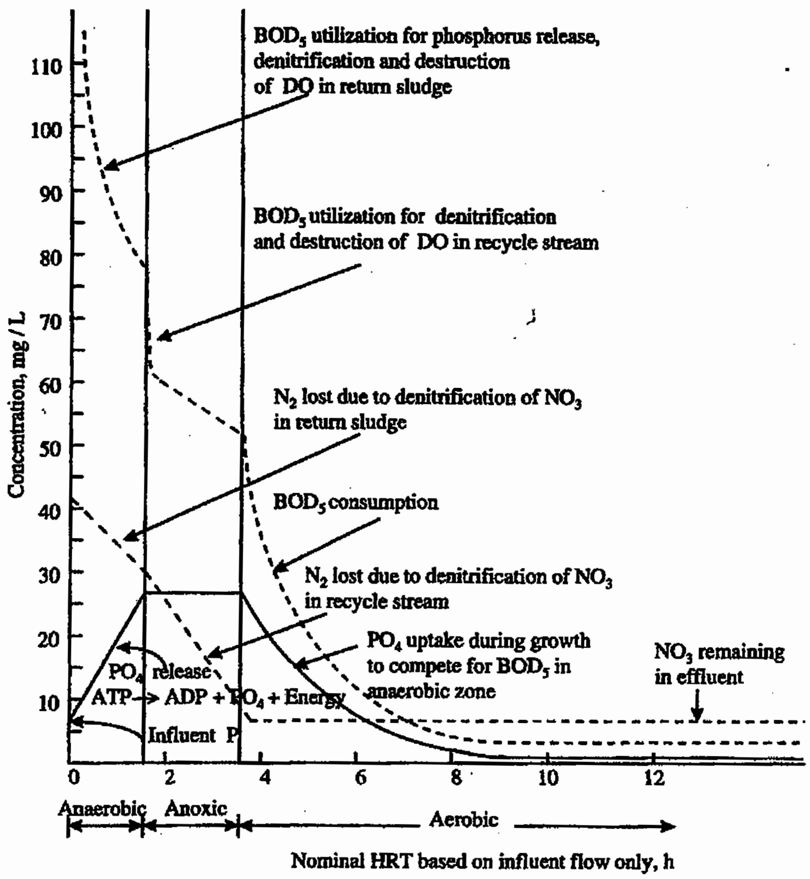


Figure 13-21 Typical Concentration Profile of Biodegradable Organic Matter, Total Phosphorus, and Nitrogen in Effluent (not considering dilution effect) in a Three-Stage BNR System.

method to estimate the uptake of biodegradable COD is six times the influent phosphorus concentration. Overall COD uptake for phosphorus release in municipal wastewater is approximately 50 mg/L. It may be noted that this uptake of organic matter is a temporary storage that is finally stabilized in the aerobic zone. The biodegradable COD is also consumed for denitrification. This will happen in the anaerobic zone if sludge carrying nitrate is returned in the anaerobic zone. Anoxic zones provided for denitrification would also consume biodegradable carbon. A good estimation is 3–4 mg/L COD per mg/L NO_3^--N and 1.3 mg/L COD per mg/L of DO. An additional 5–10 percent allowance may be given for destruction of oxygen that may dissolve because of surface turbulence. The phosphorus, total nitrogen, and BOD$_5$ profile in the three-stage BNR system is shown in Figure 13-21.

6. Calculate the recycle ratios for denitrification in anoxic zones using total nitrogen balance through the system. Total NO_3^--N destroyed by denitrification is equal to TKN in influent minus the nitrogen fixed in the cell growth, and NH_4^+-N, soluble org.-N and NO_3^--N lost in the effluent.
7. Check the alkalinity of the system. There will be destruction of alkalinity at a rate of 7.1 mg/L as $CaCO_3$ per mg/L of NH_4^+-N nitrified. Denitrification will recover alkalinity at a rate of 3.6 mg/L alkalinity as $CaCO_3$ per milligram per liter of NO_3^--N denitrified.

8. Check environmental factors and optimize the design by reducing their effects. Anaerobic zone must not have oxygen and nitrate. The oxidation-reduction potential (ORP) should be well below zero. Entry of oxygen from surface aeration should be minimized. Carryover of NO_3^- from anoxic zone through the perforated baffle may adversely affect the performance of the anaerobic zone. On-line monitoring of ORP is useful to determine if anaerobic conditions are being maintained. The maximum specific growth rate of *Acinetobacter* is high at higher pH. Phosphorus uptake in an aerobic zone decreases significantly at pH below 6.0.
9. Denitrifiers require an anoxic environment (NO_3^- and NO_2^- present, DO absent). Extreme care should be taken to minimize the potential for the entry of DO. High concentrations of SCVFAs enhance denitrification.
10. Nitrifiers on the other hand require aerobic conditions. A minimum DO of 2 mg/L is needed for high activity of nitrifiers. At an average, 4.6 mg/L of DO are needed per milligram per liter NH_4^+-N nitrified. Nitrification is adversely affected by low temperature. As alkalinity is consumed, pH decreases and nitrification is adversely affected below pH 6.5. An alkalinity buffer of 50–60 mg/L as $CaCO_3$ should always be maintained to provide for a stable environment. pH adjustment may be needed if alkalinity of wastewater is less than 150 mg/L as $CaCO_3$ and TKN is high. The alkalinity of typical municipal wastewater is 200 mg/L as $CaCO_3$, and TKN is 40 mg/L. This much alkalinity is sufficient to nitrify 20 mg/L NH_4^+-N and maintain the required buffer with recovery of alkalinity in the denitrification facility.
11. Mechanical equipment and reactor configuration are basic elements of plant design and operation. Multiple sequenced zones are beneficial. The anaerobic zones are generally multiple square basins connected in series by baffled walls. The mixers are submerged paddles or a turbine type that keep the solids in suspension and provides proper blending. Overmixing or excessive power input may create high turbulence and surface aeration. Manufacturers typically oversize mixers with useful turndown capability. The common perforated wall between anaerobic and anoxic zone should not cause nitrate feedback into the anaerobic zone.
12. The aeration device and basin geometry are important to maintain required DO in the aeration basin. Overaeration may result in the carryover of DO in anaerobic and anoxic basins. The aeration devices commonly used are diffused air systems and mechanical aeration. The diffuse air system provides greater flexibility in meeting the varying demands of oxygen. The power source aeration uses different types of rotors and is commonly used in oxidation ditches. The system may be inflexible in meeting the oxygen demand in the entire length of the basin.
13. It is important to provide sufficient free fall and no obstruction between different stages and zones to ensure passage of biological foam and scum downstream. Trapping of foam creates serious operational problems.

13-8 SECONDARY CLARIFIER

The mixed-liquor suspended solids must be settled in a secondary clarifier to produce well-clarified effluent. The design criteria and design procedure for solids removal systems have been presented in Chapter 12. The basic design considerations discussed in Chapter 12 include (1) overflow rate or surface-settling rate, (2) detention period, (3) weir-loading rate, (4) tank shape and dimensions, (5) solid-loading rate, (6) influent structure, (7) effluent structure, and (8) sludge collection and removal. The secondary clarifier in general must perform two functions: (a) provide clarification to produce high-quality effluent and (b) provide thickening of settled solids in the underflow. Therefore, sufficient depth must be provided so that the solids are not lost in the effluent and at the same time there is storage for the settled solids for thickening and maintaining adequate sludge blanket. If sufficient sludge blanket is not maintained, unthickened sludge will be returned to the aeration basin. The design factors are presented in detail in the Design Example. Some typical design values used for secondary clarifiers are listed below:[4,11,12]

1. Overflow rates at average and peak design flows are 15–32 and 40–48 $m^3/m^2 \cdot d$.
2. Solids loadings at average and peak design flows are 49–144 and 100–220 $kg/m^2 \cdot d$
3. The tanks can be circular, rectangular, or square. Circular tanks are 10–60 m in diameter (preferably not to exceed five times the sidewater depth). The desirable range of depth in circular and rectangular clarifiers is 4–6 m.
4. Influent and effluent structures and sludge collection equipment for rectangular and circular basins are given in Chapter 12.

13-8-1 Zone or Hindered Settling (Type III)

The above design values apply to a conventional secondary treatment facility receiving municipal wastewater. If the settling behavior of particles in a clarifier is expected to be different than that from a conventional plant, the settling tests must be conducted to develop the design information for design purposes. The solids concentration in a clarifier is high; therefore, the settling behavior is characterized by zone or hindered settling (Type III).

In hindered or zone settling an interface develops, and solids settle like a blanket, maintaining the same relative position with respect to each other. A relatively clear layer of water is produced above the settling interface. The settling rate of the interface is a function of the solids concentration and their characteristics. The settled depth with time of aerated sludge in a graduated cylinder is shown in Figure 13-22(a). Clarification occurs in the upper zone, while the lower zone of the column exhibits both thickening and compression of solids. Two different approaches are used to develop the design data from the batch column settling tests: (a) single settling column test and (b) solid-flux approach. Both methods are presented below.

Design Based on Single-Batch Test Data. A hindered settling column test [Figure 13-22(a)] is conducted to calculate the area required for (a) sludge thickening and (b) effluent clarification. The design value of the overflow rate is obtained from the larger of

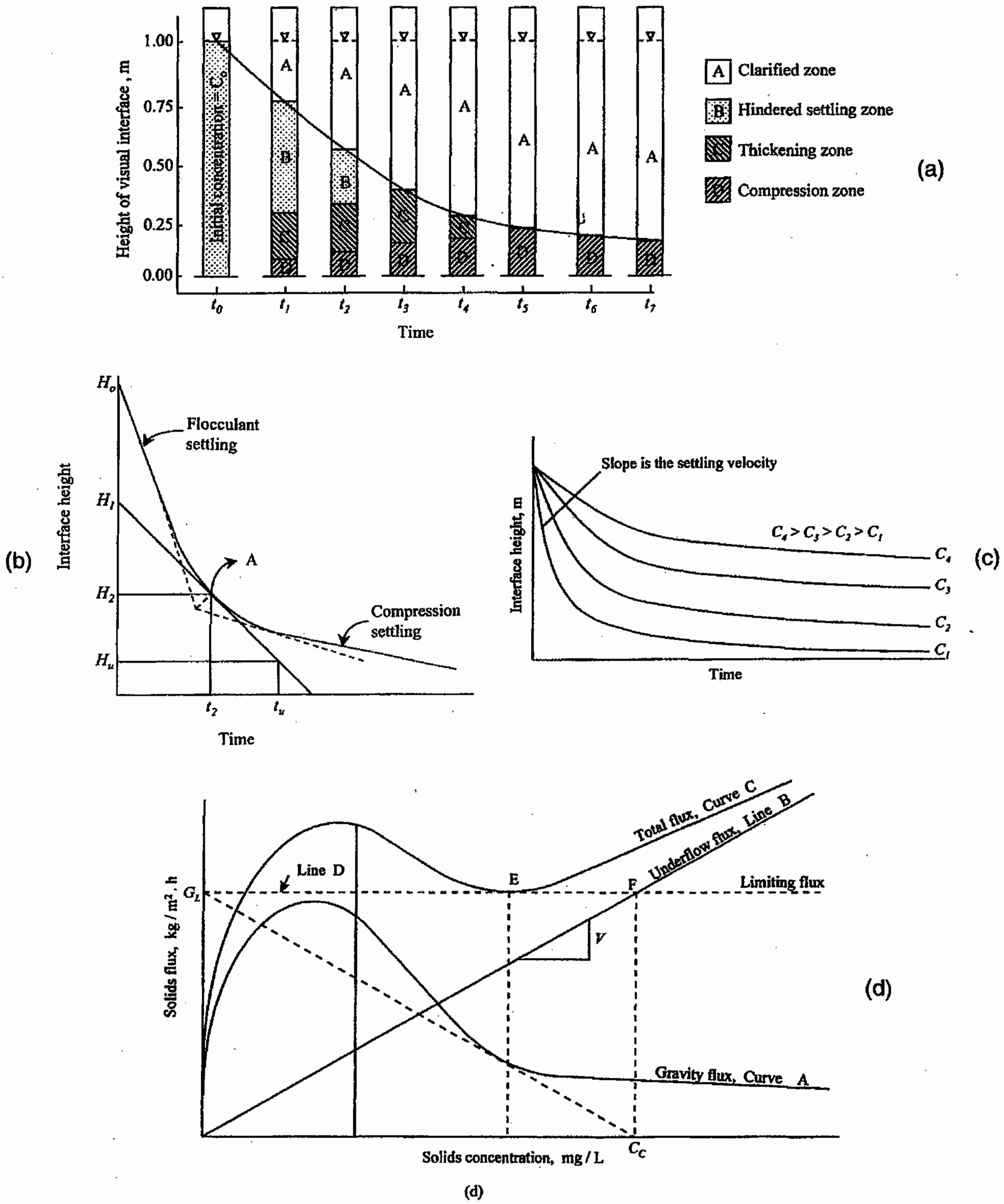


Figure 13-22 Hindered or Zone Settling Behavior of Solids and Obtaining Design Values for Clarifiers: (a) zone settling column test at one solids concentration, (b) graphical procedure for obtaining clarifier area, (c) zone settling column test at different solids concentrations for calculating the gravity flux values, and (d) procedure for calculating the design value of solids flux.

the two areas. The zone-settling equations [Eqs. (13-42)–(13-45)] and design procedure are given below:

$$A_1 = \frac{Qt_u}{H_o} \tag{13-42}$$

$$H_u = \frac{C_o H_o}{C_u} \tag{13-43}$$

$$Q_c = Q\left(\frac{H_o - H_u}{H_o}\right) \tag{13-44}$$

$$A_2 = \frac{Q_c}{V} \tag{13-45}$$

where

A_1 = area required for sludge thickening, m^2
A_2 = area required for clarification, m^2
C_o = concentration of MLSS, g/m^3
C_u = the desired underflow concentration, g/m^3
H_o = initial height of the interface, m
H_u = the depth at which the underflow concentration C_u is reached, m
Q = influent flow, m^3/s
Q_c = clarified effluent, m^3/s
t_u = time to reach the desired underflow concentration, s
V = settling velocity of the interface, m/s

The values of t_u and V are obtained by a graphical method. Tangents are drawn on the flocculant or hindered settling and compression regions of the zone settling curve [Figure 13-22(b)]. A bisector of the angle between these tangents is drawn that cuts the curve at point A. Another tangent is drawn on the curve at point A. This point is important because depth and solids concentration corresponding to this point governs the design of sludge storage and removal systems. The time corresponding to the intersection point of the tangent at A and horizontal line through H_u is t_u. This procedure and design examples on this topic may be found in Refs. 4, 6, 8, 10, and 11.

Design Based on Solids Flux Analysis. The solids flux analysis is used more widely to design the clarifiers. Solids flux is defined as the mass of solids per unit volume passing through a unit area perpendicular to the direction of the flow. In secondary clarifiers, it is the product of the solids concentration (kg/m^3) and the velocity (m/s). Thus, the preferred unit for solids flux is $kg/m^2{\cdot}h$. The downward movement of solids in a secondary clarifier is brought about by (1) the hindered gravity settling of the solids relative to the water and (2) the transport velocity caused by the underflow (withdrawal of sludge). The solids flux caused by gravity and underflow and combined solids flux are expressed by Eqs. (13-46)–(13-48):

$$G_g = V_g C_i \tag{13-46}$$

$$G_u = V_u C_i \tag{13-47}$$

$$G_t = G_g + G_u \tag{13-48}$$

where

G_g = solids flux caused by gravity, kg/m²·h
G_u = solids flux caused by underflow, kg/m²·h
G_t = combined flux caused by gravity and underflow, kg/m²·h
C_i = concentration of solids, mg/L (g/m³)
V_g = hindered settling velocity, m/h
V_u = downward velocity due to underflow, m/h

The hindered settling velocity at different solids concentration is determined by settling column tests. The results of hindered settling column tests are shown in Figure 13-22(c). The slope of the initial portion of the curve is the hindered settling velocity. The velocity caused by underflow is the downward movement resulting from withdrawal of the returned sludge Q_r and area of the clarifier.

The procedure for determining the limiting solids flux for design procedures is given below and is shown in Figure 13-22(d). The calculations for designing a clarifier are provided in the Design Example.

1. The solids flux curve A [Figure 13-22(d)] caused by gravity is prepared from the hindered settling tests. This test is conducted at different solids concentrations as shown in Figure 13-22(c).
2. The underflow flux (curve B) is prepared from the downward velocity resulting from the return sludge.
3. The total flux curve (curve C) is drawn by adding the G_g and G_u values. A tangent is drawn horizontally (line D) from the lowest point E on the total flux curve that gives the value G_L of limiting solids flux concentration or maximum allowable solids loading on the clarifier. The underflow solids concentration C_c, corresponding to the limiting solids flux G_L, is obtained by drawing a tangent on the curve A from the limiting sludge value or a vertical line from the intersection of lines B and D (point F). If the clarifier is designed for a desired underflow solids concentration, the point C_c in Figure 13-22(d) will be known. The limiting solids flux concentration G_L is found by drawing a line tangent to the curve A. This procedure may be found in Refs. 4, 6, 8, 10, and 11. The procedure for obtaining the limiting solids flux and designing a final clarifier is shown in the Design Example.

13-9 EQUIPMENT MANUFACTURERS OF BIOLOGICAL WASTE TREATMENT PROCESSES

A list of manufacturers of mixers, aeration equipment, clarifiers, and sludge pumps is provided in Appendix D. Basic consideration for equipment selection and responsibilities of the design engineer and equipment suppliers are given in Sec. 2-10. The selected equipment should provide the desired operational flexibility and maintenance requirements and also meet the design criteria.

13-10 INFORMATION CHECKLIST FOR DESIGN OF BNR AND CLARIFICATION FACILITIES

The design engineer should make the following important decisions and develop the following data concerning the design of a biological nutrient removal facility:

1. Select a biological treatment process. This includes a critical evaluation of various suspended growth and attached bed reactors and their modifications. The selected process train must provide the required effluent quality under adverse loading conditions.
2. Develop the chemical characteristics of the wastewater reaching the biological treatment facility. A material mass balance analysis must be conducted to establish the effect of return flows to the biological nutrients removal facility. The expected range of flows (minimum, average, and peak design values) and concentrations of soluble BOD_5, total suspended solids (TSS), nutrients, and toxic chemicals under average and sustained loading conditions should also be established.
3. Develop biological kinetic coefficients and treatability data. Laboratory or pilot studies may be necessary if the kinetic coefficients cannot be estimated. Such studies may be particularly necessary for industrial or combined municipal–industrial wastewaters.
4. Develop a preliminary site plan, piping layout, and location of collection boxes, return sludge pumps, and the like with respect to the biological reactor and solids removal system. Information on the existing facility should also be obtained.
5. Obtain design criteria prepared by the concerned regulatory agencies (if any) for biological reactors and solids removal systems.
6. Obtain effluent quality criteria from the secondary treatment facility in terms of BOD_5, TSS, total nitrogen, and total phosphorus.
7. Develop data on settling behavior of the biological solids. If necessary, conduct laboratory studies to determine the solids settling rate, solid flux values, solids loading rate, and overflow rate.
8. Obtain equipment manufacturers and equipment selection guides.

13-11 DESIGN EXAMPLE

13-11-1 Design Criteria Used

The following design criteria shall be used for the design of the biological reactor and solid separation facilities.[87]

Step A: Biological Reactor

1. Design a biological nutrient removal process with an anaerobic, anoxic, and aerobic reactor sequence followed by final clarification.[j]
2. The anaerobic reactor shall receive influent and the returned sludge from the clarifier. The anoxic reactor will receive effluent from the anaerobic reactor and internal recycle from the aerobic reactor. Maximize plug flow regime in both anaerobic and anoxic reactors.
3. The effluent shall have BOD_5/TSS/TN/NH_4^+-N/TP concentrations of 10/10/10/1/1 mg/L or better. The components of TN are NO_3^--N, NH_4^+-N, suspended Org.-N and soluble Org.-N. The achievable concentrations of NO_3^--N = 8 mg/L, NH_4^+-N = 0.5 mg/L. It assumed that 1 mg/L soluble nitrogen in effluent includes NH_4^+-N and soluble Org.-N. The remaining 1 mg/L nitrogen is caused by Org.-N in the effluent TSS.
4. Provide four independent process trains operating in parallel. Necessary cross-connections shall be provided to bypass or distribute flows from one process train to the other under emergency situations.
5. Provide deep and square anaerobic and anoxic reactors with hydraulic detention times of not less than 1.5 hour each. Provide multiple chambers separated by baffle walls. Each compartment will have a mixer with vertical shaft to provide adequate mixing and to keep the solids in suspension but minimize the entry of atmospheric oxygen into the liquid.
6. The anaerobic reactor shall achieve phosphorus release and denitrification of nitrate in the return sludge. The anoxic reactor shall provide denitrification of nitrate in the internal recycle liquid.
7. Equipment for measuring raw wastewater flow, return sludge, waste sludge, internal recycle, and air supply shall be provided.
8. Aeration equipment shall provide complete mixing of the mixed liquor suspended solids (MLSS) and shall be capable of maintaining a minimum of 2.0 mg/L dissolved oxygen in the mixed liquor in all parts of the aerobic reactor at all times.
9. Diffusers and piping shall be capable of delivering 150 percent of the theoretical average air requirements under field conditions. The diffusers and pipings shall be arranged in a manner that routine inspection and maintenance of the diffusers and piping can be accomplished without draining the aeration basin.
10. Blowers shall be capable of delivering the maximum design air requirements considering the largest single unit to be out of service.
11. The sludge pump and piping for return activated sludge shall be designed to provide a capacity up to 150 percent of average design flow. The return activated sludge pump shall be capable of providing a variable capacity ranging from 0 to 150 percent of the average design flow.[k] A standby pumping unit equal in capacity to the

[j]If a conventional aeration basin with nitrification in a single reactor or oxidation ditch is designed, anaerobic and anoxic reactors will not be required. The design procedure for an aerobic reactor with nitrification is covered in Sec. 13-11-5, Steps C, D, and F–J; Sec. 13-11-6, Steps E and F; and Sec. 13-11-7, Steps B–D. It may be noted that material mass balance calculations will be significantly reduced because phosphorus and nitrogen balance will not be needed.

[k]The values given in Table 13-3 for a complete-mix system (Q_r/Q = 0.25–1.00) are the normal operating range.

largest single pump shall be provided. The sludge piping and channels shall be so arranged that routine flushing of solids can be accomplished.

12. The internal recycle system between aeration basin and anoxic basin shall have a capacity of 200 percent of average design flow.
13. All sidestreams from the sludge-handling facilities (thickeners, digesters, and dewatering units) shall be returned to the aeration basin. A material mass balance analysis shall be performed at the average design flow to determine the combined average flows and concentrations of BOD_5, TSS, total nitrogen, and total phosphorus.
14. The sludge wasting shall be achieved from each aeration basin.
15. The influent and effluent structures shall be designed, and the basin hydraulics shall be checked at peak design flow or average design flow plus the design return sludge flow when only three process trains are in operation.
16. The aeration basin shall be designed as a single-stage, suspended growth carbon oxidation–nitrification process. The biological kinetic coefficients and operational parameters for the design purposes shall be determined from carefully controlled laboratory studies. The following kinetic coefficients and design parameters shall be used:

$$\theta_c = 12 \text{ d}$$

$$Y = 0.60 \text{ g TVSS/g BOD}_5\text{, and } 0.20 \text{ g TVSS/g NH}_4^+\text{-N}$$

$$\text{MLVSS} = 3000 \text{ mg/L}$$

$$k_d = 0.06\text{/d for BOD}_5\text{, and } 0.05\text{/d for nitrification}$$

Return sludge concentration = 10,000 mg/L (TSS)

Ratio of MLVSS to MLSS = 0.8

The influent BOD_5 and biodegradable biomass = 68 percent of BOD_L (Sec. 3-4-2)

The biological solids (TSS) are 65 percent biodegradable.

BOD_L/biodegradable biomass = 1.42. This fraction is obtained from the biodegradable biomass expressed by $C_5H_7O_2N$.[1]

17. The following flow, BOD_5, and TSS values shall be used for mass balance analysis. These values are the influent values to the plant and are presented in Table 6-9:

$$\text{Average design flow} = 0.440 \text{ m}^3\text{/s } (38{,}016 \text{ m}^3\text{/d})$$

$$BOD_5 = 250 \text{ mg/L}$$

$$\text{TSS} = 260 \text{ mg/L}$$

The characteristics of design flow to the aeration basin shall be established after the material mass balance analysis.

[1]The oxygen required to stabilize the biodegradable biomass ($C_5H_7O_2N$) into CO_2, H_2O, and NH_3 is 1.42.

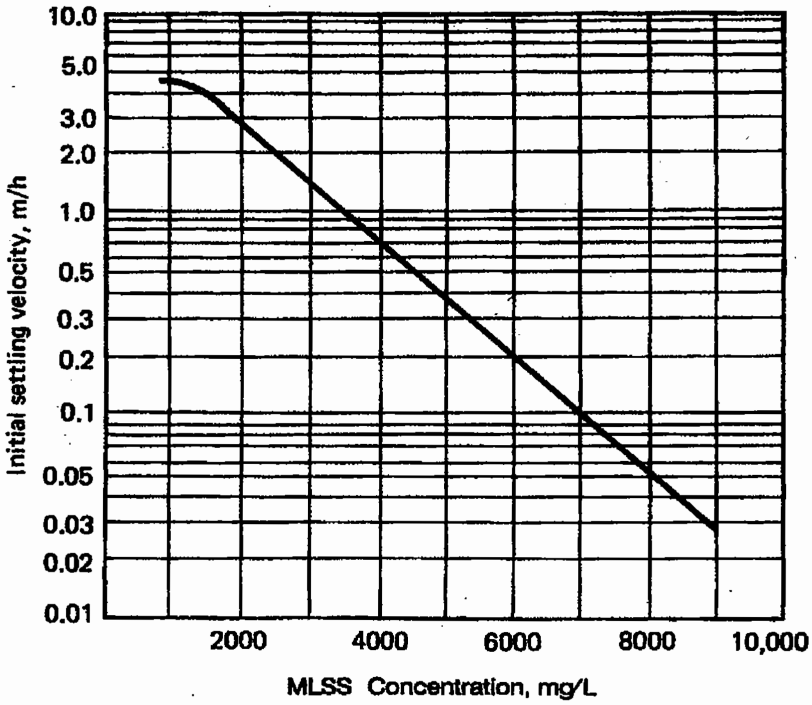


Figure 13-23 Solids-Settling Rate Developed from Experimental Results for Design of Secondary Clarifier.

Step B: Secondary Clarifier.[m] The following design criteria shall be used for the solids separation facility:

1. Provide four circular clarifiers, each clarifier having independent operation with respect to the aeration basins.
2. Design the clarifiers for average design flow plus the recirculation.
3. Design the influent and effluent structures, and check the hydraulics at peak design flow.
4. Return sludge from each clarifier shall have an independent sludge withdrawal arrangement with flow measurement and control devices.
5. The design of the clarifier shall be based on the solids-settling rate obtained from laboratory results illustrated in Figure 13-23.
6. The surface area of the clarifier shall be large enough to meet the clarification as well as the thickening requirements for the effluent and the underflow, respectively.
7. The water depth of the clarifier shall be sufficient to provide an adequate clearwater zone, thickening zone, and sludge storage zone.
8. The overflow rates at average and peak design flow conditions shall not exceed 15 and 35 $m^3/m^2 \cdot d$, respectively.[n]
9. The solids-loading rates at average and peak design flows shall not exceed 50 and 150 $kg/m^2 \cdot d$, respectively.[o]

[m]For comparison purposes, see the design criteria for the primary sedimentation facility given in Sec. 12-8-1.

[n]Based on influent flow only.

[o]Based on influent flow plus return sludge at design MLSS concentration.

10. Scum baffles and scum collection system shall be provided.
11. The effluent weir shall be designed to prevent turbulence. The weir loading shall not exceed 124 $m^3/m \cdot d$ (10,000 gpd/ft) and 372 $m^3/m \cdot d$ (30,000 gpd/ft) at average and peak design flows, respectively.

13-11-2 Unit Arrangement and Piping Layout

The preliminary layout of anaerobic, anoxic, and aerobic reactors; final clarifiers; return and waste sludge pumps and pipings; internal recycle; and aeration systems are shown schemetically in Figure 13-24. The generalized process train includes a junction splitter box that receives primary treated effluent and the sidestreams from sludge-processing areas. The combined flow is then discharged over four identical weirs for division into four pressure pipes that lead to four anaerobic reactors. Each weir is adjustable,[p] has a stop gate,[q] and is equipped with a head measurement system over the weir.

Three square compartments of the anaerobic reactor in each process train are connected in series by baffle walls. One vertical mixer in each compartment provides mixing for solids suspension. Influent and return sludge enter the influent channel of the first compartment. The mixed liquor from the anaerobic reactor enters the influent channel of the first compartment of the anoxic reactor that also has three identical square compartments in series and identical mixers. Internal recycle flow from the aeration basin is also brought into the influent channel of the first compartment of the anoxic basin. The denitrified flow from the third compartment of the anoxic reactor is discharged into an influent channel that distributes flow into the aeration basin through multiple ports.

In aeration basins, the swing-type diffusers are arranged in several rows perpendicular to the direction of flow. These diffusers provide complete mixing and also supply the required oxygen demand. A portion of the aerated MLSS is wasted from each aeration basin to the sludge-processing area. The well-aerated MLSS is discharged over a rectangular weir and then into a collection box. The MLSS is piped from each collection box to the respective clarifiers. The effluent from the final clarifier is discharged into a common junction box and then conveyed to the disinfection facility. The return sludge from each clarifier is pumped to the first compartment of the respective anaerobic reactor. This arrangement essentially provides four independent process trains arranged in parallel. Proper cross-connections are made for emergency situations.

13-11-3 Design Strategy

The BNR system is complex to design. In the Design Example a three-zone system (anaerobic, anoxic, and aerobic) is provided. Although the three zones carry out different functions, they are integrated into a combination whereby their functions, such as substrate utilization and biomass growth, overlap. The following design sequence is used to explain fully the procedure and document the basis for design and information sources.

[p]The adjustable weirs shall be used to divide the flows equally into anaerobic basins.

[q]The stop gate shall be used to remove any process train from operation.

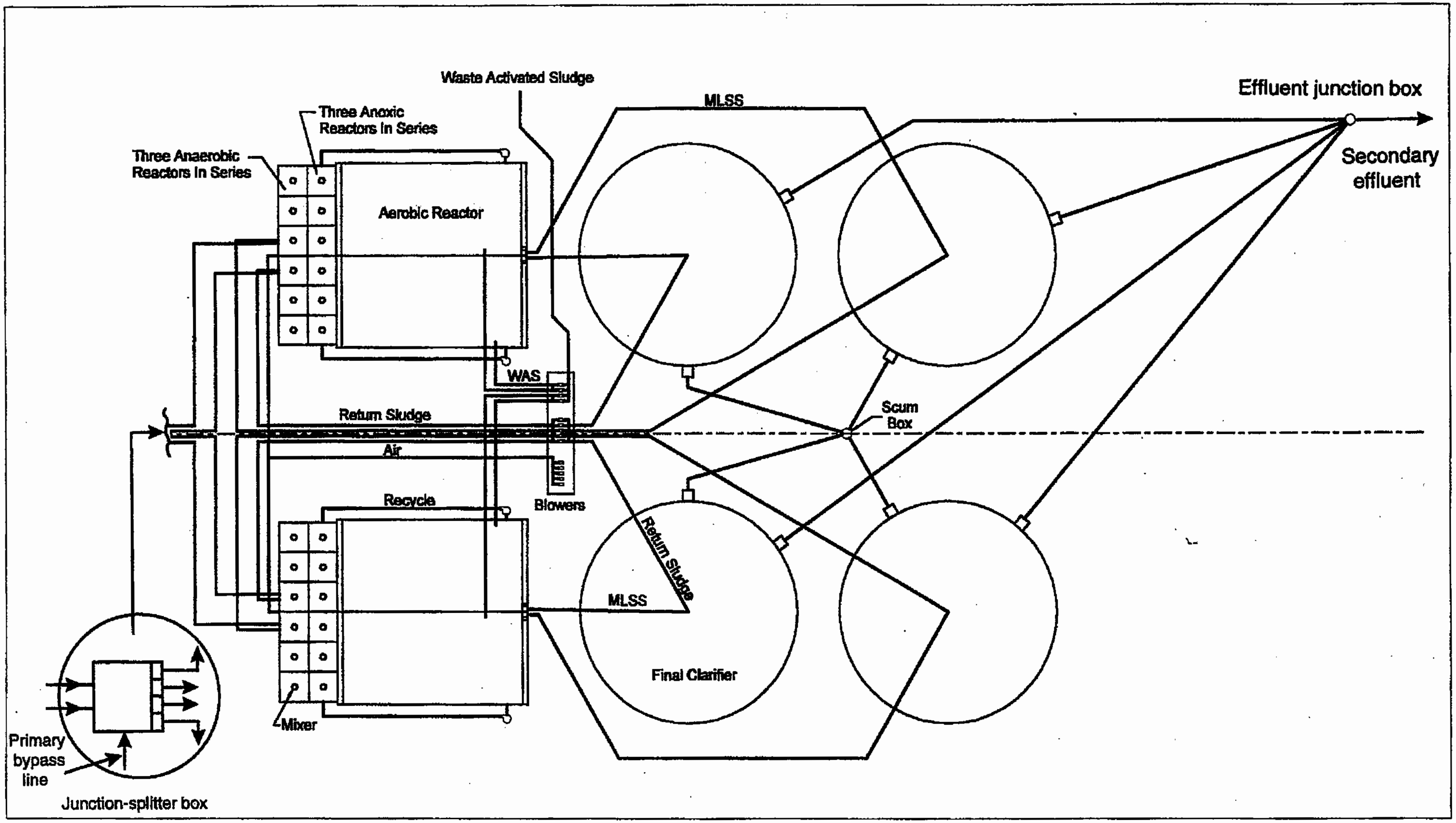


Figure 13-24 Schematic Flow and Piping Arrangement of Biological Nutrient Removal Facility.

The information is utilized from one zone to another because the three zones cannot be completely separated from each other.

1. Conduct a material mass balance analysis to develop influent characteristics to the BNR system.
2. Calculate the HRT of anaerobic zone.
3. Calculate the SRT, biomass increase, and HRT of anoxic zone.
4. Calculate the SRT, biomass increase, and HRT of aerobic zone.
5. Calculate the dimension of anaerobic, anoxic, and aerobic reactors.
6. Design the equipment requirement for mixing anaerobic and anoxic basins.
7. Develop oxygen requirements and the aeration system for aerobic reactor.
8. Select influent and effluent structure, perform hydraulic calculations, and develop hydraulic profile.
9. Design the solids separation system.

13-11-4 Design Calculations, Material Mass Balance, and Determination of Influent Quality

The sludge-processing facilities produce liquids that are normally returned to the plant. The return flows increase hydraulic, organic, solids, and nutrients loadings to the treatment units below the point of discharge. Many designers prefer to return these flows to the wet well and blend them with the incoming wastewater. In many cases, however, these flows may also be returned to the primary sedimentation or secondary treatment units. Each location of returned flow must be carefully evaluated. Considerations should be given to (1) relative volume and incremental solids and organic loadings added by the returned flow, (2) geometry and location of treatment units, (3) treatment processes and available treatment capacity, and (4) odor problems that may result because of the return flows. A material mass balance analysis is needed to determine the combined flows and the concentrations of BOD_5; TSS; organic, ammonia, nitrate, and total nitrogen; and total phosphorus. If the sidestreams are produced intermittently, such as in the case of thickeners, anaerobic digesters, and belt filter presses, an equalization basin may be provided to avoid surge loadings. In the Design Example, the flows from the thickener, digester, and dewatering facilities are returned to the aeration basin. The performance of sludge processing facilities are given in Chapters 4 and 16–18. The material mass balance procedure is given in this section. An example of all return flows to the head of the plant may be found in Ref. 4. A computer software was developed and used to obtain the final and alternative material mass balance strategies (see Sec. 13-11-4, Step B).

Step A: First Iteration (see Figure 13-25 for reference).

1. Calculate the characteristics of raw wastewater (stream 1), primary sludge (stream 3), and primary effluent (stream 4).
   The characteristics of all these streams in terms of flow, TSS, BOD_5, Org.-N, NH_4^+-N, NO_3^--N, TN, and TP have been developed in Chapter 12 and are summa-

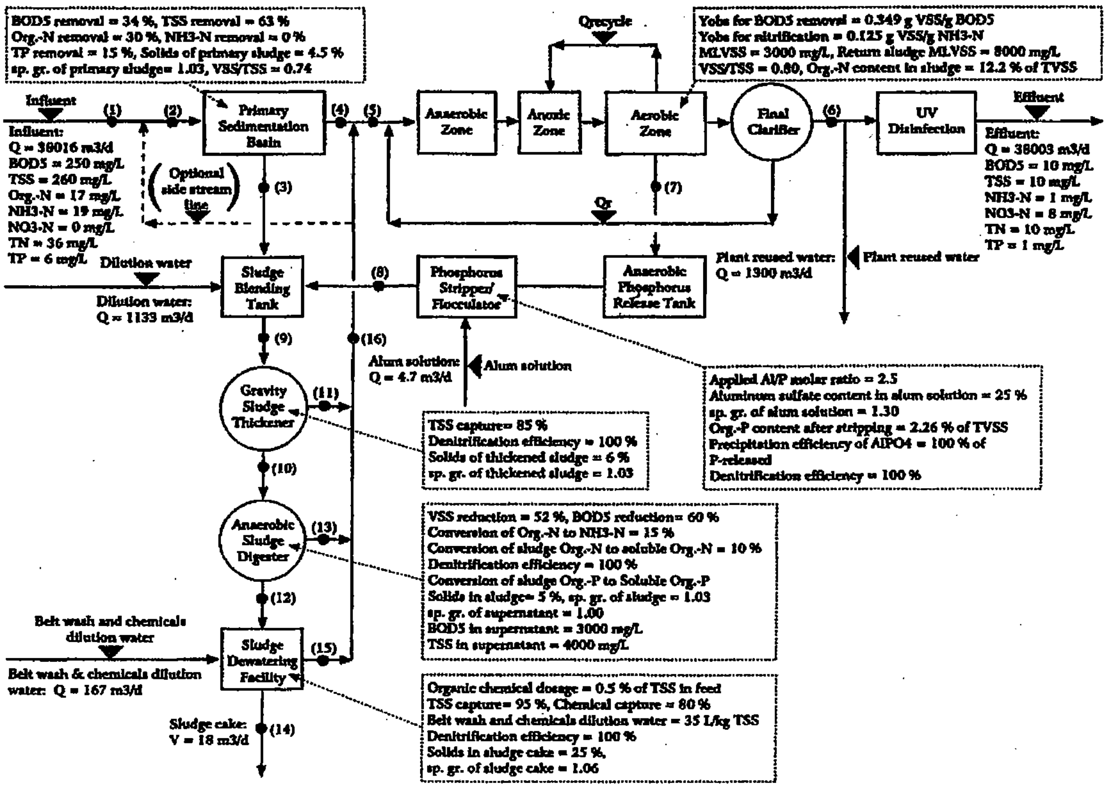


Figure 13-25 Stream Identification and Process Performance for Material Mass Balance Analysis.

rized in Table 12-8. The values for above streams, along with those for several other streams calculated in this section, are provided in Table 13-12 for ready reference. Several other parameters useful for material mass balance and other information are computed below:

a. Determine percent Org.-N and TP in primary sludge (stream 3). Refer to values given in Table 13-12.

$$\text{Org.-N} = \frac{194\ \text{kg/d}}{6227\ \text{kg/d} \times 0.74^{\mathrm{r}}} = 0.42 \text{ or } 4.2\% \text{ of TVSS}$$

$$\text{TP} = \frac{37\ \text{kg/d}}{6227\ \text{kg/d} \times 0.74} = 0.008 \text{ or } 0.80\% \text{ of TVSS}$$

b. Determine the organic fraction and biodegradable fraction of TSS in the primary sludge (stream 3).

The biodegradable fraction of TSS is calculated from BOD_5 exerted by TSS. Assume that the biodegradable fraction of TSS = $f_{\text{biodegradable}}$.

[r] The organic fraction of primary sludge is assumed to be 0.74 of TSS. This a typical value for primary sludge.

TABLE 13-12 Characteristics of Streams in First Iteration of Material Mass Balance Analysis

		Constituent												
		kg/d (Values in parenthesis are in mg/L)									Fraction of TSS		Fraction of TVSS	
Stream No.	Flow (m^3/d)	BOD_5	TSS	Org.-N	NH_4^+-N	NO_3^--N	TN	NPP[a]	PP[a]	TP	TVSS/TSS	Biodegradable Solids/TSS	Org.-N/TVSS	NPP/TVSS
1	38016	9504	9884	646	722	0	1369			228				
		(250)	(260)	(17)	(19)	0	(36)			(6)				
2	38016	9504	9884	646	722	0	1369			228				
		(250)	(260)	(17)	(19)		(36)			(6)				
3	134	3231	6227	194	2.6	0	197			37	0.74[b]	0.54	0.042	0.008
4	37882	6273	3657	452	720	0	1172			192				
		(166)	(97)	(12)	(19)		(31)			(5)				
5	37882	6273	3657	452	720	0	1172			192				
		(166)	(97)	(12)	(19)		(31)			(5)				
6	37882[c]	379	379	38	38	303	379			38				
		(10)[d]	(10)[d]	(1)[d]	(1)[d]	(8)[d]	(10)[d]			(1)[d]				
7	643	1515	2410	235	0.6	5.1	241			154	0.80	0.64	0.122	0.080
8	648[e]	1515	2921[f]	235	0.6	0[g]	236	44	110	154	0.66	0.54	0.122	0.023

9	1911	4757	9159	430	4.3	0[h]	434	82	110	192	0.72	0.54	0.065	0.012
10	126	4043	7785	366	0.3	0	366	70	110	180	0.72	0.54	0.065	0.012
11	1785	714	1374	64	4	0	68	12		12				
		(400)	(770)	(36)	(2.2)		(38)	(7)		(7)				
12	92	1515	4734	294	40	0	334	63	110	173	0.55	0.33	0.11	0.024
13	34	102	136	18	15	0	33	7		7				
		(3000)	(4000)	(529)	(441)		(971)	(206)		(206)				
14	17	1439	4516	279	7.4	0	286	59	111	170	0.55	0.33	0.11	0.024
15	241	78	243	15	33	0	48	3		3				
		(315)	(1000)	(62)	(158)	(0)	(220)	(12)		(12)				
16	2060	894	1753	97	52	0	149	22		22				
		(433)	(850)	(47)	(25)	(0)	(75)	(11)		(11)				

[a]NPP = nonprecipitated phosphorus and PP = precipitated phosphorus.
[b]TVSS/TSS in primary sludge = 0.74 (assumed).
[c]Effluent flow = Influent flow − Volume of waste activated sludge (WAS). In first iteration, it is assumed that WAS = 0.
[d]Values are calculated from the effluent standard (Table 6-1). Note that 1 mg/L NH_4^+-N concentration in effluent also includes soluble Org.-N in the effluent and 1 mg/L Org.-N is caused by 10 mg/L of biomass (TSS) in the effluent.
[e]Increase in volume is caused by addition of liquid alum (4.7 m^3/d).
[f]Increase in TSS (total 511 kg/d) is caused by precipitation of $AlPO_4$ and $Al(OH)_3$.
[g]NO_3^--N is denitrified in anaerobic phosphorus stripper.
[h]NO_3^--N is denitrified in thickener.

$$\text{BOD}_5 \text{ exerted by TSS, kg BOD}_5\text{/d} = (\text{kg TSS/d}) \times f_{\text{biodegradable}} \times (1.42 \text{ kg BOD}_L\text{/kg biodegradable solids}^{s}) \times (0.68 \text{ kg BOD}_5\text{/kg BOD}_L{}^{s})$$

$$(3231 \text{ kg BOD}_5\text{/d}) = (6227 \text{ kg TSS/d}) \times f_{\text{biodegradable}} \times (1.42 \text{ kg/kg}) \times (0.68 \text{ kg/kg})$$

Solving $f_{\text{biodegradable}}$:

$f_{\text{biodegradable}} = 0.54$ kg biodegradable solids/kg TSS

2. Determine the characteristics of influent flow to the primary sedimentation basin (stream 2) and influent flow to biological wastewater treatment facility (stream 5). The characteristics of stream 16 are developed at the end of the first iteration. Initially, the concentrations of various components in this stream are not known, and therefore are assumed to be zero. Sidestream 16 has two alternate return options, that is, before or after primary treatment. In either case, there is no effect of this stream during the first iteration.
   - The characteristics of stream 2 are the same as those for stream 1 (Table 13-12).
   - The characteristics of stream 5 are the same as those for stream 4.
3. Determine the quantity of various constituents in the plant effluent (stream 6). The effluent standards are given in Table 6-1. The mass of various constituents are calculated from flow and concentration data. These values are summarized in Table 13-12.
4. Develop the characteristics of waste activated sludge (WAS) (stream 7). The BNR system is designed to achieve BOD_5 stabilization, nitrification, and denitrification. The increase in biomass is calculated from overall BOD_5 removal and nitrification. The biomass production from denitrification is small and is ignored for mass balance. The denitrification effect and phosphorus release and uptake are covered in detail in Secs. 13-7-1 and 13-7-2. The kinetic coefficients θ_c, Y, and k_d for BOD_5 removal and nitrification are given in the design criteria (see Sec. 13-11-1, Step A, 16).
   a. Compute $Y_{\text{obs, BOD}}$ for overall BOD_5 removal (see also Sec. 13-11-5, Step C, 3, a).

$$Y_{\text{obs, BOD}} = \frac{Y_{\text{BOD}}}{1 + k_{d,\text{ BOD}}\,\theta_c}$$

$$= \frac{0.60 \text{ kg TVSS/kg BOD}_5}{1 + (0.06 \text{ d}^{-1})(12 \text{ d})}$$

$$= 0.349 \text{ kg TVSS/kg BOD}_{5,\text{ removed}}$$

   b. Compute $Y_{\text{obs, N}}$ for nitrification (see also Sec. 13-11-5, Step C, 4, a).

$$Y_{\text{obs, N}} = \frac{Y_{\text{N}}}{1 + k_{d,\text{ N}}\,\theta_c}$$

$$= \frac{0.20 \text{ kg TVSS/kg NH}_4^+\text{-N}}{1 + (0.05 \text{ d}^{-1})(12 \text{ d})}$$

$$= 0.125 \text{ kg TVSS/kg NH}_4^+\text{-N}_{\text{removed}}$$

[s]See design criteria (Sec. 13-11-1, Step A, 16).

c. Compute the soluble BOD_5 in the effluent.

$$\begin{array}{lcl} \text{The soluble} & \text{Total} & \text{BOD}_5 \text{ exerted} \\ \text{BOD}_5 \text{ in the} = & \text{BOD}_5 \text{ in} & - \text{ by TSS in the} \\ \text{effluent } (S) & \text{the effluent} & \text{effluent} \end{array}$$

$$\begin{array}{l} \text{BOD}_5 \text{ exerted} \\ \text{by TSS in the} \\ \text{effluent} \end{array} = (10 \text{ mg TSS/L}^{t}) \times (0.65 \text{ biodegradable solids/kg TSS})$$

$$\times (1.42 \text{ kg BOD}_L\text{/kg biodegradable solids})$$
$$\times (0.68 \text{ kg BOD}_5\text{/kg BOD}_L)$$
$$= 6.3 \text{ mg/L}$$

$$S = 10 \text{ mg/L} - 6.3 \text{ mg/L} = 3.7 \text{ mg/L}$$

d. Compute the biological solids increase due to overall BOD_5 removal.

$$\text{TVSS increase} = Y_{\text{obs}} (S_o - S)\, Q$$

$$\begin{array}{l} \text{TVSS increase} \\ \text{due to overall} \\ \text{BOD}_5 \text{ removal} \end{array} = (0.349 \text{ kg TVSS/kg BOD}_5) \times [(166 - 3.7) \text{ g BOD}_5\text{/m}^3]$$

$$\times 37882 \text{ m}^3\text{/d} \times \left(\frac{\text{kg}}{1000 \text{ g}}\right)$$

$$= 2146 \text{ kg/d}$$

e. Compute the biological solids increase due to nitrification.

$$\begin{array}{c} \text{TVSS increase} \\ \text{due to} \\ \text{nitrification} \end{array} = (0.125 \text{ kg TVSS/kg NH}_4^+\text{-N(step 4b)}) \times [(19 - 1^{u}) \text{ g NH}_3\text{-N/m}^3]$$

$$\times 37{,}882 \text{ m}^3\text{/d} \times \left(\frac{\text{kg}}{1000 \text{ g}}\right)$$

$$= 85 \text{ kg/d}$$

f. Compute total TSS increase.

$$\begin{array}{l} \text{Total} \\ \text{TVSS increase} \end{array} = 2146 \text{ kg/d} + 85\text{kg/d} = 2231 \text{ kg/d}$$

$$\text{TSS increase} = \frac{2231 \text{ kg/d}}{0.8 \text{ kg TVSS/kg TSS}^{v}} = 2789 \text{ kg/d}$$

$$\begin{array}{l} \text{TSS lost} \\ \text{in the effluent} \end{array} = 379 \text{ kg/d (see Table 13-12)}$$

$$\text{TSS in WAS} = 2789\text{kg/d} - 379\text{kg/d} = 2410\text{kg/d}$$

[t]TSS in effluent is given in the effluent criteria.

[u]Includes soluble Org.-N in the effluent.

[v]See the design criteria (Sec. 13-11-1, Step A, 16).

g. Compute the volume of WAS.

$$\text{The concentration of TSS in WAS} = \frac{3000 \text{ mg TVSS/L}}{0.8 \text{ kg TVSS/kg TSS}}$$

$$= 3750 \text{ mg/L or } 3.75 \text{ kg/m}^3$$

$$\text{Volume of WAS (sp. gr.} = 1.00) = \frac{2410 \text{ kg/d}}{3.75 \text{ kg/m}^3} = 643 \text{ m}^3\text{/d}$$

h. Compute total BOD_5 in WAS.

$$\begin{aligned} BOD_5 \text{ exerted by TSS} &= (2410 \text{ kg/d}) \times (0.65 \text{ biodegradable solids/kg TSS}) \\ &\quad \times (1.42 \text{ kg } BOD_L\text{/kg biodegradable solid}) \\ &\quad \times (0.68 \text{ kg } BOD_5\text{/kg } BOD_L) \\ &= 1513 \text{ kg/d} \end{aligned}$$

$$\text{Soluble } BOD_5 = 3.7 \text{ g/m}^3 \text{ (Step 4, c)} \times 643 \text{ m}^3\text{/d} \times \left(\frac{\text{kg}}{1000 \text{ g}}\right) = 2.4 \text{ kg/d}$$

$$\text{Total } BOD_5 = 1513 \text{ kg/d} + 2.4 \text{ kg/d} = 1515 \text{ kg/d}$$

i. Compute Org.-N, NH_4^+-N, NO_3^--N and TN in WAS.

$$\begin{aligned} \text{Org.-N} &= 0.122 \text{ kg Org.-N/kg TVSS}^{w} \times 2410 \text{ kg TSS/d} \\ &\quad \times 0.8 \text{ kg TVSS/kg TSS} \\ &= 235 \text{ kg/d} \end{aligned}$$

$$NH_4^+\text{-N} = 1 \text{ g } NH_4^+\text{-N}^{x}\text{/m}^3 \times 643 \text{ m}^3\text{/d} \times \left(\frac{\text{kg}}{1000 \text{ g}}\right) = 0.64 \text{ kg/d}$$

$$NO_3^-\text{-N} = 8 \text{ g } NO_3^-\text{-N/m}^3 \times 643 \text{ m}^3\text{/d} \times \left(\frac{\text{kg}}{1000 \text{ g}}\right) = 5.1 \text{ kg/d}$$

$$\text{TN} = 235 \text{ kg/d} + 0.64 \text{ kg/d} + 5.1 \text{ kg/d} = 241 \text{ kg/d}$$

j. Compute the total NO_3^--N lost by denitrification in anaerobic and anoxic reactors.

$$\begin{aligned} \text{Total } NO_3^-\text{-N lost by denitrification} &= \text{TN in the influent to biological system} \\ &\quad - \text{TN lost in the effluent} - \text{TN in WAS} \\ &= 1172 \text{ kg/d (stream 5 in Table 13-12)} \\ &\quad - 379 \text{ kg/d (stream 6 in Table 13-12)} - 241 \text{ kg/d} \\ &= 552 \text{ kg/d} \end{aligned}$$

[w]The mixed biological growth is also expressed by the chemical formula $C_{60}H_{87}O_{23}N_{12}P$. The nitrogen content is 12.2 percent of TVSS. The soluble Org.-N in WAS is small and is ignored.

[x]Includes soluble Org.-N in effluent.

k. Compute return flow Q_r into anaerobic zone.

$$10{,}000 \text{ g/m}^3 \times (Q_r \text{ m}^3\text{/d}) = 3750 \text{ g/m}^3 \times [(Q + Q_r) \text{ m}^3\text{/d}]$$

$$Q_r = 22729 \text{ m}^3\text{/d}$$

$$\frac{Q_r}{Q} = \frac{22729 \text{ m}^3\text{/d}}{37882 \text{ m}^3\text{/d}} = 0.6$$

l. Compute recycle flow $Q_{recycle}$ into anoxic zone.
Total nitrogen lost by denitrification of NO_3^--N in Q_r and $Q_{recycle}$ must be equal to 552 kg/d if denitrification is complete.

$$552 \text{ kg/d} = (22{,}729 \text{ m}^3\text{/d}) \times (8 \text{ g } NO_3\text{-N/m}^3) \times \left(\frac{1 \text{ kg}}{1000 \text{ g}}\right)$$

$$+ (Q_{recycle} \text{ m}^3\text{/d}) \times (8 \text{ g } NO_3^-\text{-N/m}^3) \times \left(\frac{1 \text{ kg}}{1000 \text{ g}}\right)$$

$$Q_{recycle} = 46,\ 271 \text{ m}^3\text{/d}$$

$$\frac{Q_{recycle}}{Q} = \frac{46271 \text{ m}^3\text{/d}}{37882 \text{ m}^3\text{/d}} = 1.22 \approx 1.25$$

m. Compute TP in WAS.

$$\begin{aligned} \text{TP in WAS, kg/d} &= \text{TP in influent to BNR} - \text{TP in effluent} \\ &= 192 \text{ kg/d (stream 5 in Table 13-12)} \\ &\quad -38 \text{ kg/d (stream 6 in Table 13-12)} \\ &= 154 \text{ kg/d} \end{aligned}$$

$$\%\text{ TP in TSS} = \frac{154 \text{ kg/d} \times 100\%}{2410 \text{ TSS increase, kg/d (Step 4, f)}} = 6.4\%$$

$$\%\text{ TP in VSS} = \frac{6.4\% \text{ TP/TSS}}{0.8 \text{ TVSS/TSS}} = 8.0\%$$

5. Compute the characteristics WAS after PO_4^{3-}-P stripping (stream 8).
The WAS is rich in phosphorus, which will be released in the gravity thickener and anaerobic digester. As a result, the concentration of phosphorus in the sidestream would increase.[88] To avoid phosphorus buildup in the BNR system, an anaerobic phosphorus stripper is provided. In this unit, the PO_4^{3-} ions are released under anaerobic conditions, and PO_4^{3-}-P is precipitated as $AlPO_4$ by alum coagulation. Also, NO_3^--N in WAS is denitrified. Detailed information about these concepts is covered in Sec. 13-11-5, Step E.
   a. Compute the amount of phosphorus released in the anaerobic phosphorus stripper.

Assume that the phosphorus content in the mixed activated sludge is 2.3 percent of TVSS.[y]

$$\text{PO}_4\text{-P release} = (0.080 \text{ g-P/g TVSS in WAS} - 0.023 \text{ g-P/TVSS}) \times (2410 \text{ kg-TSS/d}) \times (0.8 \text{ TVSS/TSS}) = 110 \text{ kg/d}$$

b. Compute the TSS increase caused by precipitatoin of $AlPO_4$ and $Al(OH)_3$. At an Al^{3+}/P molar ratio of 2.5, over 98 percent released PO_4^{3-}-P will be precipitated as $AlPO_4$. It is assumed that 100 percent of released phosphorus is precipitated. There will be no net increase in solids due to precipitation of PO_4^{3-} ions.[z] Only Al^{3+} in $AlPO_4$ will show an increase. In addition, the excess alum will react with alkalinity to produce $Al(OH)_3$. Therefore, the total TSS increase is caused by precipitation of Al^{3+} as $AlPO_4$ and precipitation of alkalinity as $Al(OH)_3$.

$$\begin{array}{l}\text{TSS increase caused by} \\ \text{precipitation of Al}^{3+} \\ \text{as AlPO}_4\end{array} = \frac{\text{molar wt. of Al}}{\text{molar wt. of P}} \times \begin{array}{l}(\text{amount of PO}_4^{3-}\text{-P} \\ \text{precipitated or} \\ \text{released})\end{array}$$

$$= \frac{27}{31} \times 110 \text{ kg/d (Step 5, a)} = 96 \text{ kg/d}$$

$$\begin{array}{l}\text{TSS increase caused} \\ \text{by precipitation of} \\ \text{Al(OH)}_3\text{[aa]}\end{array} = \frac{\text{molar wt. of Al(OH)}_3}{\text{molar wt. of P}} \times \begin{array}{l}(\text{amount of PO}_4^{3-}\text{-P} \\ \text{precipitated or} \\ \text{released})\end{array}$$

$$\times (\text{applied Al}^{3+}\text{/P molar ratio} - \text{Al}^{3+}\text{/P molar ratio in AlPO}_4^{-3})$$

$$= \frac{78}{31} \times 110 \text{ kg/d} \times (2.5 - 1) = 415 \text{ kg/d}$$

$$\begin{array}{l}\text{Total increase of} \\ \text{TSS caused by} \\ \text{precipitation}\end{array} = 96 \text{ kg/d} + 415 \text{ kg/d} = 511 \text{ kg/d}$$

$$\text{TSS (total)} = 2410 \text{ kg/d (Step 4, f)} + 511 \text{ kg/d} = 2921 \text{ kg/d}$$

c. Compute the volume of liquid alum and total volume of sludge after precipitation of P.
Liquid alum is used as a coagulant; the $Al_2(SO_4)_3$ content in liquid alum is 25 percent; and the specific gravity of liquid alum is 1.3 (1300 kg/m^3).

[y]Based on the chemical formula $C_{60}H_{87}O_{23}N_{12}P$ for mixed biomass growth, the phosphorus content is approximately 2.3 percent of TVSS.

[z]It may be noted that the precipitated PO_4^{3-}-P is released from the cell mass; therefore PO_4^{3-}-P lost from cell = PO_4^{3-}-P gain in the precipitates.

[aa]For a more detailed discussion on this topic, see Sec. 13-11-5, Step E.

$$\begin{array}{c}\text{Total amount}\\ \text{of Al}^{3+}\text{ applied}\end{array} = \frac{\text{amount of PO}_4^{3-}\text{-P precipitated}}{\text{molar wt. of P}} \times \begin{array}{l}\text{applied Al}^{3+}\text{/P}\\ \text{molar ratio}\end{array}$$

$$\times \text{ molar wt. of Al}$$

$$= \frac{110\text{ kg/d (Step 5, a)}}{31} \times 2.5\text{ (Step 5, b)} \times 27$$

$$= 240\text{ kg Al}^{3+}\text{/d}$$

$$\begin{array}{c}\text{Volume of}\\ \text{liquid alum}\\ \text{solution}\end{array} = \frac{\text{total amount of Al}^{3+}\text{ applied} \times \text{molar wt. of Al}_2(\text{SO}_4)_3}{2^{bb} \times \text{molar wt. of Al} \times 0.25\text{ (Step 5, c)} \times 1300\text{ kg/m}^3\text{ (Step 5, c)}}$$

$$= \frac{240\text{ kg/d} \times 342}{2 \times 27 \times 0.25 \times 1300\text{ kg/m}^3}$$

$$= 4.7\text{ m}^3\text{/d}$$

The total volume of WAS after precipitation of P

$$= 643\text{ m}^3\text{/d (Step 4, g)} + 4.7\text{ m}^3\text{/d} = 648\text{ m}^3\text{/d}$$

The other components of stream 8 will be the same as those for stream 7, except that NO_3^--N = 0 kg/d because of complete denitrification in the stripper tank. The characteristics of stream 8 are given in Table 13-12.

d. Compute biodegradable and organic fractions of TSS.

$$\text{Biodegradable fraction} = \frac{1513\text{ kg/d (Step 4, h)}}{2921\text{ kg/d (Step 5, b)} \times 1.42 \times 0.68} = 0.54$$

$$\text{Organic fraction} = \frac{1}{2921\text{ kg/d}} \times \{0.8 \times 2410\text{ kg TSS/d (Step 4, f)}$$

$$+ 0\text{ (organic fraction)} \times [511\text{ kg/d (Step 5, b)}\}$$

$$= 0.66$$

6. Determine the characteristics of combined, blended sludge (stream 9).
The primary sludge and WAS are mixed and blended. Dilution water is needed to maintain hydraulic loading in the thickener (Chapter 16). Final plant effluent is used as dilution water.
   a. Compute the TSS and volume of combined sludge.

$$\text{TSS} = 6227\text{ kg/d (stream 3 in Table 13-12)} + 2921\text{ kg/d (Step 5, b)}$$
$$= 9148\text{ kg/d}$$

$$\begin{array}{l}\text{Total volume of}\\ \text{combined sludge}\end{array} = 134\text{ m}^3\text{/d (stream 3 in Table 13-12)} + 648\text{ m}^3\text{/d (Step 5, c)}$$

$$= 782\text{ m}^3\text{/d}$$

[bb]There are two atoms of Al^{3+} in each molecule of $Al_2(SO_4)_3$.

b. Compute the volume of dilution water.
Thickener area is calculated based on solids loading of 46.9 kg/m²·d.

$$\text{Thickener area} = \frac{9148 \text{ kg/d (Step 6, a)}}{46.9 \text{ kg/m}^2\cdot\text{d}} = 195 \text{ m}^2$$

Flow to the thickener is determined based on a hydraulic loading of 9.8 m³/m²·d.
Flow to the thickener = 9.8 m³/m²·d × 195 m² = 1911 m³/d

$$\begin{aligned}\text{Volume of dilution water} &= 1911 \text{ m}^3\text{/d} - 782 \text{ m}^3\text{/d (Step 6, a)}\\ &= 1129 \text{ m}^3\text{/d}\end{aligned}$$

c. Compute the characteristics of blended sludge.
The mass of various compounds in the blended sludge (stream 9) are obtained by adding the respective values of streams 3 and 8 and those in the dilution water. The plant effluent is used as dilution water, and the amounts of various components in dilution water are proportional to the flows. For example,

$$\begin{aligned}\begin{matrix}\text{BOD}_5 \text{ and TSS}\\ \text{in dilution water}\end{matrix} &= \frac{379 \text{ kg/d (stream 6 in Table 13-12)} \times 1129 \text{ m}^3\text{/d}}{37{,}882 \text{ m}^3\text{/d}}\\ &= 11 \text{ kg/d (each)}\\ \text{BOD}_5 \text{ in stream 9} &= 3231 \text{ kg/d (stream 3 in Table 13-12)}\\ &\quad + 1515 \text{ kg/d (Step 4, h)} + 11 \text{ kg/d}\\ &= 4757 \text{ kg/d}\end{aligned}$$

$$\text{TSS in stream 9} = 9148 \text{ kg/d (Step 6, a)} + 11 \text{ kg/d} = 9159 \text{ kg/d}$$

Similarly, the mass of other components in the blended sludge is obtained. These values are summarized in Table 13-12.

$$\begin{matrix}\text{Biodegradable}\\ \text{fraction of TSS}\end{matrix} = \frac{4757 \text{ kg/d}}{9159 \text{ kg/d} \times 1.42 \times 0.68} = 0.54$$

$$\begin{aligned}\begin{matrix}\text{Organic fraction}\\ \text{(TVSS/TSS)}\end{matrix} &= \frac{1}{9159 \text{ kg/d}} [0.74 \times 6227 \text{ kg-TSS/d (stream 3 in Table 13-12)}\\ &\quad + 0.66 \times 2921 \text{ kg/d (Step 5, a)} + 0.80 \times 11 \text{ kg/d}\\ &= 0.72\end{aligned}$$

7. Determine the characteristics of thickened sludge (stream 10).
The thickener performance data are as follows: solids capture efficiency = 85 percent, solids content in thickened sludge = 6 percent TSS, and sp. gr. of thickened sludge = 1.03.
a. Compute the volume of thickened sludge.

$$\begin{aligned}\begin{matrix}\text{TSS in}\\ \text{thickened sludge}\end{matrix} &= 9159 \text{ kg/d (Step 6, c)} \times 0.85 \text{ (capture efficiency)}\\ &= 7785 \text{ kg/d}\\ \text{Volume} &= \frac{7785 \text{ kg/d}}{0.06 \times 1030 \text{ kg/m}^3} = 126 \text{ m}^3\text{/d}\end{aligned}$$

b. Determine BOD_5, Org.-N, NH_4^+-N, NO_3^--N, and TP in the thickened sludge. Assume that there is no conversion of Org.-N to NH_4^+-N in the gravity thickener. The capter rates of BOD_5, Org.-N, and nonprecipitated phosphorus (NPP) are 85 percent. NH_4^+-N is completely soluble. In addition, it is assumed that precipitated phosphorus (PP) is 100 percent captured in the thickened sludge.

With 85 percent capture rate of BOD_5, Org.-N, and NPP, the quantity of these components in the thickened sludge can be calculated from the data in stream 9 in Table 13-12:

BOD_5 = 4757 × 0.85 = 4043 kg/d
Org.-N = 430 × 0.85 = 366 kg/d
NPP = 82 × 0.85 = 70 kg/d
With a 100 percent capture rate, the quantity of PP = 110 kg/d.
TP = NPP + PP = 70 kg/d + 110 kg/d = 180 kg/d.

$$NH_4^+\text{-N} = 4.3 \text{ kg/d (Table 13-12, stream 9)} \times \frac{126 \text{ m}^3\text{/d (Step 7, a)}}{1911 \text{ m}^3\text{/d}} = 0.3 \text{ kg/d}$$

(stream 9, Table 13-12)

NO_3^--N = 0 kg/d (because of complete denitrification)
TN = 366 kg/d + 0 kg/d + 0.3 kg/d = 366 kg/d

Biodegradable fraction of TSS = 0.54 (same as stream 9 in Table 13-12)

Organic fraction (TVSS/TSS) = 0.72 (same as stream 9 in Table 13-12)

8. Determine the characteristics of thickener flow (stream 11).
The volume of thickener overflow and mass of different constituents in the thickener overflow are obtained from thickener influent (stream 9) and thickened sludge (stream 10). These values are provided in Table 13-12.
9. Determine the characteristics of anaerobically digested sludge (stream 12).
The TSS in the thickened sludge are solubilized in the anaerobic digester. A portion of the organic matter is stabilized, producing gases. Also, a portion of Org.-N is solubilized, and some is converted to NH_4^+-N. The following assumptions are made:

BOD_5 stabilization = 60 percent
VSS destruction = 52 percent
conversion of Org.-N into NH_4^+-N = 15 percent
conversion of Org.-N into soluble Org.-N = 10 percent
conversion of NPP into soluble P = 30 percent
PP capture = 100 percent
solids concentration in the digested sludge = 5 percent
sp. gr. of the digested sludge = 1.03
BOD_5 in supernatant= 3000 mg/L
TSS in supernatant = 4000 mg/L

It is also assumed that no liquid volume change occurs in the digester, and TSS ≈ TS after digestion.

a. Determine TSS remaining after digestion.

TVSS = 7785 kg/d (Step 7, a) × 0.72 (Step 7, b) = 5605 kg/d
TVSS stabilized = 5605 kg/d × 0.52 = 2915 kg/d
TVSS remaining = 5605 kg/d − 2915 kg/d = 2690 kg/d
TSS remaining = 7785 kg/d − 2915 kg/d = 4870 kg/d

b. Determine volume and TSS in digested sludge and digester supernatant.

Volume of influent thickened sludge (V_{influent}) = Volume of digested sludge removed from digester (V_{sludge}) + Volume of digester supernatant ($V_{\text{supernatant}}$)

TSS remaining in digested sludge ($W_{\text{remaining}}$) = TSS in digested sludge removed from digester (W_{sludge}) + TSS lost in digester supernatant ($W_{\text{supernatant}}$)

$$V_{\text{influent}} = 126 \text{ m}^3/\text{d (Step 7, a)}$$
$$W_{\text{remaining}} = 4870 \text{ kg/d (Step 9, a)}$$

$$V_{\text{sludge}} = \frac{W_{\text{sludge}}}{0.05 \times 1030}$$

$$V_{\text{supernatant}} = \frac{W_{\text{supernatant}}}{0.004 \times 1000}$$

$$126 \text{ m}^3/\text{d} = \frac{W_{\text{sludge}}}{0.05 \times 1030} + \frac{W_{\text{supernatant}}}{0.004 \times 1000}$$

$$W_{\text{supernatant}} = 4870 \text{ kg/d} - W_{\text{sludge}}$$

Substitute the value of $W_{\text{supernatant}}$ in the above equation and solve for W_{sludge}.
All mass and volume values for digested sludge and supernatant are
Volume of digested sludge removed from digester $V_{\text{sludge}} = 92 \text{ m}^3/\text{d}$
TSS in digested sludge removed from digester $W_{\text{sludge}} = 4734$ kg/d
Volume of digester supernatant $V_{\text{supernatant}} = 34 \text{ m}^3/\text{d}$
TSS in digester supernatant $W_{\text{supernatant}} = 136$ kg/d

c. Determine other components in the digested sludge.

$$\begin{aligned}\text{BOD}_5 &= 4043 \text{ kg/d (Step 7, b)} \times [1 - 0.6 \text{ (stabilized)}] \\ &\quad - 3000 \text{ g/m}^3 \times (34 \text{ m}^3/\text{d}) \times \frac{\text{kg}}{1000 \text{ g}} \\ &= 1515 \text{ kg/d}\end{aligned}$$

The total mass of other components in the digested sludge is obtained by adding the insoluble portion in solids and the soluble portion in liquid.

Org.-N $= 366$ kg/d (Step 7, b) $\times$ [1 − 0.1 (solubilized)]

$$- 0.15 \text{ (converted to } NH_4^+\text{-N)} \times \frac{4734 \text{ kg/d (Step 9, b)}}{4870 \text{ kg/d (Step 9, a)}}$$

$$+ [366 \text{ kg/d (Step 7, b)} \times 0.1 \text{ (solubilized)}] \times \frac{92 \text{ m}^3\text{/d (Step 9, b)}}{126 \text{ m}^3\text{/d (Step 7, a)}}$$

$= 294$ kg/d

NH_4^+-N $= [366$ kg/d (Step 7, b) $\times$ 0.15 + 0.3 kg/d (Step 7, b)]

$$\times \frac{92 \text{ m}^3\text{/d (Step 9, b)}}{126 \text{ m}^3\text{/d (Step 7, a)}} = 40 \text{ kg/d}$$

NO_3-N $= 0$ kg/d (due to complete denitrification)

TN $= 294$ kg/d + 40 kg/d + 0 kg/d = 334 kg/d

NPP in digested sludge removed from digester

$$= 70 \text{ kg/d (Step 7, b)} \times (1 - 0.3) \times \frac{4734 \text{ kg/d (Step 9, b)}}{4870 \text{ kg/d (Step 9, a)}}$$

$$+ 70 \text{ kg/d} \times 0.3 \times \frac{92 \text{ m}^3\text{/d (Step 9, b)}}{126 \text{ m}^3\text{/d (Step 7, a)}}$$

$= 63$ kg/d

PP capture in digested sludge is 100 percent of that in thickened sludge and is equal to 110 kg/d (Step 7, b).

TP $= 63$ kg/d + 110 kg/d = 173 kg/d

Biodegradable fraction of TSS $$= \frac{1515 \text{ kg/d (Step 9, c)}}{4734 \text{ kg/d (Step 9, b)} \times 1.42 \text{ g/g} \times 0.68 \text{ g/g}}$$

$= 0.33$

Organic fraction (TVSS/TSS) $$= \frac{2690 \text{ kg/d (Step 9, a)}}{4870 \text{ kg/d (Step 9, a)}} = 0.55$$

10. Determine the characteristics of digester supernatant (stream 13).

Volume of digested supernatant (stream 13) $= 34$ m^3/d (Step 9, b)

TSS in digested supernatant $= 136$ kg/d (Step 9, b)

$$\text{BOD}_5 = 3000 \text{ g/m}^3 \times (34 \text{ m}^3\text{/d}) \times \frac{\text{kg}}{1000 \text{ g}} = 102 \text{ kg/d}$$

$$\text{Org.-N} = 366 \text{ kg/d (Step 7, b)} \times (1 - 0.1 - 0.15) \times \frac{136 \text{ kg/d (Step 9, b)}}{4870 \text{ kg/d (Step 9, a)}}$$

$$+ (366 \text{ kg/d(Step 7, b)} \times 0.1) \times \frac{34 \text{ m}^3\text{/d (Step 9, b)}}{126 \text{ m}^3\text{/d (Step 7, a)}}$$

$$= 18 \text{ kg/d}$$

$$\text{NH}_4^+\text{-N} = [366 \text{ kg/d (Step 7, b)} \times 0.15 + 0.3 \text{ kg/d (Step 7, b)}]$$

$$\times \frac{34 \text{ m}^3\text{/d (Step 9, b)}}{126 \text{ m}^3\text{/d (Step 7, a)}} = 15 \text{ kg/d}$$

$$\text{NO}_3^-\text{-N} = 0 \text{ kg/d (because of complete denitrification)}$$

$$\text{TN} = 18 \text{ kg/d} + + 0 \text{ kg/d} + 15 \text{ kg/d} = 33 \text{ kg/d}$$

$$\text{NPP} = 70 \text{ kg/d (Step 7, b)} \times (1 - 0.3) \times \frac{136 \text{ kg/d}}{4870 \text{ kg/d}}$$

$$+ 70 \text{ kg/d (Step 7, b)} \times 0.3 \times \frac{34 \text{ m}^3\text{/d (Step 9, b)}}{126 \text{ m}^3\text{/d (Step 7, a)}}$$

$$= 7.0 \text{ kg/d}$$

11. Determine the characteristics of dewatered sludge (stream 14).
The dewatering facility is a belt filter press. Polymers are added for conditioning the digested sludge. The following assumptions are used: TSS capture = 95 percent; organic polymer added = 0.5 percent of TSS on a dry weight basis; capture rate of organic polymers = 80 percent; belt wash water is the plant final effluent, total volume of belt wash water, and chemical dilution water = 35 L/kg TSS in the feed sludge; solids content in sludge cake = 25 percent; sp. gr. of sludge cake = 1.06; sp. gr. of supernatant = 1.00; and no BOD_5 is added by chemical conditioning. It should be noted that belt wash water is applied over the belt at the rollers. Therefore, all wash water goes with the filtrate.
    a. Determine volume and characteristics of belt wash water. The volume of chemical dilution water is small and is ignored.

$$\begin{array}{c}\text{Volume of belt}\\ \text{wash and chemical}\\ \text{dilution water}\end{array} = 4734 \text{ kg/d (Step 9, b)} \times 35 \text{ L/kg} \times \frac{\text{m}^3}{1000 \text{ L}}$$

$$= 166 \text{ m}^3\text{/d}$$

The concentrations of various constituents in the plant effluent (stream 6) are given in Table 13-12. The mass of various constituents in belt wash and chemical dilution water are calculated in proportion to the flow: BOD_5 = 1.7 kg/d,[cc] TSS = 1.7 kg/d, Org.-N = 0.2 kg/d, NH_4^+-N = 0.2 kg/d, NO_3^--N = 1.4 kg/d, TN = 1.7 kg/d, and TP = 0.2 kg/d. It is assumed that complete denitrification occurs in dewatering facility.

b. Calculate TSS in the sludge cake and volume of the sludge cake.

$$\text{TSS removed} = [4734 \text{ kg/d (Step 9, b)}] \times 0.95 = 4497 \text{ kg/d}$$

$$\text{Organic polymer removed} = 4734 \text{ kg/d (Step 9, b)} \times 0.005 \times 0.8 = 19 \text{ kg/d}$$

$$\text{Total TSS} = 4497 \text{ kg/d} + 19 \text{ kg/d} = 4516 \text{ kg/d}$$

$$\text{Volume of sludge cake} = \frac{4516 \text{ kg/d}}{0.25 \times 1060 \text{ kg/m}^3 \text{ (sp. gr.} = 1.06)} = 17 \text{ m}^3\text{/d}$$

c. Calculate the mass of other constituents in the sludge cake.
BOD_5, Org.-N and TP in the sludge cake have a capture rate of 95 percent.

$$\text{Quantity, kg/d} = \left(\begin{array}{c}\text{mass in}\\ \text{digested sludge,}\\ \text{kg/d}\end{array}\right) \times 0.95$$

$$BOD_5 = [1515 \text{ kg/d (Step 9, c)}] \times 0.95 = 1439 \text{ kg/d}$$

Other values are summarized in Table 13-12. A sample calculation for NH_4^+-N in proportion to the total volume of the stream is given below:

$$NH_4^+\text{-N} = [40 \text{ kg/d (Step 9, c)}] \times \frac{17 \text{ m}^3\text{/d (Step 11, b)}}{92 \text{ m}^3\text{/d (Step 9, b)}} = 7.4 \text{ kg/d}$$

$$NO_3^-\text{-N} = 0 \text{ kg/d (caused by complete denitrification)}$$

d. Calculate biodegradable fraction and TVSS/TSS.
The fractions of biodegradable solids and TVSS are the same as those for digested sludge. The effect of polymer is insignificant. TVSS/TSS = 0.55 (Step 9, c).

[cc]BOD_5 in belt wash water $= \dfrac{379 \text{ kg/d (stream 6 in Table 13-12)}}{37882 \text{ m}^3\text{/d (stream 6 in Table 13-12)}} \times 166 \text{ m}^3\text{/d} = 1.7$ kg/d.

12. Calculate the flow and mass of various constituents in the filtrate (stream 15).

The flow and mass of various constituents in filtrate = respective values in [(digested sludge) + (belt wash water) + (Organic polymer remaining) − (sludge cake)]

Volume of filtrate = 92 m^3/d + 166 m^3/d − 17 m^3/d = 241 m^3/d

TSS in filtrate = 4734 kg/d (Step 9, b) + 1.7 kg/d (Step 11, a) + 4734 kg/d (Step 9, b) × 0.005 × (1 − 0.8) − 4497 kg/d (Step 11, b)
= 243 kg/d

Values for other constituents are summarized in Table 13-12.

13. Calculate the characteristics of the combined sidestream (stream 16).
The flow and chemical constituents in stream 16 are obtained by adding the respective values of flow, BOD_5, Org.-N, NH_4^+-N, NO_3^--N, and TP in streams 11, 13, and 15. The flow and TSS calculations are shown below. The values of all other constituents are provided in Table 13-12.

Total volume of combined side stream = 1785 m^3/d (stream 11 in Table 13-12) + 34 m^3/d (Step 9, b) + 241 m^3/d (stream 15 in Table 13-12)
= 2060 m^3/d

TSS in combined side stream = 1374 kg/d (stream 11 in Table 13-12) + 136 kg/d (Step 9, b) + 243 kg/d (Step 12)
= 1753 kg/d

The value of NO_3^--N is considered to be zero because of complete denitrification. The concentrations are calculated from flow and mass data. These values are summarized in Table 13-12.

Step B: Final Results. The previous computational procedure must be repeated for several iterations to obtain the final results. A computer program using an Exel® spreadsheet was developed to generate the final results. This program has the capability to return stream 16 before or after primary sedimenation basin.

Note: The effluent quantity discharged from the plant will be slightly less than the influent flow. This is because of the following factors: (1) evaporation losses, (2) loss of water in production of digester gases, and (3) moisture contained in the sludge cake. In the material mass balance analysis, the loss of water caused by evaporation and gas production has not been included. The flow of effluent is approximately equal to the influent flow (stream 1) plus volume of alum solution (5 m^3/d) minus the volume of sludge cake (stream 14) (see Figure 13-25 and Table 13-13 for reference).

13-11-5 Basic Design Calculations for Reactor Sizing, Biomass Generation, Return Flows, and Air Requirements

The design calculations for the BNR system are given below.

The flow and concentrations of chemical constituents in the influent to the BNR system are estimated on the basis of material mass balance analysis. The final results after final iteration are given in Table 13-13. The influent to the BNR system is characterized as BOD_5 = 180 mg/L, TSS = 137 mg/L, Org.-N = 13.9 mg/L, NH_4^+-N = 19.5 mg/L, NO_3-N = 0 mg/L, TN = 33.4 mg/L, and TP = 5.4 mg/L. The influent flow to the BNR system is 40,050 m^3/d (sidestream 5, Table 13-13).

Because of uncertainties associated with many assumptions used in the material mass balance analysis, it is a common practice to increase these values by 5–10 percent. Therefore, the following values are used for the design of BNR system:

$$\begin{aligned}
\text{Flow} &= 42{,}000 \text{ m}^3\text{/d } (0.486 \text{ m}^3\text{/s}) \\
\text{BOD}_5 &= 200 \text{ mg/L} \\
\text{COD} &= 350 \text{ mg/L (assuming COD:BOD}_5 = 1.75) \\
\text{TSS} &= 150 \text{ mg/L} \\
\text{Org.-N} &= 15 \text{ mg/L} \\
\text{NH}_4^+\text{-N} &= 20 \text{ mg/L} \\
\text{NO}_3\text{-N} &= 0 \text{ mg/L} \\
\text{TN} &= 35 \text{ mg/L} \\
\text{TP} &= 6 \text{ mg/L}
\end{aligned}$$

It is also necessary to consider the following design conditions: total alkalinity in the influent to the BNR system = 200 mg/L as $CaCO_3$, and the sustained winter and summer temperatures in the reactor are 15°C and 24°C, respectively.

Step A: Anaerobic Zone: HRT and Reactor Volume. At the present time, the kinetic equations for phosphorus release and uptake of SCVFAs in the anaerobic zone are not fully developed. The volume requirement for anaerobic zone is therefore calculated from several experimental studies and field observations. A brief discussion on this subject may be found in Sec. 13-7-2. It is assumed that the sludge growth does not occur during phosphorus release. Easily biodegradable organic matter is taken in for energy storage as PHB. The sludge growth occurs when the stored energy as PHB is oxidized for extra phosphorus uptake in the aerobic zone. Therefore, the growth of poly-P organisms related to phosphorus release and uptake is included in the growth of heterotrophic population that carries out BOD_5 stabilization.

TABLE 13-13 *Characteristics of Streams in Final Iteration of Material Mass Balance Analysis*

Stream No.	Flow (m^3/d)	BOD_5	TSS	Org.-N	NH_4^+-N	NO_3^--N	TN	NPP[a]	PP[a]	TP	TVSS/TSS	Biodegradable Solids/TSS	Org.-N/TVSS	NPP/TVSS
		Constituent: kg/d (Values in parentheses are in mg/L)									Fraction of TSS		Fraction of TVSS	
1	38016	9504	9884	646	722	0	1369			228				
		(250)	(260)	(17)	(19)		(36)			(6)				
2	38016	9504	9884	646	722	0	1369			228				
		(250)	(260)	(17)	(19)		(36)			(6)				
3	134	3231	6227	194	2.6	0	197			37	0.74	0.54	0.042	0.008
4	37882	6273	3657	452	720	0	1172			192				
		(166)	(97)	(12)	(19)		(31)			(5)				
5	40050	7210	5498	558	776	0	1333			216				
		(180)	(137)	(13.9)	(19.5)	(0)	(33.4)			(5.4)				
6	39303[b]	393	393	39	39	314	393			39				
		(10)	(10)	(1)	(1)	(8)	(10)			(1)				

7	747	1760	2800	273	0.8	6.0	280			176	0.80	0.65	0.122	0.079
8	752[c]	1760	3385[d]	273	0.8	0	274	50	126	176	0.66	0.54	0.122	0.023
9	2011[e]	5003	9623	468	4.4	0	482	88	126	214	0.71	0.54	0.068	0.013
10	132	4253	8180	398	0.3	0	398	75	126	201	0.71	0.54	0.068	0.013
11	1878	750	1444	70	4.1	0	74	13		13				
		(400)	(768)	(37)	(2.2)		(40)	(7)		(7)				
12	97	1596	5008	320	44	0	364	67	126	193	0.54	0.33	0.12	0.025
13	35	105	140	19	16	0	35	7.4		7.4				
		(3000)	(4000)	(533)	(453)		(986)	(211)		(211)				
14	18	1516	4778	304	8.2	0	312	64	126	190	0.55	0.33	0.12	0.025
15	255[f]	82	257	16	36	0	52	3.6		3.6				
		(320)	(1008)	(63)	(141)	(0)	(204)	(14)		(14)				
16	2168	937	1841	105	56	0	161	24		24				
		(432)	(849)	(48)	(26)	(0)	(77)	(11)		(11)				

[a]NPP = nonprecipitated phosphorus and PP = precipitated phosphorus.
[b]Final plant discharge (38,003 m^3/d) = effluent from final clarifier (39,303 m^3/d) − plant reuse water (1300 m^3/d).
[c]Increase in volume is caused by addition of liquid alum (5 m^3/d).
[d]Increase in TSS (total 585 kg/d) is caused by precipitation of $AlPO_4$ and $Al(OH)_3$.
[e]Increase in volume is caused by addition of dilution water (1125 m^3/d).
[f]Increase in volume is caused by belt wash water (176 m^3/d).

1. Determine the achievable TP concentration in the effluent.

Influent BOD_5 to the BNR system = 200 mg/L

Ratio of COD to BOD_5 = 1.75

Expected influent COD = 1.75 × 200 mg/L = 350 mg/L

TP in the influent to BNR system = 6.0 mg/L

$$\text{Ratio of COD to TP} = \frac{350 \text{ mg/L}}{6.0 \text{ mg/L}} = 58.3$$

The concentration of TP in the effluent is obtained from Figure 13-18. At COD/TP ratio of 58.3, the effluent TP concentration = 0.6 mg/L. It is clear that 1 mg/L TP limit in the effluent is therefore achievable.

2. Determine design HRT of the anaerobic zone, $\theta^{\text{design}}_{\text{anaerobic}}$.
To trigger *ortho*-P release in the anaerobic zone, there should be sufficient SCVFAs in the influent. Since the COD/BOD_5 ratio in influent is less than 2, sufficient SCVFAs are expected to trigger the necessary *ortho*-P release. It is not necessary to provide extra volume for fermentation in the anaerobic zone for generation of SCVFAs. Therefore, a minimum HRT of 1.5 h for the anaerobic zone ($\theta^{\text{design}}_{\text{anaerobic}}$) is provided. This value is generally used for typical municipal wastewater and is sufficient to meet the final TP effluent limit of less than 1 mg/L.

3. Determine the volume of the anaerobic zone, $V_{\text{anaerobic}}$.

$$V_{\text{anaerobic}} = \frac{(1.5 \text{ h}) \times (42000 \text{ m}^3/\text{d})}{24 \text{ h/d}} = 2625 \text{ m}^3$$

Step B: Anoxic Zone: SRT, Sludge Growth, HRT, and Reactor Volume. The SRT, biomass-growth, HRT, and reactor volume in the anoxic zone are calculated from denitrification kinetics (review Secs. 13-2-2 and 13-7-1 for background information).

1. Calculate design SRT for denitrification, $\theta^{\text{design}}_{c,\text{ DN}}$.
The steps involve calculation of specific growth rate and HRT.
   a. Calculate the maximum specific growth rate ($\mu'_{\text{max, DN}}$) of microorganisms carrying out denitrification under field conditions [Eq. (13-49)].

$$\underset{\substack{\text{(maximum}\\\text{specific growth}\\\text{rate under field}\\\text{condition)}}}{\mu'_{\text{max, DN}}} = \underset{\substack{\text{(maximum specific}\\\text{growth rate)}}}{\mu_{\text{max, DN}}} \times \underset{\substack{\text{(factor for}\\\text{temperature}\\\text{correction)}}}{F_{T,\text{ DN}}} \times \underset{\substack{\text{(factor for}\\\text{DO correction)}}}{F_{\text{DO, DN}}} \qquad (13\text{-}49)$$

The value of $\mu_{\text{max, DN}}$ is usually in the range of 0.2–0.4 d^{-1}. A typical value of 0.3 d^{-1} is normally used for municipal wastewater. The temperature and DO correction factors are both dependent upon the field conditions. The temperature correction factor is calculated from Eq. (13-50) at a maximum sustained winter temperature T (°C).

$$F_{T,\ DN} = \theta_{T,\ DN}^{(T-20)} \tag{13-50}$$

The range of $\theta_{T,\ DN}$ values is 1.03–1.20, and the typical value is 1.08.[13] The DO correction factor is calculated from Eq. (13-51):

$$F_{DO,\ DN} = (1 - DO_{max,\ DN}) \tag{13-51}$$

where $DO_{max,\ DN}$ is the maximum reachable DO in the anoxic zone. In most cases, a $DO_{max,\ DN}$ value of 0.10 mg/L is assumed. Therefore,

$$\begin{aligned} \mu'_{max,\ DN} &= (0.30\ d^{-1}) \times 1.08^{(15-20)} \times (1 - 0.10) \\ &= (0.30\ d^{-1}) \times 0.6806 \times 0.90 = 0.184\ d^{-1} \end{aligned}$$

b. Calculate the minimum SRT for denitrification, $\theta_{c,\ DN}^{min}$.
The minimum SRT is calculated from Eq. (13-23).

$$\theta_{c,\ DN}^{min} \approx \frac{1}{\mu'_{max,\ DN} - k_{d,\ DN}}$$

The value of $k_{d,\ DN}$ (endogenous decay coefficient for denitrification microorganisms) is usually in the range of 0.02–0.08 d^{-1}. A typical value of 0.04 d^{-1} is used in this Design Example.

$$\theta_{c,\ DN}^{min} \approx \frac{1}{0.184\ d^{-1} - 0.04\ d^{-1}}$$

$$= 6.9\ d$$

c. Determine the design SRT for denitrification, $\theta_{c,\ DN}^{design}$
The required SRT ($\theta_{c,\ DN}^{required}$) is calculated from Eq. (13-52):

$$\underset{\text{(required SRT)}}{\theta_{c,\ DN}^{required}} = \underset{\text{(safety factor)}}{F_{process,\ DN}} \times \theta_{c,\ DN}^{min} \tag{13-52}$$

$F_{process,\ DN}$ is a safety factor used to give a sufficient safety margin for SRT of the anoxic zone. This is necessary because of many inherent uncertainties in process design and operation. A typical value of 1.5 is used in this example.

$$\theta_{c,\ DN}^{required} = 1.5 \times 6.94\ d = 10.4\ d$$

A conservative value fo 12 d for $\theta_{c,\ DN}^{design}$ is used for the anoxic zone.

2. Calculate sludge growth in the anoxic zone, $p_{x,\ DN}$.
The value of yield coefficient is needed.
a. Calculate design yield coefficient for denitrification, $Y_{obs,\ DN}$ from Eq. (13-20).

$$Y_{obs,\ DN} = \frac{Y_{DN}}{1 + k_{d,\ DN} \times \theta_{c,\ DN}^{design}}$$

The value of Y_{DN} is the maximum cell yield coefficient for denitrification. The value of Y_{DN} is in the range of 0.5–0.7 g VSS per g NO_3^--N denitrified. A typical value of 0.6 is assumed in this case.

$$Y_{obs,\,DN} = \frac{0.6 \text{ g VSS/g } NO_3^-\text{-N}}{1 + [(0.04 \text{ d}^{-1}) \times (12 \text{ d})]}$$

$$= 0.405 \text{ g VSS/g } NO_3^-\text{-N}$$

b. Calculate the increase in cell mass of strict denitrifiers, $p_{x,\,DN}$. The increase in cell mass resulting from the growth of strict denitrifiers is calculated from Eq. (13-53):

$$p_{x,\,DN} = Y_{obs,\,DN} \times \left\{ \begin{pmatrix} \text{TKN} \\ \text{in influent} \end{pmatrix} - \begin{pmatrix} \text{organic nitrogen} \\ \text{fixed into biomass} \end{pmatrix} - \begin{pmatrix} NO_3^-\text{-N} \\ \text{lost in effluent} \end{pmatrix} - \begin{pmatrix} NH_4^+\text{-N + Soluble Org.-N} \\ \text{lost in effluent} \end{pmatrix} \right\} \qquad (13\text{-}53)$$

The organic nitrogen in the biomass is 12.2 percent of TVSS[dd] by weight. The total biomass increase in a BNR system is caused by denitrification, BOD_5 stabilization, and nitrification. The overall population and net increase in the biomass of nitrifiers and denitrifiers in the BNR system are small in comparison to the population of microorganisms that carry out BOD_5 stabilization. Since a single sludge system is used, the total organic nitrogen fixed into the biomass can be approximately estimated from the net growth resulting from overall BOD_5 removal.

The increase in biomass concentration caused by BOD_5 stabilization is calculated from Eq. (13-22), and the procedure is shown in the material mass balance analysis (Sec. 13-11-4, Step A, 4). These steps are repeated below:

Approximate concentration increase in total biomass due to BOD_5 stabilization [Eq. (13-22)][ee]

$$\approx Y_{obs,\,BOD_5} \times \left[\begin{matrix} BOD_5 \\ \text{in influent} \end{matrix} - \begin{matrix} BOD_5 \\ \text{in influent} \end{matrix} \right]$$

$$= (0.349 \text{ g VSS/g } BOD_5) \times [(200 - 3.7) \text{ mg } BOD_5\text{/L}]$$

$$= 68.5 \text{ mg/ VSS/L}$$

Organic nitrogen fixed into biomass

$$= (0.122 \text{ g Org.-N/g VSS}) \times (68.5 \text{ mg VSS/L})$$

$$= 8.4 \text{ mg Org.-N/L}$$

[dd]The nitrogen content of 12.2 percent of VSS is calculated from the general chemical formula $C_{60}H_{87}O_{23}N_{12}P$ for microbial call mass. This value is also used in calculation of material mass balance in Sec. 13-11-4, Step A, 4i.

[ee]The biomass calculated from this relationship gives a higher value. This is because a small portion of BOD_5 is also consumed in deoxygenation of DO in the return sludge and recycled stream. Also, Y_{obs} for denitrification and phosphorus release and uptake are different from that for $Y_{obs,\,BOD_5}$. A more accurate value is given in Step C, 5, A.

Approximately 0.3–0.5 mg/L soluble organic nitrogen and ammonia nitrogen are lost in the effluent. A value of 1.0 mg of soluble Org.-N plus NH_4^+-N is used.

$$p_{x,\,DN} = (0.405\text{ g VSS/g }NO_3^-\text{-N (Step B, 2, a))} \times [35\text{ mg TKN/L} - 8.4\text{ mg Org.-N/L} - 8\text{ mg }NO_3^-\text{-N/L} - 1\text{ mg }(NH_4^+\text{-N} + \text{soluble Org.-N})/\text{L}]$$
$$= 7.1\text{ mg VSS/L}$$

3. Calculate design HRT of anoxic zone, θ_{anoxic}^{design}.

The required HRT of anoxic zone, $\theta_{anoxic}^{required}$, is calculated from Eq. (13-18).

$$\theta_{anoxic}^{required} = \frac{24 \times \theta_{c,\,DN}^{design} \times p_{x,\,DN}}{f_{x,\,DN} \times X}$$

In a BNR system, although the population of strict denitrifiers is small, a much larger population of total heterotrophs is actually capable of carrying out denitrification. Therefore, a factor $f_{x,\,DN}$ is used for this purpose. The $f_{X,\,DN}$ is a fraction of heterotrophic microorganisms that are assumed to carry out denitrification in a mixed culture. A value of $f_{x,\,DN} = 0.50$ would give an indication that 50 percent of the total population of microorganisms in a suspended growth mixed culture is actually carrying out denitrification.

$$\theta_{anoxic}^{required} = \frac{(24\text{ h/d}) \times (12\text{ d}) \times (7.1\text{ mg VSS/L})}{0.5 \times (3000\text{ mg TVSS/L})}$$
$$= 1.4\text{ h}$$

Provide a design HRT of the anoxic zone, $\theta_{anoxic}^{design} = 1.5$ h.[ff] Thus the HRT for the anoxic zone is the same as that for the anaerobic zone.

4. Determine the volume of the anoxic zone, V_{anoxic}.

$$V_{anoxic} = \frac{(1.5\text{ h}) \times (42000\text{ m}^3/\text{d})}{24\text{ h/d}} = 2625\text{ m}^3$$

Step C: Aerobic Zone: SRT, Biomass Growth, and HRT. The volume requirements and sludge growth in the aerobic zone is calculated on the basis of both carbonaceous BOD stabilization and nitrification kinetics (consult Secs. 13-2-2, 13-3-1, and 13-7-1 for additional information).

1. Calculate the design SRT for aerobic cell growth, $\theta_{c,\,aerobic}^{design}$.

In a single-stage, combined BOD_5 stabilization and nitrification, the overall performance of the process is governed by nitrification during sustained winter condition. Therefore, is this example, the design of the aerobic zone is based on the SRT for nitrification ($\theta_{c,\,N}^{design}$). The calculation steps are provided below:

a. Calculate the maximum specific growth rate $\mu'_{max,\,N}$ of nitrifiers under the field condition.

[ff]In a separate stage denitrification facility, the HRT calculations based on kinetics equations give an HRT value around 4 h. Experience has shown that the denitrification zone needed in a BNR system is considerably less (see Table 13-11).

The value of $\mu'_{\max,\,N}$ is calculated from Eq. (13-54).

$$\underset{\substack{\text{(maximum}\\ \text{specific growth}\\ \text{rate under field}\\ \text{condition)}}}{\mu'_{\max,\,N}} = \underset{\substack{\text{(maximum specific}\\ \text{growth rate)}}}{\mu_{\max,\,N}} \times \underset{\substack{\text{(factor for}\\ \text{temperature}\\ \text{correction)}}}{F_{T,\,N}} \times \underset{\substack{\text{(factor for}\\ \text{DO correction)}}}{F_{DO,\,N}}$$

$$\times \underset{\substack{\text{(factor for}\\ \text{pH correction}}}{F_{pH,\,N}} \qquad (13\text{-}54)$$

The value of $\mu_{\max,\,N}$ is in a wide range of 0.3–3.0 d^{-1}. The typical value used in the literature is 0.47 d^{-1}. The correction factors $F_{T,\,N}$, $F_{DO,\,N}$, and $F_{pH,\,N}$ are obtained from Eqs. (13-55), (13-56), and (13-57).

$$F_{T,\,N} = e^{0.098(T-15)} \qquad (13\text{-}55)$$

$$F_{DO,\,N} = \frac{DO_{min,\,N}}{K_{DO,\,N} + DO_{min,\,N}} \qquad (13\text{-}56)$$

$$F_{pH,\,N} = 1 - 0.833 \times (7.2 - pH_{min,\,N}) \qquad (13\text{-}57)$$

The value of $F_{T,\,N}$ is calculated from the sustained winter temperature T of 15°C. Th $K_{DO,\,N}$ is the half velocity constant, and its value is in the range of 1.0–1.3 mg DO/L. A value of $K_{DO,\,N} = 1.0$ mg DO/L is assumed in this example. The $DO_{min,\,N}$ is the minimum allowable concentration of DO in the aerobic zone. A value of $DO_{min,N} = 2.0$ mg/L is normally recommended for the nitrification process.[13,87] The $pH_{min,\,N}$ is the minimum allowable pH value in the aerobic zone for nitrification. It is assumed that a favorable operating pH in the aerobic zone is 7.2.

$$\mu'_{\max,\,N} = (0.47\ d^{-1}) \times e^{0.098(15-15)} \times \frac{2.0\ \text{mg/L}}{1.0\ \text{mg/L} + 2.0\ \text{mg/L}}$$

$$\times [1 - 0.833 \times (7.2 - 7.2)]$$

$$= (0.470\ d^{-1}) \times 1.0 \times 0.6667 \times 1.0 = 0.313\ d^{-1}$$

b. Calculate minimum SRT for nitrification, $\theta^{min}_{c,\,N}$.
The minimum SRT, $\theta^{min}_{c,\,N}$, is calculated from Eq. (13-23).

$$\theta^{min}_{c,\,N} \approx \frac{1}{\mu'_{\max,\,N} - k_{d,\,N}}$$

The value of $k_{d,\,N}$ (endogenous decay coefficient for nitrifiers) is usually in the range of 0.03–0.06 d^{-1} with a typical value of 0.05 d^{-1}. Therefore,

$$\theta_{c,\mathrm{N}}^{\min} \approx \frac{1}{0.313\ \mathrm{d}^{-1} - 0.05\ \mathrm{d}^{-1}}$$
$$= 3.8\ \mathrm{d}$$

c. Determine the design SRT for nitrification, $\theta_{c,\mathrm{N}}^{\mathrm{design}}$.
The required SRT for nitrification under field conditions is calculated from Eq. (13-58):

$$\underset{\text{(required SRT)}}{\theta_{c,\mathrm{N}}^{\mathrm{required}}} = \underset{\substack{\text{(safety factor for}\\ \text{process}\\ \text{considerations)}}}{\mathrm{F}_{\mathrm{process,\,N}}} \times \underset{\substack{\text{(safety factor for}\\ \text{kinetic}\\ \text{considerations)}}}{\mathrm{F}_{\mathrm{kinetic,\,N}}} \times \theta_{c,\mathrm{N}}^{\min} \qquad (13\text{-}58)$$

Because of many uncertainties associated with the nitrification process, two safety factors are used. These ae (a) $F_{\mathrm{process,\,N}}$, a safety factor to account for uncertainty against the process design and operation, and (b) $F_{\mathrm{kinetic,\,N}}$, an extra safety factor to compensate for the sensitivity of nitrification kinetics under the field conditions. The suggested values of these safety factors for both conditions are in the range of 1.5–2.0.[4,11,13] In this example, the assumed values are $F_{\mathrm{process,\,N}} = 1.5$ and $F_{\mathrm{kinetic,\,N}} = 2.0$.

$$\theta_{c,\mathrm{N}}^{\mathrm{required}} = 3.8\ \mathrm{d} \times 1.5 \times 2.0 = 11.4\ \mathrm{d}$$

A 12-d design value of $\theta_{c,\mathrm{N}}^{\mathrm{design}}$ used for the design of the aerobic zone is therefore a conservative value. This value is in conformance with the actual $\theta_{c,\,\mathrm{N}}$ for many BNR systems currently in operation and was assigned in the design criteria (Sec. 13-11-1 Step A, 16).

d. Calculate the achievable concentration of NH_4^+-N in the effluent.
The concentration of NH_4^+-N in the effluent is calculated from the following equations:

$$U_{\mathrm{N}} = \frac{k'_{\mathrm{N}} \cdot \mathrm{NH_4^+\text{-}N}}{K_{\mathrm{N}} + \mathrm{NH_4^+\text{-}N}}$$

$$\frac{1}{\theta_{c,\mathrm{N}}^{\mathrm{required}}} = Y_{\mathrm{N}}\, U_{\mathrm{N}} - k_{d,\mathrm{N}}$$

$$k'_{\mathrm{N}} = \frac{\mu'_{\mathrm{max,\,N}}}{Y_{\mathrm{N}}}$$

$$K_{\mathrm{N}} = 10^{0.051T - 1.158}$$

All terms in the above equations have been defined in several earlier equations and in Ref. 4. The typical value of cell yield coefficients for *Nitrosomonas* and *Nitrobacter* are 0.17 g VSS per gram NH_4^+-N oxidized and 0.02 g VSS per gram NO_2^--N oxidized, respectively.[51] An overall value of 0.2 g VSS per gram NH_4^+-N is used here.

$$k' = \frac{\mu_{max}}{Y_N} = \frac{0.313\ d^{-1}}{0.2\ g\ VSS/g\ NH_4^+\text{-N}} = 1.565\ d^{-1}$$

$$\frac{1}{12\ d} = 0.2\ d^{-1}\ U_N - 0.05\ d^{-1}$$

$$U_N = 0.67\ d^{-1}$$
$$K_N = 10^{0.051 \times 15 - 1.158}$$
$$= 0.405$$

$$0.67\ d^{-1} = \frac{1.565 \times NH_4^+\text{-N}}{0.405 + NH_4^+\text{-N}}$$

$$NH_4^+\text{-N} = 0.3\ mg/L$$

2. Calculate the consumption of BOD_5 in anaerobic and anoxic zones.
   BOD_5 is stabilized in the aerobic zone. In addition, a significant portion of BOD_5 is also utilized in the anaerobic and anoxic zones for phosporus release, denitrification, and deoxygenation of DO in the return and recycle flows.
   a. Calculate the consumption of BOD_5 caused by phosphorus release and uptake.
      Approximately 1.0 mg acetate·COD/L is utilized by the *Acinetobacter* for each milligram per liter of phosphorus released in the anaerobic zone. This biodegradable organic matter is stored as PHB in the cell mass and is eventually stabilized in the aerobic zone for energy and cell growth. Therefore, it is a temporary uptake of BOD_5 in the anaerobic zone and has no net effect until final stabilization.
   b. Calculate the consumption of BOD_5 for denitrification.
      The BOD_5 is also consumed for denitrification in the anaerobic and anoxic zones and can be calculated from Eq. (13-59):

$$\begin{array}{c}(BOD_5\text{ consumed})\\ \text{because of}\\ \text{denitrification}\end{array} = \frac{BOD_5}{BOD_L} \times 3.7\left(\frac{g\ BOD_L}{g\ NO_3^-\text{-N}}\right) \times \begin{array}{c}[(TKN)\\ \text{in influent}\end{array} - \begin{array}{c}(\text{Org.-N})\\ \text{fixed into}\\ \text{biomass}\end{array}$$
$$- \begin{array}{c}(NO_3^-\text{-N})\\ \text{lost in effleunt}\end{array}$$
$$- \begin{array}{c}(NH_4^+\text{-N and Soluble Org.-N})]\\ \text{lost in effluent}\end{array} \tag{13-59}$$

In the above equation, 3.7 g of BOD_L is consumed per gram of NO_3^--N denitrified (Sec. 13-7-1). The consumption of BOD_L for reduction of NO_2^--N is small and is ignored.

$$\begin{array}{c}(BOD_5\text{ consumed})\\ \text{because of}\\ \text{denitrification}\end{array} = \frac{(0.68\ g\ BOD_5)}{g\ BOD_L} \times \frac{(3.7\ g\ BOD_L)}{g\ NO_3^-\text{-N}} \times \{(35\ mg\ TKN/L)$$
$$- (8.4\ mg\ \text{Org.-N/L})^{gg} - (8\ mg\ NO_3^-\text{-N/L})$$
$$- [1\ mg\ (NH_4^+\text{-N and soluble Org.-N})/L]\}$$
$$= 44.3\ mg\ BOD_5/L$$

[gg]8.4 mg Org.-N/L is fixed into the biomass (Sec. 13-11-5 Step B, 2, b).

c. Calculate the consumption of BOD_5 caused by deoxygenation of DO.
The BOD_5 is also consumed because of deoxygenation of DO in both the anaerobic and anoxic zones. In a BNR system, the DO is brought into the anaerobic zone by the return flow and into the anoxic zone by the recycled flow. A small amount of BOD_5 is consumed for deoxygenation of DO in these flows. It is calculated from Eq. (13-60):

$$\begin{pmatrix}\text{BOD}_5\text{ consumed}\\ \text{caused by}\\ \text{deoxygenation}\end{pmatrix} = \frac{\text{BOD}_5}{\text{BOD}_\text{L}} \times 1.3 \times [R_{r,\,\text{recycle}} \times \text{DO}_{\text{max, N}} - (1 + R_{r,\,\text{recycle}}) \times \text{DO}_{\text{max, DN}}] \qquad (13\text{-}60)$$

In the above equation, 1.3 g of BOD_L is consumed to deoxygenate 1 g of DO in the return and recycled flows (Sec. 13-7-1, Denitrification). The $R_{r,\,\text{recycle}}$ is the combined ratio of returned and recycled flows with respect to influent flow. The return flow ratio is 0.6 and is calculated in Sec. 13-11-4, Step 4, k. The recycled flow ratio is 1.7 and is calculated in Sec. 13-11-5, Step F, 3. Therefore, a combined ratio of $R_{r,\,\text{recycle}} = 2.3$. The terms $\text{DO}_{\text{max, N}}$ and $\text{DO}_{\text{max, DN}}$ are the design maximum expected concentrations of DO in the aerobic and anoxic zones. These values are assumed to be 3.0 and 0.1 mg/L, respectively. It should be noted that a minimum DO of 2.0 mg/L is used to design the aerobic reactor (Step C, 1, a).

$$\begin{pmatrix}\text{BOD}_5\text{ consumed}\\ \text{caused by}\\ \text{deoxygenation}\end{pmatrix} = \frac{(0.68\text{ g BOD}_5)}{\text{BOD}_\text{L}} \times \frac{(1.3\text{ g BOD}_\text{L})}{\text{g DO}} \times [(2.3 \times 3.0\text{ mg DO/L}) - (1 + 2.3) \times 0.1\text{ mg DO/L}]$$
$$= 5.8\text{ mg BOD}_5\text{/L}$$

3. Calculate the sludge growth caused by BOD_5 removal in the aerobic zone, p_{x,BOD_5}. Eq. (13-20) is used to calculate the cell yield coefficient.
   a. Calculate the design cell yield coefficient caused by BOD_5 removal, $Y_{\text{obs, BOD}_5}$

$$Y_{\text{obs, BOD}_5} = \frac{Y_{\text{BOD}_5}}{1 + k_{d,\,\text{BOD}_5} \times \theta_{c,\,\text{aerobic}}^{\text{design}}}$$

The Y_{BOD_5} is the maximum cell yield coefficient because of carbonaceous BOD consumption. The value of Y_{BOD_5} is in the range of 0.4–0.8g VSS per g BOD_5 removed. The value of $k_{d,\,\text{BOD}_5}$ (for BOD_5 removal) is usually in the range of 0.025–0.075 d^{-1}. $Y_{\text{BOD}_5} = 0.6$ g VSS per g BOD_5, and $k_{d,\,\text{BOD}_5} = 0.06\ d^{-1}$ is given in the design criteria (Sec. 13-11-1, Step A, 16).

$$Y_{\text{obs, BOD}_5} = \frac{0.6 \text{ g VSS/g BOD}_5}{1 + [(0.06 \text{ d}^{-1}) \times (12 \text{ d})]}$$

$$= 0.349 \text{ g VSS/g BOD}_5$$

b. Calculate the concentration of soluble BOD_5 in the effluent.

$$\begin{matrix}\text{Soluble BOD}_5 \\ \text{in effluent}\end{matrix} = 3.7 \text{ mg BOD}_5\text{/L (Sec. 13-11-4, Step A, 4, c)}$$

c. Calculate the concentration increase in cell mass caused by BOD_5 removal, $p_{x,\text{ BOD}_5}$ (Eq. 13-61):

$$p_{x,\text{ BOD}_5} = Y_{\text{obs, BOD}_5} \times \left\{ \begin{pmatrix}\text{BOD}_5 \\ \text{in influent}\end{pmatrix} - \begin{pmatrix}\text{BOD}_5 \\ \text{consumed due to} \\ \text{phosphorus release}\end{pmatrix}^{hh} - \begin{pmatrix}\text{BOD}_5 \\ \text{consumed due to} \\ \text{denitrification}\end{pmatrix} - \begin{pmatrix}\text{BOD}_5 \\ \text{consumed due to} \\ \text{deoxygenation}\end{pmatrix} - \begin{pmatrix}\text{BOD}_5 \\ \text{in effluent}\end{pmatrix} \right\} \quad (13\text{-}61)$$

$$\begin{aligned} p_{x,\text{ BOD}_5} &= (0.349 \text{ g VSS/g BOD}_5) \\ &\quad \times [(200 - 0 - 44.3 \text{ (Step C, 2, b)} - 5.8 \text{ (Step C, 2, c)} - 3.7 \\ &\quad \text{(Step C, 3, b)) mg BOD}_5\text{/L}] \\ &= 0.349 \text{ g VSS/g BOD}_5 \times 146.18 \text{ mg BOD}_5\text{/L} \\ &= 51.0 \text{ mg VSS/L} \end{aligned}$$

4. Calculate the sludge growth caused by nitrification in the aerobic zone, $p_{X,\text{ N}}$.
   a. Calculate the design cell yield coefficient caused by nitrification, $Y_{\text{obs, N}}$. Eq. (13-20) is used to calculate cell yield coefficient.

$$Y_{\text{obs, N}} = \frac{Y_N}{1 + k_{d,\text{ N}} \times \theta_{c,\text{ aerobic}}^{\text{design}}}$$

The Y_N is the maximum cell yield coefficient caused by nitrification. The value of Y_N is in the range of 0.1–0.3 g of VSS per g of NH_4^+-N nitrified. These values are $Y_N = 0.2$ g of VSS per g of NH_4^+-N nitrified, and $k_{d,\text{ N}} = 0.05 \text{ d}^{-1}$ are given in the design criteria (Sec. 13-11-1, Step A, 16). Therefore,

$$Y_{\text{obs, N}} = \frac{0.2 \text{ g VSS/g NH}_4^+\text{-N}}{1 + [(0.05 \text{ d}^{-1}) \times (12 \text{ d})]}$$

$$= 0.125 \text{ g VSS/g NH}_4^+\text{-N}$$

[hh] BOD_5 uptake for phosphorus release is finally stabilized for energy and cell growth in the aerobic zone. Therefore, the BOD_5 uptake for phosphorus release in the anaerobic zone is temporary and is assumed to be zero. Y_{obs} is calculated in Sec. 13-11-4, Step, A, 4, a.

b. Calculate the increase in concentration of cell mass caused by nitrification, $p_{x,\,N}$ [Eq. (13-62)]:

$$p_{x,\,N} = Y_{obs,\,N} \times \left[\begin{pmatrix} \text{TKN} \\ \text{in influent} \end{pmatrix} - \begin{pmatrix} \text{Org.-N} \\ \text{fixed into biomass} \\ \text{due to} \\ \text{BOD}_5 \text{ removal} \end{pmatrix} - \begin{pmatrix} \text{NH}_4^+\text{-N + soluble Org.-N} \\ \text{lost in effluent} \end{pmatrix} \right] \tag{13-62}$$

$$\begin{aligned} &= (0.125 \text{ g VSS/g NH}_4^+\text{-N}) \times \{(35 \text{ mg TKN/L}) \\ &\quad - (0.122 \text{ g Org.-N/g VSS}) \times (51.0 \text{ mg VSS/L (Step C, 3, c)}) \\ &\quad - [1 \text{ mg (NH}_4^+\text{-N + soluble Org.-N)/L } \} \\ &= 3.5 \text{ mg VSS/L} \end{aligned}$$

5. Calculate the ratio of heterotrophs and nitrifiers in the MLVSS.
Generally the population of nitrifiers is small with respect to the population of heterotrophs.
   a. Calculate the increase in TVSS concentration caused by growth of total cell mass in the mixed culture.
   The mixed culture in a BNR system consists of heterotrophs and autotrophs. The growth of the mixed culture is the result of BOD_5 stabilization, nitrification, and denitrification.

$$\begin{pmatrix} \text{Increase in} \\ \text{TVSS} \\ \text{concentration} \\ \text{caused by} \\ \text{total cell} \\ \text{mass} \\ (p_{x,\,TVSS}) \end{pmatrix} = \begin{pmatrix} \text{Increase in} \\ \text{TVSS} \\ \text{concentration} \\ \text{caused by} \\ \text{BOD}_5 \\ \text{stabilization} \\ (p_{x,\,BOD_5}) \end{pmatrix} + \begin{pmatrix} \text{Increase in} \\ \text{TVSS} \\ \text{concentration} \\ \text{caused by} \\ \text{nitrification} \\ (p_{x,\,N}) \end{pmatrix} + \begin{pmatrix} \text{Increase in} \\ \text{TVSS} \\ \text{concentration} \\ \text{caused by} \\ \text{denitrification} \\ (p_{x,\,DN}) \end{pmatrix}$$

   In the above equation, $p_{x,\,BOD_5} = 51.0$ mg VSS/L (Step C, 3, c), $p_{x,\,N} = 3.5$ mg VSS/L (Step C, 4, b), and $p_{x,\,DN} = 7.1$ mg VSS/L (Step B, 2, b).

$$\begin{aligned} p_{x,\,TVSS} &= (51.0 + 3.5 + 7.1) \text{ mg VSS/L} \\ &= 61.6 \text{ mg VSS/L} \end{aligned}$$

   b. Calculate the increase in TVSS concentration caused by growth of heterotrophs. In the mixed culture, the population of heterotrophic organisms consists of biomass that carrries out BOD_5 stabilization and denitrification.

$$\begin{aligned} \begin{pmatrix} \text{Increase in} \\ \text{TVSS} \\ \text{concentration} \\ \text{due to} \\ \text{heterotrophs} \end{pmatrix} &= p_{x,\,BOD_5} + p_{x,\,DN} = (51.0 + 7.1) \text{ mg VSS/L} \\ &= 58.1 \text{ mg VSS/L} \end{aligned}$$

c. Calculate the fraction of heterotrophs and nitrifiers in the MLVSS.

$$\begin{matrix}\text{Fraction}\\ \text{of heterotrophs}\\ \text{in the mixed culture}\end{matrix} = \frac{58.1 \text{ mg VSS/L}}{61.6 \text{ mg VSS/L}} = 0.94$$

$$\begin{matrix}\text{Fractions}\\ \text{of autotrophs}\\ \text{in the mixed culture}\end{matrix} = \frac{3.5 \text{ mg VSS/L}}{61.6 \text{ mg VSS/L}} = 0.06$$

6. Calculate design HRT of the aerobic zone, $\theta_{\text{aerobic}}^{\text{design}}$.
The HRT of the aerobic zone is calculated from the requirements for both BOD_5 removal and for nitrification. The larger of two values governs the design.
a. Calculate from Eq. (13-18) the required HRT for BOD_5 removal, $\theta_{\text{aerobic,BOD}_5}^{\text{required}}$.

$$\theta_{\text{aerobic, BOD}_5}^{\text{required}} = \frac{24 \times \theta_{c,\text{ aerobic}}^{\text{design}} \times p_{x,\text{ BOD}_5}}{\begin{pmatrix}\text{fraction}\\ \text{of heterotrophies}\end{pmatrix} \times X}$$

The fraction of heterotrophic microorganisms that carry out BOD_5 removal in the mixed culture is 0.94.

The MLVSS concentration X maintained in the aerobic zone is 3000 mg TVSS/L (see Sec. 13-11-1, Step A, 16).

$$\theta_{\text{aerobic,BOD}_5}^{\text{required}} = \frac{(24 \text{ h/d}) \times (12 \text{ d}) \times (51.0 \text{ mg VSS/L})}{0.94 \times (3000 \text{ mg TVSS/L})}$$

$$= 5.2 \text{ h}$$

b. Calculate the required HRT of the aerobic zone for nitrification, $\theta_{\text{aerobic,N}}^{\text{required}}$

$$\theta_{\text{anoxic,N}}^{\text{required}} = \frac{(24 \text{ h/d}) \times (12\text{d}) \times (3.5 \text{ mg VSS/L})}{0.06 \times (3000 \text{ TVSS/L})}$$

$$= 5.6 \text{ h}$$

The required HRT for nitrification ($\theta_{\text{aerobic,N}}^{\text{required}}$) governs the design. Use a conservative HRT value of 6 h for design of the aerobic zone.

c. Determine the volume of aerobic zone, V_{aerobic}

$$V_{\text{aerobic}} = \frac{(6 \text{ h}) \times (42{,}000 \text{ m}^3/\text{d})}{24 \text{ h/d}} = 10{,}500 \text{ m}^3$$

Step D: Quantity of Waste Activated Sludge (WAS)

1. Calculate the increase in total quantity of biomass.
The calculation has been shown that the increase in TVSS concentration caused by growth of total cell mass in the mixed culture, $p_{x,\text{ TVSS}} = 61.6$ mg TVSS/L (Step C, 5, a).

$$\begin{array}{c}\text{Increase}\\ \text{in total}\\ \text{quantity}\\ \text{of MLVSS}\end{array} = (61.6 \text{ g TVSS/m}^3) \times (42{,}000 \text{ m}^3\text{/d}) \times (10^{-3} \text{ kg/g})$$

$$= 2587 \text{ kg TVSS/d}$$

$$\begin{array}{c}\text{Increase}\\ \text{in total}\\ \text{quantity}\\ \text{of MLSS}\end{array} = \frac{2587 \text{ kg TVSS/d}}{0.8 \text{ g TVSS/g TSS}^{ii}} = 3234 \text{ kg TSS/d}$$

2. Calculate the volume of WAS.
A portion of solids is lost in the effluent. The remaining solids are wasted from the aeration basin. The flow, Q_{WAS}, and weight of WAS are calculated below:

$$\begin{array}{c}\text{Increase}\\ \text{in total}\\ \text{MLSS}\end{array} = \begin{array}{c}\text{TSS}\\ \text{lost}\\ \text{in effluent}\end{array} + \text{TSS wasted in WAS}$$

$$3234 \text{ kg TSS/d} = \left[(42{,}000 \text{ m}^3\text{/d} - Q_{WAS}) \times (10 \text{ g TSS/m}^3) + Q_{WAS} \times \left(\frac{3000 \text{ g TSS/m}^3}{0.8 \text{ g TVSS/g TSS}}\right)\right] \times (10^{-3} \text{ kg/g})$$

Solving the above equation

$$Q_{WAS} = 752 \text{ m}^3\text{/d}$$

3. Calculate the volume of effluent discharged.

$$\begin{aligned}\text{The effluent flow rate} &= 42{,}000 \text{ m}^3\text{/d} - 752 \text{ m}^3\text{/d}\\ &= 41{,}248 \text{ m}^3\text{/d}\end{aligned}$$

4. Calculate the quantity of TSS in WAS and in effluent.

$$\begin{array}{c}\text{Total TSS}\\ \text{in WAS}\end{array} = (752 \text{ m}^3\text{/d}) \times \left(\frac{3000 \text{ g TSS/m}^3}{0.8 \text{ g TVSS/g TSS}}\right) \times (10^{-3} \text{ kg/g})$$

$$= 2820 \text{ kg TSS/d}$$

$$\text{TSS in effluent} = \frac{(42{,}000 - 752) \text{ m}^3\text{/d} \times 10 \text{ g/m}^3}{1000 \text{ g/kg}}$$

$$= 413 \text{ kg/d}$$

[ii]MLVSS/MLSS = 0.8 is given in the design criteria (see Sec. 13-11-1, Step A, 16).

5. Calculate the BOD_5 content of WAS.

$$\text{Total } BOD_5 = \frac{2820 \text{ kg TSS}}{\text{d}} \times \frac{0.65 \text{ g biodegradable solids}}{\text{g TSS}} \times \frac{1.42 \text{ g } BOD_L}{\text{g biodegradable solids}} \times \frac{0.68\ BOD_5}{BOD_L}$$

$$= 1770 \text{ kg/d}$$

6. Calculate the quantity of nitrogen in WAS.
Assuming the organic nitrogen in biomass is 12.2 percent of TVSS,[jj] the total quantity of organic nitrogen in the WAS is

$$\begin{aligned}\text{Org.-N in WAS} &= (2820 \text{ kg TSS/d}) \times (0.8 \text{ g TVSS/g TSS}) \\ &\quad \times (0.122 \text{ g Org.-N/g TVSS}) \\ &= 275 \text{ kg Org.-N/d}\end{aligned}$$

$$\begin{aligned}NH_4^+{-}N \text{ in WAS} &= (752 \text{ m}^3\text{/d}) \times [1 \text{ g } (NH_4^+\text{-N and soluble Org.-N})/\text{m}^3] \\ &\quad \times (10^{-3} \text{ kg/g}) \\ &= 0.75 \text{ kg } (NH_4^+\text{-N and soluble Org.-N})/\text{d}\end{aligned}$$

$$\begin{aligned}NO_3^-\text{-N in WAS} &= (752 \text{ m}^3\text{/d}) \times (8 \text{ g } NO_3^-\text{-N/m}^3) \times (10^{-3} \text{ kg/g}) \\ &= 6 \text{ kg } NO_3^-\text{-N/d}\end{aligned}$$

$$\begin{aligned}\text{TN in WAS} &= \text{Org.-N in WAS} + NH_4^+\text{-N in WAS} + NO_3^-\text{-N in WAS} \\ &= (275 \text{ kg Org.-N/d}) \\ &\quad + [0.75 \text{ kg } (NH_4^+\text{-N and soluble Org.-N})/\text{d}] \\ &\quad + (6 \text{ kg } NO_3^-\text{-N/d}) \\ &= 282 \text{ kg TN/d}\end{aligned}$$

7. Calculate the quantity of phosphorus in the WAS.
The total phosphorus in the WAS consists of organic phosphorus fixed into the sludge and soluble PO_4^{3-}-P in the liquid. The concentration of soluble PO_4^{3-}-P is small in comparison with that of organic phosphorus and is ignored. The total quantity of organic phosphorus in the WAS is calculated below:

$$\begin{aligned}\text{Org.-P in WAS} &= \text{TP in influent} - \text{TP in efluent} \\ &= [(42{,}000 \text{ m}^3\text{/d}) \times (6 \text{ g TP/m}^3) \\ &\quad - (41{,}248 \text{ m}^3\text{/d}) \times (1 \text{ g TP/m}^3)] \times (10^{-3} \text{ kg/g}) \\ &= 211 \text{ kg Org.-P/d}\end{aligned}$$

[jj]See results of stream 7 after final iteration of mass balance analysis (Table 13-13).

$$\begin{array}{l}\text{Org.-P} \\ \text{content} \\ \text{in WAS}\end{array} = \frac{211\text{ kg Org.-P/d}}{(2820\text{ kg TSS/d}) \times (0.8\text{ g TVSS/g TSS})} \times 100\%$$

$$= 9.4\%\text{ of TVSS}$$

8. Summarize the characteristics of WAS.
   The characteristics of WAS developed in this section are summarized below. These values are very close to those calculated in the material mass balance analysis (Table 13-13).

$$\text{Flow} = 752\text{ m}^3\text{/d}$$

$$\text{TSS} = 2820\text{ kg/d}$$

$$\text{BOD}_5 = 1770\text{ kg/d}$$

$$\text{Org.-N} = 275\text{ kg/d}$$

$$\text{NH}_4^+\text{-N} = 0.75\text{ kg/d}$$

$$\text{NO}_3^-\text{-N} = 6\text{ kg/d}$$

$$\text{Total N} = 282\text{ kg/d}$$

$$\text{Total P} = 211\text{ kg/d}$$

$$\text{Concentration of total P} = 9.4\text{ percent of TVSS}$$

Step E: Stripping of Phosphorus from WAS. The waste activated sludge is rich in phosphorus. A significant portion of the phosphorus will release from the cell under anaerobic conditions. In order to reduce the phosphorus buildup in the BNR system through sidestreams, an anaerobic phosphorus stripper is provided. In this unit, the PO_4^{3-} ions are released under anaerobic conditions, and alum precipitation of PO_4-P as $AlPO_4$ is achieved (see Sec. 13-11-4, Step A, 5). It may be noted that in an anaerobic phosphorus stripper, a carbon source may be needed for complete release of phosphorus. Therefore, provision should be made to pump thickener overflow, or primary sludge.

1. Calculate the quantity of phosphorus released in the anaerobic phosphorus stripper.
   The amount of phosphorus in WAS is 211 kg/d (Step D, 7). Assume that the phosphorus content in the biomass is 2.3 percent of VSS[kk] after P-stripping.

$$\begin{array}{c}\text{PO}_4\text{-P released} \\ \text{in P-stripper}\end{array} = (211\text{ kg Org.-P/d}) - (0.023\text{ g-P/TVSS})$$

$$\times (2820\text{ kg-TSS/d (Step D, 4)}) \times (0.8\text{ TVSS/TSS})$$

$$= 159\text{ kg PO}_4\text{-P/d}$$

[kk]The phosphorus content is approximately 2.3 percent of TVSS in the mixed culture, expressed by the chemical formula $C_{60}H_{87}O_{32}N_{12}P$.

2. Calculate the volume of coagulant required for P-stripping and the design capacity of the chemical feed system.
The liquid alum is provided as a coagulant. A molar ratio of 2.5[ll] for Al^{3+} (applied)/P (released) is used. It is also assumed that the $Al_2(SO_4)_3$ content in the liquid alum is 25 percent, and the specific gravity of liquid alum is 1.3.

$$\begin{aligned}\text{Total quantity of } Al^{3+} \text{ required} &= \frac{PO_4\text{-P released}}{\text{molar wt. of P}} \times Al^{3+}/P \text{ molar ratio} \\ &\quad \times \text{molar wt. of Al} \\ &= \frac{159 \text{ kg } PO_4\text{-P/d}}{31 \text{ g/mole}} \times (2.5 \text{ mole Al/mole P}) \times (27 \text{ g/mole}) \\ &= 346 \text{ kg } Al^{3+}/\text{d}\end{aligned}$$

$$\begin{aligned}\text{Volume of liquid alum required} &= \frac{\text{quantity of } Al^{3+} \text{ required} \times \text{molar wt. of } Al_2(SO_4)_3}{2 \times \text{molar wt. of Al} \times 0.25 \times 1300 \text{ kg/m}^3} \\ &= \frac{346 \text{ kg/d} \times 342}{2 \times 27 \times 0.25 \times 1300} \\ &= 6.7 \text{ m}^3/\text{d}\end{aligned}$$

Provide the design capacity of the chemical feed system at 150 percent of the required volume.

$$\text{Design capacity of alum feeding system} = 1.5 \times 6.7 \text{ m}^3/\text{d} \approx 10 \text{ m}^3/\text{d}$$

3. Calculate the total volume of sludge after P-stripping.
The total volume of WAS after P-stripping is that of the raw WAS plus the liquid alum applied.

$$\text{Total volume of P-stripped WAS} = 752 \text{ m}^3/\text{d} + 6.7 \text{ m}^3/\text{d} = 759 \text{ m}^3/\text{d}$$

4. Calculate the increase in TSS after P-stripping.
At a Al^{3+}/P molar ratio of 2.5, over 98 percent of released PO_4-P will be precipitated as $AlPO_4$.[88] It is assumed that 100 percent of soluble phosphorus released is precipitated.

$$\text{Quantity of } PO_4\text{-P fixed in } AlPO_4 = \text{Quantity of } PO_4\text{-P released in P-stripper} = 159 \text{ kg } PO_4\text{-P/d (Step E, 1)}$$

[ll]The mole ratio of 2.5 has an excess amount of alum in solution.

The net increase in TSS is the results of precipitation of $AlPO_4$ and $Al(OH)_3$.

a. Calculate the increase in the quantity of TSS caused by precipitation of $AlPO_4$. PO_4 is released from the cellular mass; therefore, there is a reduction in cell mass equivalent of the quantity of PO_4 released. Its precipitation would not cause any net increase in TSS. The only increase in TSS will be caused by the fixation of Al^{3+} in $AlPO_4$.

$$\begin{array}{c}\text{Increase in TSS}\\ \text{caused by}\\ \text{precipitation}\\ \text{of AlPO}_4{}^{\text{mm}}\end{array} = \frac{\text{molar wt. of Al}^{3+}}{\text{molar wt. of P}} \times \begin{array}{l}\text{Quantity of}\\ \text{PO}_4\text{-P fixed}\\ \text{in AlPO}_4\end{array}$$

$$= \frac{27}{31} \times (159 \text{ kg PO}_4\text{-P/d, Step E,1})$$

$$= 138 \text{ kg TSS/d}$$

b. Calculate the increase in TSS caused by precipitation of $Al(OH)_3$. In addition to precipitation of $AlPO_4$, the excess alum will react with alkalinity to produce $Al(OH)_3$.[nn]

$$\begin{array}{c}\text{Increase in TSS}\\ \text{caused by}\\ \text{precipitation}\\ \text{of Al(OH)}_3{}^{\text{oo}}\end{array} = \frac{\text{molar wt. of Al(OH)}_3}{\text{molar wt. of P}} \times \begin{array}{l}\text{Quantity of}\\ \text{PO}_4\text{-P released}\\ \text{in P-stripper}\end{array}$$

$$\times (\text{Al}^{3+}/\text{P molar ratio} - 1)$$

$$= \frac{78}{31} \times (159 \text{ kg PO}_4\text{-P/d}) \times (2.5 - 1)$$

$$= 600 \text{ kg TSS/d}$$

c. Calculate the increase in total quantity of TSS after P-stripping.

$$\begin{array}{c}\text{Total increase}\\ \text{in TSS}\\ \text{after P-stripping}\end{array} = 138 \text{ kg TSS/d} + 600 \text{ kg TSS/d}$$

$$= 738 \text{ kg TSS/d}$$

[mm]A simplified chemical reaction is $Al_2(SO_4)_3 + 2\ PO_4^{3-} \rightarrow 2\ AlPO_4\downarrow + 3\ SO_4^{2-}\downarrow$.

[nn]A simplified chemical reaction is $Al_2(SO_4)_3 + 3\ Ca(HCO_3)_2 \rightarrow 2\ Al(OH)_3\downarrow + 3\ CaSO_4 + 6\ CO_2$.

[oo]1 g mole P consumes 1 g mole Al^{3+}, which precipitates as $AlPO_4$. Also, 1 g mole Al^{3+} produces 1 g mole $Al(OH)_3$.

5. Calculate the total quantity of TSS in WAS after P-stripping.

$$\begin{aligned}\text{Total quantity of TSS in WAS after P-stripping} &= 2820 \text{ kg-TSS/d (Step D, 4)} \\ &\quad + 738 \text{ kg TSS/d (Step E, 4, c)} \\ &= 3558 \text{ kg TSS/d}\end{aligned}$$

Step F: Estimation of the Return Sludge and Internal Recycle Rates

1. Calculate the return sludge flow Q_r into the anaerobic zone.
   The return flow Q_r is calculated from the mass balance analysis. This analysis was performed in Sec. 13-11-4, Step A, 4, k. This step is not repeated here. The required ratio of Q_r to Q is 0.60.

$$Q_r = 0.60 \times (42{,}000 \text{ m}^3\text{/d}) = 25{,}200 \text{ m}^3\text{/d} = 0.292 \text{ m}^3\text{/s}$$

2. Calculate the quantity of NO_3^--N lost by denitrification.

$$\begin{aligned}\begin{matrix}\text{Loss of} \\ NO_3^-\text{-N} \\ \text{by denitrification}\end{matrix} &= \begin{matrix}\text{Quantity} \\ \text{of TKN} \\ \text{in influent}\end{matrix} - \begin{matrix}\text{Quantity} \\ \text{of TN} \\ \text{in WAS}\end{matrix} - \begin{matrix}\text{Quantity} \\ \text{of TN} \\ \text{in effluent}\end{matrix} \\ &= (35 \text{ g TKN/m}^3) \times (42{,}000 \text{ m}^3\text{/d}) \times (10^{-3} \text{ kg/g}) \\ &\quad - 282 \text{ kg TN/d}^{pp} - (10 \text{ g TN/m}^3)^{qq} \times (41{,}248 \text{ m}^3\text{/d}) \\ &\quad \times (10^{-3} \text{ kg/g}) \\ &= 776 \text{ kg } NO_3^-\text{-N denitrified/d}\end{aligned}$$

3. Calculate the recycle flow $Q_{recycle}$ in the anoxic zone.
   Total quantity of NO_3^--N denitrified is equal to the sum of the quantities of NO_3^--N in the return sludge and NO_3^--N in the recycle flow.

$$\underset{\text{(denitrified)}}{776 \text{ kg } NO_3^-\text{-N/d}} = (8 \text{ g } NO_3^-\text{-N/m}^3) \times [(25{,}200 \text{ m}^3\text{/d}) + Q_{recycle}] \times (10^{-3} \text{ kg/g})$$

   Solving the above equation for $Q_{recycle}$:

$$Q_{recycle} = 71{,}800 \text{ m}^3\text{/d} = 0.831 \text{ m}^3\text{/s}$$

   The recycle ratio is

$$\frac{Q_{recycle}}{Q} = \frac{71{,}800 \text{ m}^3\text{/d}}{42{,}000 \text{ m}^3\text{/d}} = 1.7$$

[pp]TN in the WAS is 282 kg/d (Sec. 13-11-5, Step D, 6).

[qq]NO_3^--N = 8 mg/L, Org.N in effluent solid = 1 mg/L, and NH_4^+-N + soluble Org.N = 1 mg/L.

4. Calculate the $(Q_r + Q_{\text{recycle}})$ ratio.

$$\frac{Q_r + Q_{\text{recycle}}}{Q} = \frac{Q_r}{Q} + \frac{Q_{\text{recycle}}}{Q}$$
$$= 0.6 + 1.7$$
$$= 2.3$$

Step G: F/M Ratio and Organic Loading and Alkalinity Remaining

1. Check *F/M* ratio of the BNR system.
   The overall *F/M* ratio of the BNR system is checked on the basis of the total mixed culture in the entire system. The concentration of BOD_5 in the influent to the BNR system is 200 mg/L. Total HRT of the BNR system is $\theta_{BNR} = 9$ h.[rr] It is assumed that the overall weighted average concentration of MLVSS in three reactors is 3000 mg VSS/L. Therefore, the *F/M* ratio can be calculated using Eq. (13-23):

$$F/M = \frac{Q\,S_o}{V\,X} = \frac{S_o}{\theta_{BNR}\,X}$$
$$= \frac{(24\text{ h/d}) \times 200\text{ g BOD}_5/\text{m}^3 \times (10^3\text{ g/kg})}{9.0\text{ h} \times 3000\text{ g VSS/m}^3 \times (10^3\text{ g/kg})}$$
$$= 0.18\text{ kg BOD}_5/\text{kg VSS·d } (0.18\text{ lb BOD}_5/\text{lb VSS·d})$$

2. Check the organic loading rate (OLR) in the BNR system.
   The overall organic loading (kg BOD_5/m^3 of reactor volume) is checked on the basis of total influent BOD_5 and total reactor volume.

$$\text{OLR} = \frac{S_o\,Q}{\text{Total volume}} = \frac{S_o}{\theta_{BNR}}$$
$$= \frac{(24\text{ h/d}) \times (200\text{ g BOD}_5/\text{m}^3)}{(9\text{ h}) \times (10^3\text{ g/kg})}$$
$$= 0.53\text{ kg BOD}_5/\text{m}^3\text{·d } (33.1\text{ lb BOD}_5/10^3\text{ ft}^3\text{·d})$$

3. Check alkalinity.
   a. Calculate the alkalinity recovered because of denitrification in anaerobic and anoxic zones.

[rr] $\theta_{BNR} = 1.5$ h (anaerobic zone) + 1.5 h (anoxic zone) + 6 h (aerobic zone) = 9 h (see Sec. 13-11-5, Step A, 2, Step B, 3, and Step C, 6, b).

$$\begin{array}{c}\text{Recovery of}\\ \text{alkalinity caused by}\\ \text{denitrification}\end{array} = 3.57^{ss} \times \left[\begin{array}{c}\text{TKN}\\ \text{in influent}\end{array} - \begin{array}{c}\text{Org.-N}\\ \text{fixed into biomass}\end{array} - \begin{array}{c}NO_3^-\text{-N}\\ \text{in effluent}\end{array} - \begin{array}{c}(NH_4^+\text{-N + soluble Org.-N})\\ \text{in effluent}\end{array}\right]$$

$$\begin{aligned} &= (3.57 \text{ mg } CaCO_3/\text{mg } NO_3^-\text{-N denitrified})\\ &\quad \times \{(35 \text{ mg TKN/L}) - (0.122 \text{ g Org.-N/g VSS})\\ &\quad \times (61.6 \text{ mg VSS/L}^{tt}) - (8 \text{ mg } NO_3^-\text{-N/L})\\ &\quad - [1 \text{ g } (NH_4^+\text{-N + soluble Org.-N})/m^3]\}\\ &= 66 \text{ mg } CaCO_3/L \end{aligned}$$

b. Calculate the alkalinity destroyed by nitrification in aerobic zone.

$$\begin{array}{c}\text{Alkalinity}\\ \text{destroyed by}\\ \text{nitrification}\end{array} = 7.14^{uu} \times \left[\begin{array}{c}\text{TKN}\\ \text{in influent}\end{array} - \begin{array}{c}\text{Org.-N}\\ \text{fixed into biomass}\end{array} - \begin{array}{c}(NH_4^+\text{-N + soluble Org.-N})\\ \text{in effluent}\end{array}\right]$$

$$\begin{aligned} &= (7.14 \text{ mg } CaCO_3/\text{mg } NH_4^+\text{-N nitrified})\\ &\quad \times \{(35 \text{ mg TKN/L}) - (0.122 \text{ g Org.-N/g VSS})\\ &\quad \times (61.6 \text{ mg VSS/L}^{vv})\\ &\quad - [1 \text{ g } (NH_4^+\text{-N+ soluble Org.-N})/m^3)]\}\\ &= 189 \text{ mg } CaCO_3/L \end{aligned}$$

c. Calculate the alkalinity remaining in the effluent from the BNR system. Total alkalinity in the influent is 200 mg $CaCO_3$/L. If the increase in alkalinity because of BOD_5 stabilization is ignored, the alkalinity remaining in the effluent will be

$$\begin{array}{c}\text{Alkalinity}\\ \text{remaining}\\ \text{in effluent}\end{array} = (200 + 66 - 189) \text{ mg } CaCO_3/L = 77 \text{ mg } CaCO_3/L$$

The remaining alkalinity of 77 mg/L as $CaCO_3$ is sufficient to maintain the desirable buffer, and the pH will remain in the desirable range of around 7.2.

Step H: Theoretical Oxygen Requirement (ThOR) in the Aerobic Zone. The theoretical oxygen requirement (ThOR) in a BNR system consists of ultimate carbonaceous BOD ($CBOD_L$) and ultimate nitrogenous BOD ($NBOD_L$).

[ss]3.57 g alkalinity as $CaCO_3$ is produced when 1 g of NO_3^--N is reduced (see Sec. 13-7-1).

[tt]The net TVSS concentration increase in biomass is 61.6 mg VSS/L (Sec. 13-11-5, Step C, 5, a).

[uu]7.14 g alkalinity as $CaCO_3$ is consumed when 1 g of NH_4^+-N is oxidized (see Sec. 13-7-1).

[vv]Total TVSS increase in BNR (Sec. 13-11-5, Step C, 5, a).

1. Calculate the oxygen requirement for $CBOD_L$.

The generalized mass balance expression to calculate the oxygen requirement for $CBOD_L$ removal and explanation of important terms are given below:

$$\begin{array}{c}\text{Oxygen}\\ \text{requirement}\\ \text{for } CBOD_L\\ \text{removal}\\ (\text{mg } O_2/\text{L})\end{array} = \begin{bmatrix}BOD_L\\ \text{in influent}\\ (\text{mg } BOD_L/\text{L})\end{bmatrix} - \begin{bmatrix}\text{Soluble } BOD_L\\ \text{in effluent}\\ (\text{mg } BOD_L/\text{L})\end{bmatrix} - \begin{bmatrix}BOD_L \text{ fixed}\\ \text{in cell mass}\\ (\text{mg } BOD_L/\text{L})\end{bmatrix}$$

$$- \begin{bmatrix}BOD_L \text{ consumed}\\ \text{by denitrification}\\ (\text{mg } BOD_L/\text{L})\end{bmatrix} + \begin{bmatrix}BOD_L \text{ consumed}\\ \text{due to deoxygenation}\\ \text{of return and recycle flows}\\ (\text{mg } BOD_L/\text{L})\end{bmatrix}$$

The BOD_L in the influent is calculated from BOD_5 concentration in the influent to the BNR facility. The soluble BOD_5 in the effluent is 3.7 mg/L (Step C, 3, b). The BOD_L fixed in the cell mass is calculated from the increase in cell mass resulting from BOD_5 stabilization, nitrification, and denitrification. The total increase in cell mass because of these growths is 61.6 mg VSS/L (Step C, 5, a). The BOD_5 consumed by denitrification is 44.3 mg BOD_5/L (Step C, 2, b). The combined ratio of return and recycle flows is 2.3 (Step F, 4). The operating DO in the aerobic zone is 2.0 mg DO/L (Step C, 1, a). The maximum DO in the anoxic zone is 0.1 mg DO/L (Step B, 1, a). The ratio of BOD_L consumed for deoxygenation of DO is 1.3 g of BOD_L per gram of DO (Sec. 13-7-1).

$$\begin{array}{c}\text{Oxygen}\\ \text{requirement}\\ \text{for } CBOD_L\\ \text{removal}\end{array} = \left[\frac{200 \text{ mg } BOD_5/\text{L}}{0.68 \text{ g } BOD_5/\text{g } BOD_L}\right] - \left[\frac{3.7 \text{ mg } BOD_5/\text{L}}{0.68 \text{ g } BOD_5/\text{g } BOD_L}\right]$$

$$- \left[(0.65 \text{ g biodegradable solids/g TSS}) \times \left(\frac{61.6 \text{ mg VSS/L}}{0.8 \text{ g VSS/g TSS}}\right)\right.$$

$$\left.\times (1.42 \text{ g } BOD_L/\text{g biodegradable solids})\right] - \left[\frac{44.3 \text{ mg } BOD_5/\text{L}}{0.68 \text{ g } BOD_5/\text{g } BOD_L}\right]$$

$$+ (1.3 \text{ g } BOD_L/\text{g DO}) \times [(2.3 \times 2.0 \text{ mg DO/L}) - (1 + 2.3)^{ww} \times 0.1 \text{ mg DO/L}]$$

$$= (294 - 5.5 - 71.1 - 65.1 + 5.6) \text{ mg } O_2/\text{L}$$

$$= 157.9 \text{ mg } O_2/\text{L}$$

2. Calculate the oxygen requirement for $NBOD_L$ removal.

[ww]The entire flow leaving the anoxic reactor is $Q + Q_r + Q_{recycle}$, which is equal to (1 + 2.3).

The generalized mass balance expression to calculate the oxygen requirement for $NBOD_L$ removal and an explanation of important terms are given below:

$$\begin{matrix}\text{Oxygen}\\ \text{requirement}\\ \text{for } NBOD_L\\ \text{removal}\\ (\text{mg } O_2/\text{L})\end{matrix} = \frac{4.57}{(\text{g } BOD_L/\text{g N})} \times \left\{[\text{TKN}] \text{ in influent, mg/L} - \begin{matrix}[NH_4^+\text{-N} + \text{soluble Org.-N}]\\ \text{in effluent, mg/L}\end{matrix} - \begin{matrix}[\text{Org.-N fixed}]\\ \text{in cell mass, mg/L}\end{matrix}\right\}$$

In the above expression, the factor of 4.57 is used to convert grams of nitrogen to grams of oxygen consumed (Sec. 13-7-1). TKN in the influent is 35 mg TKN/L. The allowable total NH_4^+-N and soluble fraction of organic nitrogen in the effluent is 1 mg (NH_4^+-N and soluble Org.-N)/L. The organic nitrogen in the cell mass is calculated from the total increase in cell mass. The organic nitrogen content of cellular mass is 12.2 percent of VSS.

$$\begin{aligned}\begin{matrix}\text{Oxygen}\\ \text{requirement}\\ \text{for } NBOD_L\\ \text{removal}\end{matrix} &= (4.57 \text{ g } BOD_L/\text{g N}) \times [35 \text{ mg TKN/L} \\ &\quad - 1 \text{ mg } (NH_4^+\text{-N and soluble Org.-N})/\text{L} \\ &\quad - (0.122 \text{ g Org.-N/g VSS}) \times (61.6 \text{ mg VSS/L})] \\ &= (160 - 4.6 - 34.3) \text{ mg } O_2/\text{L} \\ &= 121.1 \text{ mg } O_2/\text{L}\end{aligned}$$

3. Calculate total theoretical oxygen requirement (ThOR) in the BNR system.

$$\begin{aligned}\begin{matrix}\text{Total theoretical}\\ \text{oxygen requirement}\\ \text{in BNR system}\\ (\text{mg } O_2/\text{L})\end{matrix} &= \begin{matrix}\text{Oxygen}\\ \text{requirement}\\ \text{for } CBOD_L\\ \text{removal}\\ (\text{mg } O_2/\text{L})\end{matrix} + \begin{matrix}\text{Oxygen}\\ \text{requirement}\\ \text{for } NBOD_L\\ \text{removal}\\ (\text{mg } O_2/\text{L})\end{matrix} \\ &= 157.9 \text{ mg } BOD_L/\text{L} + 121.1 \text{ mg } BOD_L/\text{L} \\ &= 279 \text{ mg } O_2/\text{L}\end{aligned}$$

4. Calculate the total quantity of the theoretical oxygen requirement (ThOR).

$$\begin{aligned}\begin{matrix}\text{Total quantity}\\ \text{of theoretical}\\ \text{oxygen}\\ \text{requirement}\end{matrix} &= (279 \text{ g } O_2/\text{m}^3) \times (42{,}000 \text{ m}^3/\text{d}) \times (10^{-3} \text{ kg/g}) \\ &= 11718 \text{ kg } O_2/\text{d}\end{aligned}$$

Step I: Standard Oxygen Requirement (SOR)

1. Apply the field correction factors to ThOR.

The SOR under field conditions is calculated from Eq. (13-63):

$$\text{SOR, kg } O_2/\text{d} = \frac{\text{ThOR}}{[(C'_{SW}\,\beta\,F_s - C)/C_{SW}]\,\alpha\,\theta^{(T\text{-}20)}} \tag{13-63}$$

where

ThOR = theoretical oxygen requirement, kg/d
C_{SW} = solubility of oxygen in tap water at standard 20°C, with a value of 9.15 mg/L (Appendix A)
C'_{SW} = solubility of oxygen in tap water at field temperature, mg/L
C = design DO concentration maintained in the aeration basin, mg/L
α = oxygen transfer correction factor (a ratio of oxygen transfer in wastewater to that in tap water), usually 0.4–0.8 for diffused aeration and 0.6–1.2 for mechanical aeration
β = salinity surface tension factor (a ratio of oxygen saturation concentration in wastewater to that in tap water), usually 0.7–0.98 and typically 0.90 for wastewater
θ = temperature correction coefficient, usually 1.015–1.040 and typically 1.024 for both diffused and mechanical aeration applications
T = average temperature of mixed liquor in the basin under field conditions, °C
F_s = comprehensive oxygen solubility correction factor for elevation, submerged depth of diffuser, and oxygen content in air bubble.

The SOR under field conditions therefore is obtained by applying several correction factors in Eq. (13-63).

2. Calculate the comprehensive oxygen solubility correction factor.

The value of F_s may be calculated from Eq. (13-64):

$$F_s = \frac{1}{2}\left[\frac{(H_{\text{abso}} + H_{\text{basin}}}{10.3} + (1 - E_{O_2})\right] \tag{13-64}$$

where

H_{abso} = absolute atmospheric pressure at elevation above sea level, measured in head of water, m
H_{basin} = static pressure caused by water depth in the aerobic zone, measured in head of water, m
E_{O_2} = oxygen transfer efficiency of air diffusers, usually 0.06–0.12 (mass of oxygen transferred/mass of oxygen supplied)
10.3 = absolute atmospheric pressure at sea level (one atm), measured in head of water, m

The average temperature of mixed liquor in the aeration basin is dependent upon the ambient average air temperature and the influent temperature. Assume that the elevation is 500 m above sea level[xx] and average operating liquid temperature in the aeration basin is 24°C.

Using Eq. (13-64), F_s is calculated from the following data: $H_{abso} = 9.7$ m for an elevation of 500 m above sea level (see Appendix A), $H_{basin} = 5$ m (average water depth of aerobic zone), and the efficiency of air diffusers is 8 percent ($E_{O_2} = 0.08$).

$$F_s = \frac{1}{2}\left[\frac{(9.7\text{ m} + 5\text{ m})}{10.3\text{ m}} + (1 - 0.08)\right]$$
$$= 1.17$$

3. Calculate the SOR.
The value of SOR is calculated from Eq. (13-63). The following typical values are used: $C = 2.0$ mg/L, $\alpha = 0.75$, $\beta = 0.90$, $\theta = 1.024$, C'_{SW} at 24°C = 8.5 mg/L (Appendix A), and $C_{SW} = 9.15$ mg/L.

$$\text{SOR} = \frac{11{,}718\text{ kg O}_2\text{/d}}{\dfrac{(8.5\text{ mg/L} \times 0.90 \times 1.17 - 2.0\text{ mg/L})}{9.15\text{ mg/L}} \times 0.75 \times (1.024)^{(24-20)}}$$
$$= 18{,}707\text{ kg O}_2\text{/d}$$

Step J: Air Supply Requirement

1. Calculate the volume of air required.
The SOR in the BNR system is 18,707 kg O_2/d. Assume that air weighs 1.201 kg/m^3 and contains 23.2 percent oxygen by weight.

$$\begin{array}{c}\text{Theoretical air supply}\\ \text{required under}\\ \text{field condition}\end{array} = \frac{18{,}707\text{ kg O}_2\text{/d}}{1.201\text{ kg/m}^3 \times 0.232\text{ g O}_2\text{/g air} \times 0.08}$$
$$= 839{,}234\text{ m}^3\text{ air/d}$$

13-11-6 Design Calculations for Reactor Sizing and Configuration

Step A: Dimensions of the BNR Reactors. A modular design of the BNR system is selected. It consists of four process trains for operation in parallel. These four trains are arranged in two identical modules. Each module consists of two trains with a common wall. Within each train, there are anaerobic, anoxic, and aerobic zones in series. Sufficient cross-connections have been provided to isolate individual units under emergency situations. The dimensions of each zone are calculated below:

1. Select the size and configuration of reactors for the anaerobic zone.

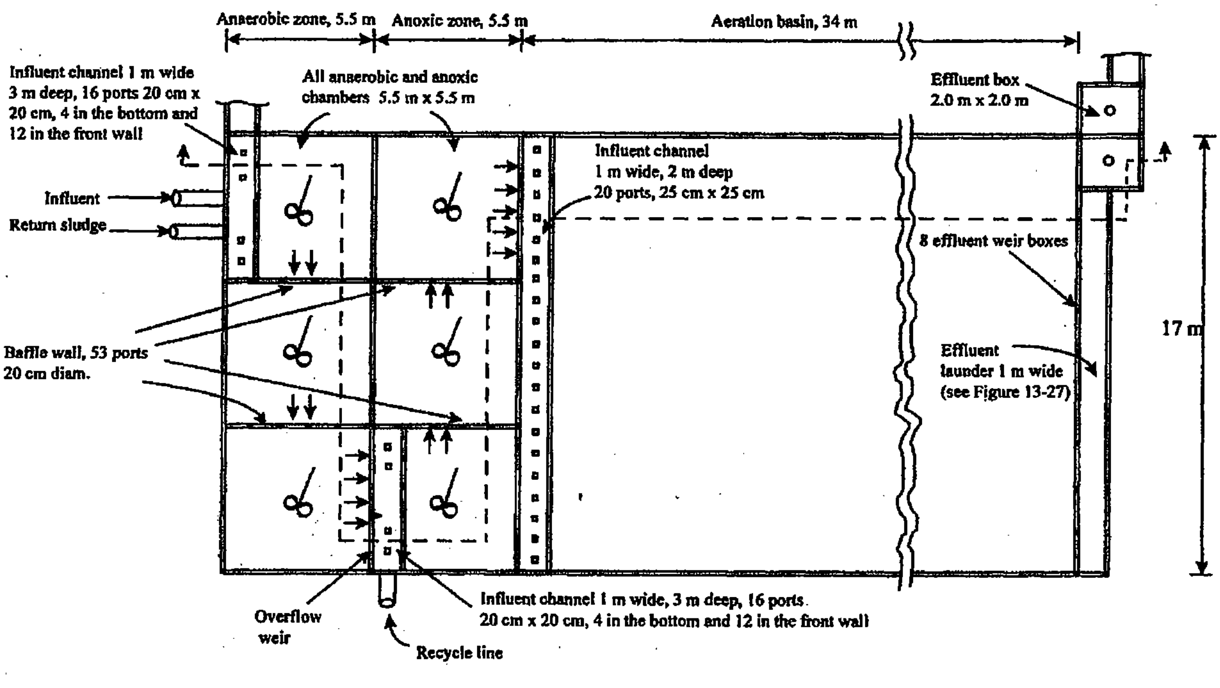
[xx]The drop in barometric pressure is approximately 0.01 atm per 1000 m of altitude above mean sea level.

Each process train will have three square reactors arranged in series. There are a total of twelve anaerobic reactors in the BNR system. Calculations for the dimensions of each anaerobic reactor are presented below:

Total volume of anaerobic zone	$= 2625\ m^3$ (Sec. 13-11-5, Step A, 3)
Volume of each reactor	$= \dfrac{2625\ m^3}{12} = 218.75\ m^3$
Provide a water depth	$= 7.25\ m$
Square area	$= \dfrac{218.75\ m^3}{7.25\ m} = 30.17\ m^2$
Length	$= \text{width} = \sqrt{30.17}\ m^2 = 5.5\ m$
The dimension of each reactor	$= 5.5\ m\ (18.0\ ft) \times 5.5\ m\ (18.0\ ft) \times 7.25\ m\ (23.8\ ft)$
Provide a freeboard	$= 0.8\ m\ (2.6\ ft)$

The total volume of twelve anaerobic reactors $= 2631\ m^3$. This is slightly larger than the required volume of $2625\ m^3$. These dimensions and other design details are shown in Figure 13-26.

2. Select the size and configuration of reactors for the anoxic zone.

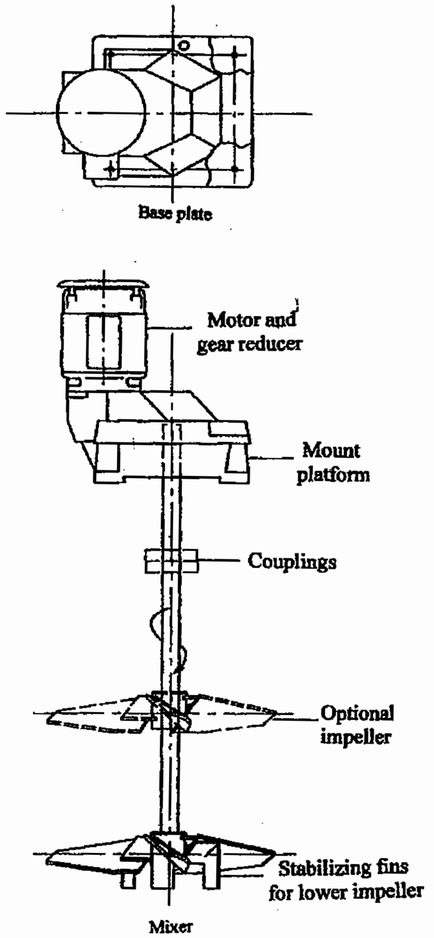


(a)

Figure 13-26 Design Details of BNR Facility: (a) plan.

Continued

(b)



(c)

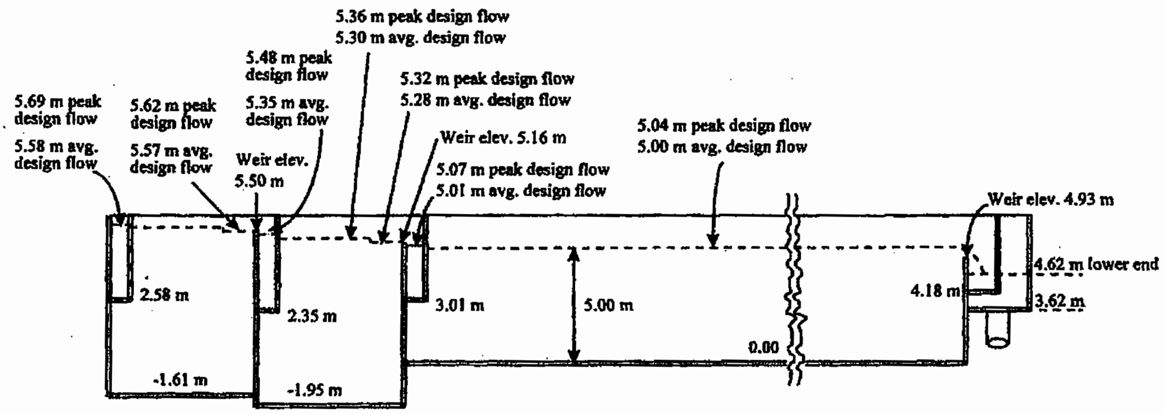


Figure 13-26—cont'd (b) mixer details (courtesy Lightnin Mixers), and (c) section and hydraulic profile.

The HRT and total volume for both anaerobic and anoxic zones are equal (Sec. 13-11-5, Step B, 4). Therefore, provide identical number, configuration, and reactor dimensions for both zones. The design details are shown in Figure 13-26.

3. Select the size and configuration of reactors for the aerobic zone.
   Provide a rectangular aeration basin in each process train. There are four aeration basins in the BNR system. Select the length-to-width ratio of 2:1 for each aeration basin. The dimensions of each aerobic reactor are calculated below:

Total volume of aerobic zone	$= 10{,}500\ \text{m}^3$ (Sec. 13-11-5, Step C, 6, c)
Volume of each reactor	$= \dfrac{10{,}500\ \text{m}^3}{4} = 2625\ \text{m}^3$
Provide a water depth	$= 5$ m
Surface area	$= \dfrac{2625\ \text{m}^3}{5\ \text{m}} = 525\ \text{m}^2$
Length (L)	$= 2$ width (W)
$2W \times W$	$= 525\ \text{m}^2$
W	$= 16.2$ m

Provide width 17.0 m-wide aeration basin:

Length	$= 2 \times 17.0\ \text{m} = 34.0\ \text{m}$
The dimension of each aeration basin	$= 17.0$ m (55.8 ft) $\times$ 34.0 m (111.5 ft) $\times$ 5 m (16.4 ft)
Volume of each aeration basin	$= 2890\ \text{m}^3$
Total volume of four aeration basins	$= 11{,}560\ \text{m}^3$
Provide a freeboard	$= 0.8$ m (2.6 ft)
Actual aeration period based on Q	$= \dfrac{2890\ \text{m}^3 \times 4}{42{,}000\ \text{m}^3/\text{d}} \times \dfrac{24\ \text{h}}{\text{d}} = 6.6\ \text{h}$

The design details of the aeration basins are shown in Figure 13-26.

Step B: Arrangement of Reactors and Flow Regime. Three square anaerobic chambers are provided in each module. The influent and return flows enter an influent channel in the first chamber that has a vertical turbine mixer. The flow passes through baffle walls that have several ports and enters into the second and third basins. Separate vertical mixers provide the needed mixing of the reactor contents. The effluent from the third basin goes over a rectangular weir and drops into the influent channel of the first anoxic chamber. In this chamber, the recycle flow from the aeration basin also enters. The influent channel has many ports through which the mixture exits and is mixed with the vertical turbine mixers that are identical to those in the aeration basin. The flow passes through the baffle walls into the second and third basins. The denitrified flow passes over a rectangular weir and drops into an influent channel to the aerobic zone that extends over the entire basin width. The flow from the influent channel discharges through the submerged ports into the aeration basin. The diffused aeration system provides mixing and oxygen requirements. On the far end of the aerobic basin is the effluent structure that consists of numerous effluent discharge ports, effluent launder, and the effluent box. The design details of each reactor are provided in Figure 13-26(a).

Step C: Influent Structure, Baffle Walls, and Effluent Weir of the Anaerobic Basin. The influent structure consists of a rectangular channel 1 m wide and 3 m deep constructed along the side of the first chamber. The influent and return sludge enter the channel. The channel has 16 discharge ports, 4 on the bottom and 12 on the front wall. The average water depth in all chambers is set at an average design condition (average design flow + return sludge). The hydraulic of the system is established at a peak design flow condition (peak design flow + return sludge). The dimensions and design details are given in Figure 13-26(a).

1. Calculate the design flows to each process train.

Average design flow to BNR $= 42{,}000\ m^3/d = 0.486\ m^3/s$

Average design flow to each process train $= \dfrac{0.486\ m^3/s}{4} = 0.122\ m^3/s$

Average return flow $= 0.486\ m^3/s \times 0.6 = 0.292^{yy}\ m^3/s$

Average return flow to each process train $= \dfrac{0.292 m^3/s}{4} = 0.073\ m^3/s$

Peak design flow to each process train $= \dfrac{1.321 m^3/s}{4} = 0.330\ m^3/s$

The flow under average design flow condition $= 0.122\ m^3/s + 0.073\ m^3/s$

$= 0.195\ m^3/s$

The flow under peak design flow condition $= 0.330\ m^3/s + 0.073\ m^3/s$

$= 0.403\ m^3/s$

2. Calculate the head loss through the influent structure.
The influent channel is 1.0 m wide, and a depth of 3 m is maintained at average design flow. The head loss in the influent channel is small. Major head loss occurs at the discharge ports.

Discharge through each port at average design flow condition $= \dfrac{0.195\ m^3/s}{16} = 0.012\ m^3/s$

Discharge through each port at peak design flow $= \dfrac{0.403\ m^3/s}{16} = 0.025\ m^3/s$

The head loss across the discharge port is calculated from Eqs. (11-9) and (11-10):

$^{yy}Q_r/Q = 0.6$ (Sec. 13-11-5, Step F, 1).

$$\Delta Z = \left\{ \frac{Q}{C_d \times A \times \sqrt{2g}} \right\}^2$$

If the discharge ports are 20 cm × 20 cm each and $C_d = 0.61$, the head loss at average and peak design flow conditions are

$$\Delta z = \left\{ \frac{0.012 \text{ m}^3/\text{s}}{0.61 \times 0.20 \text{ m} \times 0.20 \text{ m} \times \sqrt{2 \times 9.81 \text{ m/s}^2}} \right\}^2 = 0.01 \text{ m}$$

$$\Delta z_{\text{peak}} = \left\{ \frac{0.025 \text{ m}^3/\text{s}}{0.61 \times 0.20 \text{ m} \times 0.20 \text{ m} \times \sqrt{2 \times 9.81 \text{ m/s}^2}} \right\}^2 = 0.05 \text{ m}$$

3. Determine the number of ports and head loss through each baffle wall (total two baffle walls).
   Provide 53 orifices in each baffle wall (Figure 13-26). The diameter of each orifice is 20 cm.

   Head loss through each baffle wall at average and peak design flow conditions are obtained from Eqs. (11-9) and (11-10):

$$\Delta z_{\text{avg}} = \left\{ \frac{0.195 \text{ m}^3/\text{s}}{0.61 \times \pi/4 \, (0.20 \text{ m})^2 \times 53 \sqrt{2 \times 9.81 \text{ m/s}^2}} \right\}^2 = 0.002 \text{ m} \approx 0.0 \text{ m}$$

   This is small and can be ignored.

$$\Delta z_{\text{avg}} = \left\{ \frac{0.403 \text{ m}^3/\text{s}}{0.61 \times \pi/4 \times (0.20 \text{ m})^2 \times 53 \sqrt{2 \times 9.81 \text{ m/s}^2}} \right\}^2 = 0.008 \text{ m} \approx 0.01 \text{ m}$$

4. Calculate the head over the effluent weir at the third chamber of the anaerobic basin.
   Head over the weir at average and peak design flow conditions is calculated from Eq. (11-11):

$$Q = 2/3 \; C_d L' \sqrt{2g} \, H^{3/2}$$

   Since there are no end contractions, $L' = L$ (length of weir).

$$0.195 \text{ m}^3/\text{s} = 2/3 \times 0.6 \times 5.5 \text{ m} \sqrt{2 \times 9.81 \text{ m/s}^2} \, (H_{\text{avg}})^{3/2}$$

$$H_{\text{avg}} = 0.07 \text{ m}$$

$$0.403 \text{ m}^3/\text{s} = 2/3 \times 0.6 \times 5.5 \text{ m} \sqrt{2 \times 9.81 \text{ m/s}^2} \, (H_{\text{peak}})^{3/2}$$

$$H_{\text{peak}} = 0.12 \text{ m}$$

   Provide a free fall of 0.15 m at average flow.

Step D: Influent Structure, Baffle Walls, and Effluent Weir in Anoxic Chambers. The influent structure of the anoxic chamber consists of an influent channel along the side of anaerobic chamber. It receives weir overflow from the anaerobic chamber and recycle flow. The intermediate walls and effluent structure are similar to that of anaerobic chambers.

1. Calculate the design flows into anoxic chambers.
   The water depth in all chambers is established at average design flow conditions (average design flow + return sludge + recycle flow). The hydraulics of the system is established at peak design flow. The dimensions and design details are given in Figure 13-26.

The flow at average design condition $= 0.122\ \text{m}^3/\text{s} + 0.073\ \text{m}^3/\text{s} + 1.7 \times 0.122^{zz}\ \text{m}^3/\text{s} = 0.402\ \text{m}^3/\text{s}$

The flow at peak design condition $= 0.330\ \text{m}^3/\text{s} + 0.073\ \text{m}^3/\text{s} + 1.7 \times 0.122\ \text{m}^3/\text{s} = 0.610\ \text{m}^3/\text{s}$

2. Calculate head loss through the influent structure.
   The influent channel has 4 ports in the bottom and 12 ports in the front wall similar to the influent structure of the anaerobic basin. The ports are 20 cm × 20 cm square openings.

$$\Delta z_{avg} = \left\{ \frac{(0.402\ \text{m}^3/\text{s})/16}{0.61 \times 0.20\ \text{m} \times 0.2\ \text{m} \times \sqrt{2 \times 9.81\ \text{m/s}^2}} \right\}^2 = 0.05\ \text{m}$$

$$\Delta z_{peak} \left\{ \frac{(0.610\ \text{m}^3/\text{s})/16}{0.61 \times 0.20\ \text{m} \times 0.2\ \text{m} \times \sqrt{2 \times 9.81\ \text{m/s}^2}} \right\}^2 = 0.12\ \text{m}$$

3. Calculate head loss through each baffle wall (total two baffle walls).
   Provide baffle walls identical to those in anaerobic chambers. Head loss through each baffle wall at average and peak design flow conditions is calculated form Eqs. (11-9) and (11-10):

$$\Delta z_{avg} = \left\{ \frac{0.402\ \text{m}^3/\text{s}}{0.61 \times \pi/4 \times (0.20\ \text{m})^2 \times 53 \sqrt{2 \times 9.81\ \text{m/s}^2}} \right\}^2 = 0.01\ \text{m}$$

$$\Delta z_{peak} = \left\{ \frac{0.610\ \text{m}^3/\text{s}}{0.61 \times \pi/4 \times (0.20\ \text{m})^2 \times 53 \sqrt{2 \times 9.81\ \text{m/s}^2}} \right\}^2 = 0.02\ \text{m}$$

4. Calculate the head over the effluent weir at the third chamber of the anoxic basin.
   Head over the effluent weir at average and peak design flows are calculated from Eq. (11-11):

$$0.402\ \text{m}^3/\text{s} = 2/3 \times 0.6 \times 5.5\ \text{m} \sqrt{2 \times 9.81\ \text{m/s}^2}\, H_{avg}^{3/2}$$

$$H_{avg} = 0.12\ \text{m}$$

$$0.610\ \text{m}^3/\text{s} = 2/3 \times 0.6 \times 5.5\ \text{m} \sqrt{2 \times 9.81\ \text{m/s}^2}\, H_{peak}^{3/2}$$

$$H_{peak} = 0.16\ \text{m}$$

Provide a free fall of 0.15 m at average flow.

$^{zz}Q_{recycle}/Q = 1.7$ (Sec. 13-11-5, Step F, 3).

The head loss at average and peak design flow conditions is calculated for Eqs. (11-9) and (11-10).

$$\Delta z_{avg} = \left\{\frac{0.402 \text{m}^3/\text{s}}{0.61 \times 0.25 \text{ m} \times 0.25 \text{ m} \times 20 \sqrt{2 \times 9.81 \text{ m/s}^2}}\right\}^2 = 0.01 \text{ m}$$

$$\Delta z_{peak} = \left\{\frac{0.610 \text{ m}^3/\text{s}}{0.61 \times 0.25 \text{ m} \times 0.25 \text{ m} \times 20 \sqrt{2 \times 9.81 \text{ m/s}^2}}\right\}^2 = 0.03 \text{ m}$$

Step F: Effluent Structure of Aeration Basin

1. Select the arrangement of the effluent structure.
   The effluent structure consists of effluent weir boxes, a 1-m-wide effluent trough (launder), a 2 m × 2 m effluent box, and a 1.0-m-diameter outlet pressure pipe. Provide eight effluent weir boxes, each box having an adjustable rectangular weir 0.75 m in length. Provide stop-gates at each weir box for the flexibility to close some openings if a minimum head over the weirs is desirable under initial flow conditions. The details of effluent structure are shown in Figures 13-26 and 13-27.
2. Compute discharge over effluent structure.
   The recycle flow is withdrawn from the aeration basin. Therefore, the discharge over the effluent weir does not include recycle flow.

Discharge over weir under average design flow condition $= 0.195 \text{ m}^3/\text{s}$

Discharge per weir box under average design flow condition $= \dfrac{0.195 \text{ m}^3/\text{s}}{8} = 0.024 \text{ m}^3/\text{s}$

Discharge over weir under peak design flow condition $= 0.403 \text{ m}^3/\text{s}$

Discharge per weir box under peak design flow condition $= \dfrac{0.403 \text{ m}^3/\text{s}}{8} = 0.05 \text{ m}^3/\text{s}$

Calculate head over weir under average and peak design flow conditions. Use Eq. (11-11) and trial and error solution.

(Assume $L' = 0.74$ m)

$$\Delta z_{avg} = \left\{\frac{3}{2} \times \frac{0.024 \text{m}^3/\text{s}}{0.6 \times 0.74 \text{ m} \times \sqrt{2 \times 9.81 \text{ m/s}^2}}\right]^{2/3} = 0.07 \text{ m}$$

$$L' = 0.75 \text{ m} - 0.2 \times 0.07 \text{ m} = 0.74 \text{ m}$$

(Assume $L' = 0.73$ m)

$$\Delta z_{peak} = \left\{\frac{3}{2} \frac{0.050 \text{ m}^3/\text{s}}{0.6 \times 0.73 \text{ m} \times \sqrt{2 \times 9.81 \text{ m/s}^2}}\right\}^{2/3} = 0.11 \text{ m}$$

$$L' = 0.75 \text{ m} - 0.2 \times 0.11 \text{ m} = 0.73 \text{ m}$$

3. Design the effluent trough (launder).
   The design procedure for the effluent trough (launder) is given in Chapters 11 and 12. Eq. (11-17) is used to obtain an approximate solution, while Eqs. (11-12)–(11-16) provide a more exact solution. The computational procedure based on Eqs. (11-12)–(11-16) is tedious and seldom used by the designers. The design procedure with an approximate solution is given in this example. Using the notations defined in Eq. (11-17):

$$L = 17.0\text{ m} - 2.0\text{ m} = 15.0\text{ m (Figure 13-26)}$$

$$y_2 = 0.44^{aaa}\text{ m (assume)}$$

$$Q = \text{max. flow from eight weir boxes} = 0.403\text{ m}^3/\text{s}$$

$$b = 1.0\text{ m}$$

$$y_1 = \sqrt{(0.44\text{ m})^2 + \frac{2 \times (0.403\text{ m}^3/\text{s})^2}{9.81\text{ m/s}^2 \times (1\text{ m})^2 \times 0.44\text{ m}}} = 0.52\text{ m}$$

   Allow 16 percent for friction losses, turbulence, and bend, and add 0.15 m drop to ensure free fall.

$$\text{Total depth of the effluent trough} = 0.52\text{ m} \times 1.16 + 0.15\text{ m} = 0.75\text{ m}$$

4. Establish the hydraulics in the effluent box and exit pipe.
   The effluent box is 2 m × 2 m, and the exit pipe is 1.0 m in diameter. The liquid level in the effluent box will depend upon the total head loss in the exit pipe and the free liquid surface in the final clarifier. Select conditions such that a 1.0-m liquid depth is maintained in the effluent box at peak design flow conditions.

$$\text{Velocity in the exit pipe} = \frac{0.403\text{ m}^3/\text{s}}{\pi/4 \times (1\text{ m})^2} = 0.51\text{ m/s}$$

Step G: Hydraulic Profile. The hydraulic profile through the BNR at average and peak design flow conditions is given in Figure 13-26(c). The hydraulic profile is prepared from the head loss calculations obtained at average and peak design flow conditions in Sec. 13-11-6, Steps B–E.

13-11-7 Design Calculations for Equipment Selection

The design calculations for equipment selection for the BNR system include (1) mixers in anaerobic and aerobic zones, (2) aeration piping and diffusers, (3) blowers, (4) internal recirculation system, and (5) waste sludge system.

[aaa]Critical depth = 0.26 m (submerged outfall).

Step A: Mixer Power Requirement for Anaerobic and Anoxic Zones

1. Calculate the volume of each chamber of the anaerobic zone.
   The anaerobic zone has three chambers, and the dimensions of each chamber are 5.5 m × 5.5 m × 7.25 m (Sec. 13-11-6, Step A, 1).
2. Calculate the power requirement for anaerobic zone, $P_{\text{anaerobic}}$.
   The power requirement for each chamber is calculated from Eq. (13-41):

$$\left(\frac{P_{\text{anaerobic}}}{V_{\text{anaerobic}}}\right) = 0.00094\,(\mu)^{0.3}\,(\text{MLSS})^{0.298}$$
$$= 0.00094\,(1.0087)^{0.3} \times (3750)^{0.298} = 0.011\ \text{kW/m}^3$$

$$P_{\text{anaerobic}} = 0.011\ \text{kW/m}^3 \times 219.3\ \text{m}^3 = 2.5\ \text{kW}$$

   Provide each mixer with a variable-speed regulator and a 2.0-kW motor. The design details of the mixer selected are shown in Figure 13-27.
3. Calculate the power requirement for the anoxic zone, P_{anoxic}.
   Both anaerobic and anoxic reactors are identical in dimensions (Sec. 13-11-5, Step A, 2). Provide identical mixers and motors as those for the anaerobic zone. Mixer details are shown in Figure 13-2(b).

Step B: Design Capacity of Air Supply System. The theoretical air supply required under field condition = 839,234 m^3 air/d (Sec. 13-11-5, Step J).

1. Calculate the design capacity of the air supply system.
   Provide design air at 150 percent of theoretical air.

$$\begin{aligned}\text{Total design air} &= 839234\ \text{m}^3\ \text{air/d} \times 1.5\\ &= 1258851\ \text{m}^3/\text{d}\\ &= 874\ \text{m}^3/\text{min for four basins}\\ &= 219\ \text{m}^3/\text{min per basin}\end{aligned}$$

2. Check the volume of air per kg BOD$_5$ removal, per m^3 of wastewater treated, and per m^3 of aeration tank volume.
   a. Volume of air supplied per kg of BOD$_5$ removed

$$= \frac{1{,}258{,}851\ \text{m}^3\ \text{air/d}}{[(200 - 44.3^{\text{bbb}} - 3.7^{\text{ccc}})\text{g/m}^3] \times (42{,}000\ \text{m}^3/\text{d}) \times (10^{-3}\ \text{kg/g})}$$

$$= 198\ \text{m}^3\ \text{air/kg BOD}_5\ \text{removed}\ (3171\ \text{ft}^3/\text{lb})$$

   b. Volume of air supplied per m^3 of wastewater treated

$$= \frac{1{,}258{,}851\ \text{m}^3\ \text{air/d}}{42{,}000\ \text{m}^3/\text{d}}$$

$$= 30\ \text{m}^3\ \text{air/m}^3\ \text{of wastewater treated}\ (4.1\ \text{ft}^3/\text{gal})$$

[bbb]The amount of BOD$_5$ consumed because of denitrification is 44.3 mg BOD$_5$/L (Sec. 13-11-5, Step C, 2, b).
[ccc]The concentration of soluble BOD$_5$ in the effluent is 3.7 mg BOD$_5$/L (Sec. 13-11-5, Step C, 3, b).

c. Total volume of aerobic zone = 10,500 m^3 (Sec. 13-11-5, Step C, 6, c)

$$\text{Volume of air supplied per m}^3 \text{ of aerobic zone volume} = \frac{1{,}258{,}851 \text{ m}^3 \text{ air/d}}{10{,}500 \text{ m}^3}$$

$$= 120 \text{ m}^3 \text{ air/d per m}^3 \text{ aerobic zone } (120 \text{ ft}^3/\text{ft}^3{\cdot}\text{d})$$

Step C: Design Capacity of Diffused Aeration System

1. Select diffuser tube.
   Provide Dacron sock diffusers, with a standard tube dimension of 61 cm × 7.5 cm (ID), discharging 0.21 m^3 standard air per minute per tube (7.4 cfm).
2. Calculate the number of diffuser tubes and arrangement.

$$\text{Total number of diffuser tubes} = \frac{874 \text{ m}^3/\text{min}}{0.21 \text{ m}^3/\text{min/tube}}$$

$$= 4162 \text{ tubes}$$

   Provide 4320 diffusers:[ddd]

$$\text{Number of diffuser tubes per basin} = 4320/4 = 1080$$

   Provide 15 rows of diffuser tubes along the width of the aeration basin:

$$\text{Number of diffuser tubes per row} = 1080/15 = 72$$

   Provide four knee and swing joint vertical hanger pipes per row:

$$\text{Number of diffuser tubes per hanger pipe} = 72/4 = 18$$

   The arrangement of the diffuser tubes and the piping layout is illustrated in Figure 13-27. An alternate arrangement would be to provide a grid system of headers.
3. Calculate the head losses in piping and diffusers.
   The power required to supply air to the diffusers depends on the supply pressure at the blowers. The supply pressure at the blowers depends on the head losses in pipings, valves, and fittings and the depth of submergence. The technique for determining the head losses through the air piping system is similar to that for head loss calculations for wastewater pumping equipment. Basic rules are summarized below.
   a. The normal range of air velocities used to design the pipings is given in Table 13-14.
   b. The total head loss in pipe headers is generally 5–20 cm of water (2–8 in.).

[ddd]Actual number of diffuser tubes provided is 4320. This number is based on the design symmetry. There are 15 rows of diffusers in each basin, and each row has four hanger pipes, with 18 diffuser tubes in each hanger pipe. Total number of diffuser tubes = 4 basins × 15 rows × 4 hanger pipes/row × 18 diffuser tubes/hanger pipe = 4320 diffuser tubes in four basins. If 17 diffuser tubes are provided in each hanger pipe, then the total number of diffusers will be 4 × 15 × 4 × 17 = 4080 (less than required). For details, study Figure 13-27 and Table 13-16.

Step E: Influent Structure of Aeration Basin

1. Select the arrangement of the influent structure.
   The influent structure consists of a rectangular channel constructed along the entire width of the aeration basin. The channel has a width of 1 m, and water depth at average flow condition is 2 m. The influent discharges over the effluent weir of the third anoxic chamber into the channel at one side of the basin and flows toward the other end. The channel has 20 submerged ports 25 cm × 25 cm each along the bottom of the channel to distribute the influent flow into the aeration basin. The design details are shown in Figures 13-26 and 13-27.

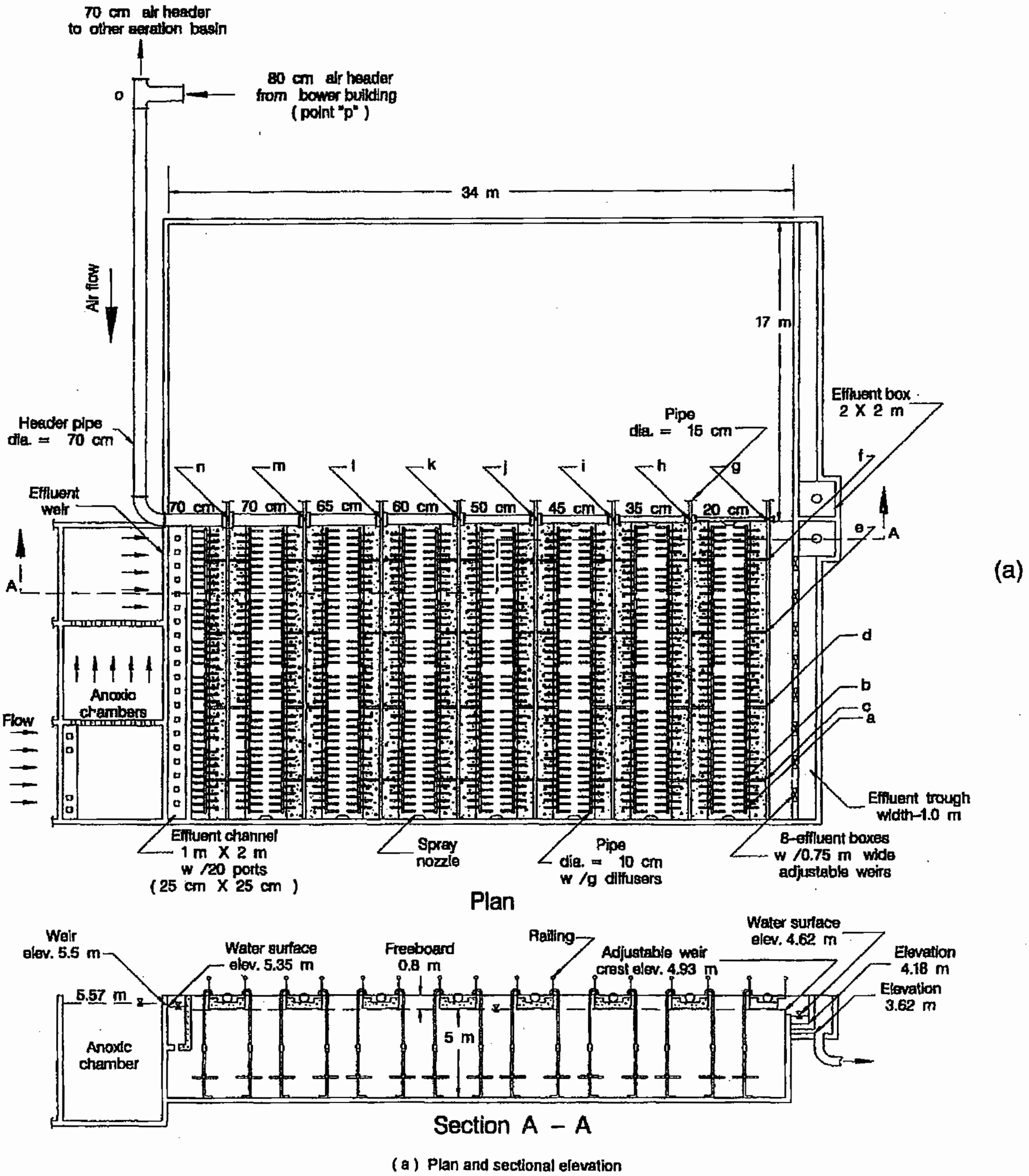


Figure 13-27 Design Details of Aeration Basin and Aeration System.

Continued

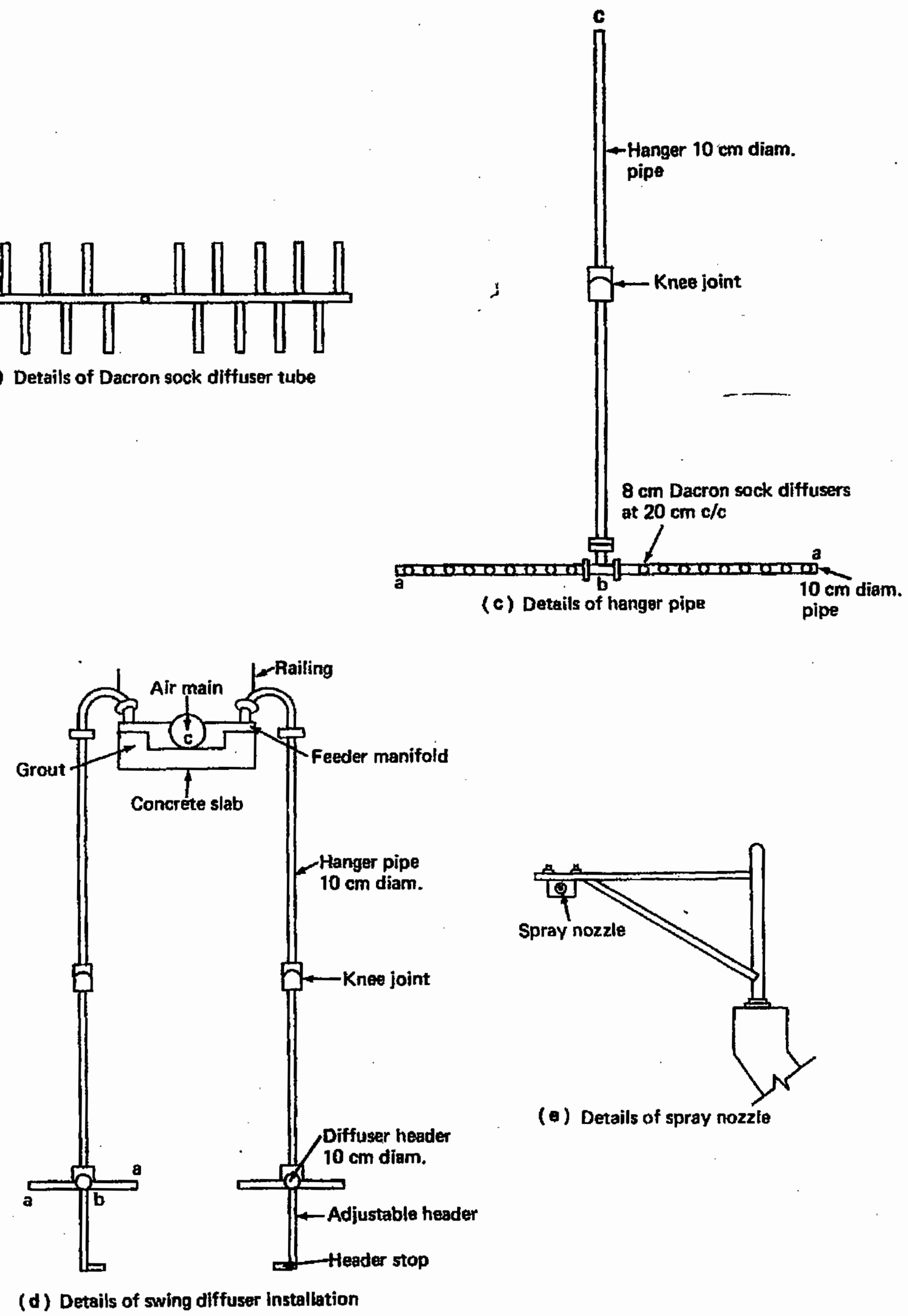


Figure 13-27—cont'd

2. Determine the influent flow into the aeration basin.
   The water depth in the aeration basin is established at average flow condition. The hydraulics is calculated at peak design flow condition. The influent to the aeration basin is the denitrified flow discharged from the anoxic basin

   The influent flow at average design condition = 0.402 m^3/s (Step D, 1)

   The influent flow at peak design condition = 0.610 m^3/s (Step D, 1)

3. Calculate the head loss across the submerged influent ports.

TABLE 13-14 Design Range of Air Velocities in Header Pipes

Pipe Diameter (cm)	Velocity, Standard Air (m/min)
2–8	360–500
8–25	500–900
25–40	900–1050
40–60	1050–2000
60–80	1200–1350
80–150	1350–1950

c. Head losses through diffusers generally range from 40–50 cm of water (16–20 in.). Most manufacturers provide rating curves for the diffusers. Allowance for clogging is also recommended by the manufacturers.

d. As an approximation, the friction factor for steel pipes carrying air may be obtained from Eq. (13-65):[89]

$$f = \frac{0.029\,(D \text{ in m})^{0.027}}{(Q \text{ in m}^3/\text{min})^{0.148}} \tag{13-65}$$

e. The head loss in the straight pipe may be calculated from Eqs. (13-66) or (13-67):[89]

$$h_L = f\,\frac{L}{D}\,h_v \tag{13-66}$$

$$h_L = 9.82 \times 10^{-8}\,\frac{fLTQ^2}{PD^5} \tag{13-67}$$

where

h_L = head loss, mm H_2O
L = equivalent length, m
D = pipe diameter, m
h_v = velocity head, mm H_2O
Q = air flow, m^3/min
P = air supply pressure, atmospheres
T = temperature in pipe, °K, obtained from Eq. (13-68)

$$T = T_o\,(P/P_o)^{0.283} \tag{13-68}$$

where

T_o = ambient air temperature, °K
P_o = ambient barometric pressure, atmospheres
P = air supply pressure, atmospheres

TABLE 13-15 Factors to Convert Fitting Losses into Equivalent Lengths of Pipe

Fittings	C Values for Equivalent Pipe Lengths
Gate valve	0.25
Long-radius ell or run of standard tee	0.33
Medium-radius ell or run of tee reduced in size 25 percent	0.42
Standard ell or run of tee reduced in size 50 percent	0.67
Angle valve	0.90
Tee through side outlet	1.33
Globe valve	2.00

Source: From Ref. 90. Used with permission of Water Environment Federation, and American Society of Civil Engineers.

f. The losses in pipe fittings (elbows, tees, valves, meters, etc.) may be computed using equivalent pipe length [Eq. (13-69)]:[11,89,90]

$$L = 55.4\, CD^{1.2} \tag{13-69}$$

where

L = equivalent length of pipe fittings in meters for pipe diameter of D
C = factor for equivalent length of pipe given in Table 13-15

Another method of calculating the pressure drop through valves and fittings is to apply the appropriate multiplier K to the velocity head, similar to those for water pipes. The values of K may be obtained in standard hydraulics textbooks.

g. The losses through air filters, blower, silencer, check valves, etc., should be obtained from the equipment manufacturers. The following values may be used as a guide:
   - air filter losses — 13–76 mm (0.5–3.0 in.)
   - silencer losses:
     centrifugal blower — 13–38 mm (0.5–1.5 in.)
     positive displacement blower — 152–216 mm (6.0–8.5 in.)
   - check valve losses — 20–203 mm (0.8–8.0 in.)

h. The overall pressure drop in pipings, diffusers, and other accessories is usually 20–40 percent of the submergence over the diffusers.

i. Air pipings with fine bubble diffusers must be of noncorroding and nonscaling material. Steel, galvanized, or plastic pipes are commonly used.

j. The head losses in the air pipings are calculated for the line that gives the maximum head loss. The sum of all the head losses plus the submergence of the diffusers gives the discharge pressure at the blower. Since the head losses in the pipings and the diffusers depend on the supply pressure and temperature of the air, an iterative procedure is required. The calculation steps for the final iteration are summarized in Table 13-16.[eee]

Step D: Blower Sizing. Blowers develop a pressure differential between the inlet and discharge points. They move air or gases under pressure. There are two types of blowers: centrifugal and rotary positive displacement. The centrifugal blowers are commonly used for pressures 50–70 kPa (7–10 psi) and air flows above 15 m^3/min (5000 cfm). These blowers have head capacity curves similar to low specific-speed centrifugal pumps (Chapter 9). The operating point is determined by the intersection of the head capacity curve and the system curve. The flow may be adjusted by throttling the inlet. Throttling the outlet of centrifugal blowers is not recommended because these machines will surge[fff] if throttled close to the shutoff head.

Rotary positive displacement blowers are used for smaller installations (less than 45 m^3/min). Select centrifugal blowers in this example. The blowers should not be throttled because damage to the machine may occur because of overheating. The high-pitched whine emitted by blowers could be very disagreeable, and silencers should be installed.

1. Calculate supply pressure at the blower.
   Supply pressure at the blower (in terms of head of water) is calculated as follows:

Total losses in pipings (Table 13-16)	= 214.35 mm
Loss in elbows, tees, valves, meters (assume 10 percent)	= 23.74 mm
Losses in air filter (manufacturer's data)	= 50.00 mm
Losses in silencers (centrifugal)	= 30.00 mm
Losses in compressor pipings and valvings used for parallel combinations (check valve, butterfly valves, relief valve, piping and connections, etc.)	= 210.00 mm
Submergence (water depth above the diffusers)	= 4500.00 mm
Diffuser losses, fine bubble diffuser tubes (manufacturers' data)	= 250.00 mm
Allowance for clogging of diffusers and miscellaneous head losses under emergency conditions	= <u>525.20</u>
Total	= 5803 mm
	= 5.80 m (19.05 ft)

   The absolute supply pressure = (5.80 m + 10.34 m)/10.34 m[ggg] = 1.56 atm
2. Compute the volume of air.
   The volume of air to be supplied by the blower is determined as follows:

Total standard air (150 percent of theoretical air) for two basins (Table 13-16)	= 453.60 m^3/min
Total air for four basins (Table 13-16)	= 907.20 m^3/min

[eee]An alternative arrangement would be to provide a pipeheader loop around the tank with feed crossheaders.

[fff]Surging is a phenomenon in which a blower starts to operate alternately at zero and full capacity. This follows with vibration and overheating.

[ggg]One std atmosphere = 10.34 m of water.

TABLE 13-16 *Head Loss Calculations in Air Pipings Designed for Aeration Basin*

Line[a]	Description	Diam. (cm)	Air Flow[b] (m^3/min)	Vel.[c] (m/min)	L[d] (m)	f[e]	h_L[f] (mm)
ab	Horizontal diffuser header contains nine diffuser tubes	10	0.21–1.89 Avg-1.05	134	3.0	0.027	0.020
bc	Hanger pipe contains horizontal diffuser header with 18 tubes	10	3.78	481	12.0	0.022	8.28
cd	Pipe header along the width of the basin supplying air to one hanger pipe	15	3.78	214	8.0	0.023	0.76
de	Pipe header along the width of the basin supplying air to two hanger pipes	15	7.56	428	8.0	0.020	2.64
ef	Pipe header along the width of the basin supplying air to three hanger pipes	15	11.34	642	8.0	0.019	5.65
fg	Pipe header along the width of the basin supplying air to four hanger pipes (one row of diffusers)	15	15.12	856	18.0	0.018	21.41
gh	Pipe header (along the length of the basin) supplying air to two rows of pipe headers (one row in each basin)	20	30.24	963	10.0	0.017	10.66
hi	Pipe header supplying air to 6 rows of pipe headers (3 rows in each basin)	35	90.72	943	10.0	0.015	5.16

ij	Pipe header supplying air to 10 rows of pipe headers (5 rows in each basin)	45	151.20	951	10.0	0.014	3.81
jk	Pipe header supplying air to 14 rows of pipe headers (7 rows in each basin)	50	211.68	1078	10.0	0.013	4.09
kl	Pipe header supplying air to 18 rows of pipe headers (9 rows in each basin)	60	272.16	963	10.0	0.013	2.72
lm	Pipe header supplying air 22 rows of pipe headers (11 rows in each basin)	65	332.64	1002	10.0	0.012	2.51
mn	Pipe header supplying air to 26 rows of pipe headers (13 rows in each basin)	70	393.12	1022	10.0	0.012	2.42
no	Pipe headers supplying air to 30 rows of pipe headers (15 rows in each basin)	70	453.60	1179	80.0	0.012	25.76
op	Air main supplying air from the compressor to the aeration basins	80	907.20	1805	179	0.011	118.28
	Total						214.35

[a]Air lines are marked in Figure 13-27.
[b]Air flow is based on standard oxygen requirement under field conditions.
[c]Velocity = air flow/area.
[d]Estimated equivalent length of pipe.
[e]Calculated from Eq. (13-65).
[f]Calculated from Eqs. (13-67) and (13-68) using $P = 156$ atm, $P_o = 0.95$ atm [for 500-m altitude (see Sec. 13-11-5, Step I, 2)], $T_o = 273 + 30°C$ (summer temp.) = 303°K.

3. Select number of blowers.

The number of centrifugal blowers chosen to deliver the required air volume should be an integer divisor of the total flow plus one extra machine for standby service.

Provide a total of five centrifugal blowers each of 230 m^3/min (8200 cfm) design capacity. These blowers shall be arranged in parallel. Each blower shall have a surge point of approximately 50 percent below the total design flow. The parallel arrangement shall provide the following operational flexibility:

- Four blowers will meet the 150 percent average air requirements (design period). The maximum air supply = 920 m^3/min.
- Three blowers will meet the average air requirements (design period)[hhh] = 690 m^3/min.
- Two blowers will meet the average air requirement (initial period)[iii] = 460 m^3/min.
- Five blowers will provide one unit for standby service.

All blowers shall be provided in a blower building as discussed in Chapter 20 and shown in Figure 13-11. Proper suction and discharge silencers and suitable foundations for compressors shall be provided.

4. Calculate power requirements.

Blower power requirements are estimated from air flow, discharge and inlet pressures, and air temperature by using Eq. (13-70).[89] This equation is based on an assumption of adiabatic conditions.

$$P_w = \frac{wRT_o}{8.41e}\left[\left(\frac{P}{P_o}\right)^{0.283} - 1\right] \tag{13-70}$$

where

P_w = the power requirement of each blower, kW
w = the air mass flow, kg/s
R = gas constant, 8.314, kJ/k mole °K
8.41 = constant for air, kg/k mole
T_o = inlet temperature, °K
P_o = absolute inlet pressure, atm
P = absolute outlet pressure, atm
e = efficiency of the machine (usually 70–80 percent)

At 75 percent efficiency

$$P_w = \frac{230\ \text{m}^3/\text{min} \times 1.201\ \text{kg/m}^3 \times 8.314\ \text{kJ/k mole °K} \times 303\ \text{°K}}{8.41\ \text{kg/mole} \times 0.75 \times 60\ \text{s/min}} \times \left[\left(\frac{1.56}{0.95}\right)^{0.283} - 1\right]$$

$$= 277\ \text{kW (365 hp)}$$

[hhh]Average air requirement $= \frac{907.20\ \text{m}^3/\text{min}}{1.5} = 605\ \text{m}^3/\text{min}$. Three blowers will provide 690 m^3/min.

[iii]Based on volume of wastewater treated, the average air requirement for the initial year $= \frac{0.304\ \text{m}^3/\text{s}}{0.440\ \text{m}^3/\text{s}} \times 605\ \text{m}^3/\text{min} = 418\ \text{m}^3/\text{min}$ (see Table 6-9 for flows). Two blowers will provide 460 m^3/min.

The procedure presented above permits tentative selection of diffused aeration equipment based on many manufacturers' claims. The design engineer must develop similar data for his design using the details supplied by the equipment manufacturers.

Step E: Waste Activated Sludge System

1. Select arrangement for waste sludge withdrawal.
   The excess solids are pumped from each aeration basin to a phosphorus stripper (P-stripper) tank. From the P-stripper tank the waste activated sludge is pumped into the sludge thickener. The total amount of WAS from four aeration basins is 2820 kg/d or 752 m^3/d (see Sec. 13-11-5, Steps D, 2 and D, 4) at a solids concentration of 3750 mg/L (g/m^3).
2. Select pumps, piping, and pumping cycle.
   Provide five identical constant-speed waste sludge pumps. One pump is dedicated to each aeration basin, while the fifth pump is common to all basins. The design pumping capacity of each pump is 1.5 m^3/min (400 gpm).[iii] The pumping duration and frequency of operation shall be controlled by the continuous solids monitoring system. In addition, the pumping operation shall also be controlled by an automatic time-controlled clock where both pumping duration and frequency of operation may be controlled in the event that the solids-monitoring system does not function. The pipings and arrangements of the waste sludge pumps are presented in Chapter 20 and are also shown in Figure 13-25. The procedure for pump selection and design of the pumping station is given in Chapter 9.

Step F: Spray Nozzles. The design of spray nozzles and their spacings are shown in Figure 13-27. Two nozzles shall be capable of producing a hard, flat spray about 10 L/min at 103 kN/m^2. The pump shall be capable of pumping the total flow of all nozzles at the required nozzle pressure.

13-11-8 Design Calculations of Solids Separation Facility

Step A: Surface Area of Secondary Clarifier

1. Establish design flow.

$$\begin{aligned}\text{Design flow to the secondary clarifier} &= \text{average design flow} + \text{return sludge flow} - \text{MLSS wasted}\\ &= 0.486\ m^3/s + 0.292\ m^3/s - 752^{kkk}\ m^3/d \times (86{,}400\ s/d)^{-1} = 0.769\ m^3/s\end{aligned}$$

$$\text{Design flow to each secondary clarifier} = 1/4 \times 0.769\ m^3/s = 0.192\ m^3/s$$

2. Prepare solids flux curves.
   From the MLSS settling curve given in Figure 13-23, prepare the solids flux curve.

[iii]1.5 m^3/min (400 gpm) pumping rate is large enough to select a nonclog pump for sludge wasting.

[kkk]Average design flow and return sludge values are calculated in Sec. 13-11-6, Step C, 1. WAS is calculated in Sec. 13-11-5, Step D, 2.

TABLE 13-17 Computation of Solids Flux Rate

Solids Concentration X (g/m^3)	1000	1500	2000	3000	4000	5000	6000	7000	8000	9000
Initial settling rate V_i, m/h	4.40	4.20	2.80	1.30	0.67	0.34	0.20	0.10	0.05	0.03
Solids flux rate, XV_i, $kg/m^2{\cdot}h$	4.4	6.3	5.6	3.9	2.7	1.7	1.2	0.7	0.4	0.3

The computation data is summarized in Table 13-17. The solids flux curve is shown in Figure 13-28.

3. Determine limiting solids loading rate.
   From Figure 13-28 determine the limiting solids flux (SF) for an underflow concentration of 10,000 mg/L. This is obtained by drawing a tangent to the solids flux curve from 10,000 mg/L solids concentration in return sludge. The solids flux value is $2.0\ kg/m^2{\cdot}h = 2.0 \times 24 = 48\ kg/m^2{\cdot}d$ ($9.81\ lb/ft^2{\cdot}d$).
4. Calculate the area and diameter of the secondary clarifier.
   The area of the clarifier is obtained from Eq. (13-71):

$$A = \frac{QX'}{SF} \tag{13-71}$$

where

A = area of the clarifier, m^2
Q = total flow to the clarifier, including recirculation, m^3/h
X' = MLSS, kg/m^3
SF = limiting solids flux obtained from Figure 13-28, $kg/m^2{\cdot}h$

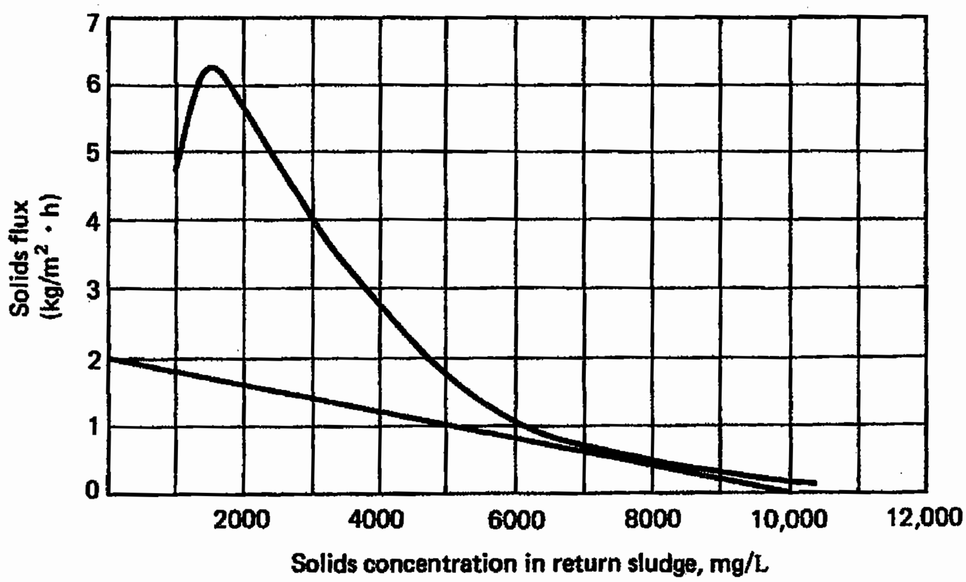


Figure 13-28 Solids Flux Curve Prepared from the Settling Data Given in Figure 13-10.

The flow to each clarifier, including return sludge flow, is $0.192\ m^3/s \times 3600\ s/h = 691\ m^3/h$.

$$A = \frac{691\ m^3/h \times 3.750\ kg/m^3}{2.0\ kg/m^2 \cdot h} = 1296\ m^2$$

$$\text{Diameter of the secondary clarifier} = \sqrt{\frac{1296\ m^2 \times 4}{\pi}}$$

$$= 40.6\ m\ (133\ ft)$$

Provide four clarifiers each of 40.7 m (134 ft) diameter:

$$\text{Actual area} = \frac{\pi}{4}(40.7\ m)^2 = 1301\ m^2$$

5. Check the overflow rate at average design flow plus recirculation.

$$\text{The overflow rate} = \frac{Q}{A} = \frac{0.192\ m^3/s \times 86{,}400\ s/d}{1301\ m^2}$$

$$= 12.8\ m^3/m^2 \cdot d\ (314\ gal/ft^2 \cdot d)$$

(satisfactory, less than $15\ m^3/m^2 \cdot d$, see Design Criteria)

6. Check the clarifier area for clarification requirement.

$$\text{Calculated overflow rate} = 12.8\ m^3/m^2 \cdot d = 0.533\ m/h$$

From Figure 13-23 the MLSS concentration corresponding to a 0.533 m/h settling rate is about 4400 mg/L. Since the design value of the MLSS concentration is 3750 mg/L, the area for clarification will be sufficient.

7. Check the overflow rate at peak design flow plus recirculation.

$$\text{At peak design flow plus recirculation the flow to each clarifier} = \frac{1.321\ m^3/s + 0.292\ m^3/s}{4}$$

$$= 0.403\ m^3/s$$

$$\text{Overflow rate} = \frac{0.403\ m^3/s \times 86{,}400\ s/d}{1301\ m^2}$$

$$= 26.8\ m^3/m^2 \cdot d$$

(satisfactory, less than $35\ m^3/m^2 \cdot d$, see Design Criteria)

8. Calculate the solids loadings.

$$\text{The limiting solids loading} = \frac{0.192\ m^3/s \times 3750\ g/m^3 \times 86{,}400\ s/d}{1000\ g/kg \times 1301 m^2}$$

$$= 47.8\ kg/m^2 \cdot d < 48\ kg/m^2 \cdot d$$

(satisfactory, see Sec. 13-11-8, Step A, 3)

$$\text{Solids loading at peak Design flow} = \frac{0.403\ \text{m}^3/\text{s} \times 3750\ \text{g/m}^3 \times 86{,}400\ \text{s/d}}{1000\ \text{g/kg} \times 1301\ \text{m}^2}$$

$$= 100.4\ \text{kg/m}^2\!\cdot\!\text{d}$$

(satisfactory, less than 150 kg/m^2·d, see Design Criteria)

Step B: Depth of Secondary Clarifier. The liquid depth of the secondary clarifier = depth of clear water zone + depth of thickening zone + depth of sludge storage zone.

1. Determine clearwater and settling zones.
   The clearwater and settling zones are generally 1.0–1.5 m and 1.5–2.0 m, respectively. Provide a total of 3.0 m of clearwater and settling zones.
2. Compute the depth of the thickening zone.
   The depth of thickening and sludge storage zones are calculated using the procedure and assumptions given in Ref. 4, 11, and 12.
   It is assumed that, under normal conditions, the mass of sludge retained in the clarifier is 30 percent of the mass of solids in the BNR reactor,[iii] and the average concentration of sludge in the thickening zone of the clarifier is 7000 mg/L.

$$\text{Total mass of solids in BNR reactor} = \frac{3750\ \text{g/m}^3 \times 16{,}822\ \text{m}^3}{1000\ \text{g/kg}}$$

$$= 63{,}083\ \text{kg}$$

$$\text{Total mass of solid in each reactor} = 63{,}083\ \text{kg}/4 = 15{,}771\ \text{kg}$$

$$\text{Total mass of solids in each clarifier} = 0.3 \times 15{,}771\ \text{kg} = 4731\ \text{kg}$$

$$\text{Depth of thickening zone} = \frac{\text{total solids in the clarifier}}{\text{concentration} \times \text{area}}$$

$$= \frac{4731\ \text{kg} \times 1000\ \text{g/kg}}{7000\ \text{g/m}^3 \times 1301\ \text{m}^2}$$

$$= 0.52\ \text{m} \approx 0.5\ \text{m}$$

3. Compute the depth of sludge storage zone.
   A long sludge storage zone to store the sludge in the clarifier is generally not provided with a BNR facility. Long storage may cause phosphorus release in the clarifier. Therefore, if the sludge-processing facilities are unable to handle the solids for such reasons as equipment breakdown or unusual sustained waste-loading conditions, a desirable control method is to increase the sludge return rate. This will increase the sludge storage in the BNR facility. At a high return sludge rate, the DO level in the sludge zone will remain favorable even if sludge storage is allowed in the clarifier.

[iii]Total volume of BNR reactor = volumes of anaerobic, anoxic, and aerobic zones.
$= \cancel{2631\ \text{m}^3} + 2631\ \text{m}^3 + 11560\ \text{m}^3$
$= 16822\ \text{m}^3$ (Sec. 13-11-6, Steps A, 1; A, 2; and A, 3).

Provide the sludge storage capacity for 1 day under sustained peak flow rate and BOD_5 loadings. Assume that the sustained flow rate and sustained BOD_5 factors are 2.5 and 1.5, respectively.[4]

Total solids produced under sustained loadings $= 1.5 \times 2.5 \times 2820$ kg/d (Sec. 13-11-5, Step D, 4)
$= 10{,}575$ kg/d

Provide 1 day storage in the clarifier

Total solids stored per clarifier $= 10{,}575$ kg/d/4 $= 2644$ kg

Total solids in the clarifier $= 2644$ kg (storage zone) $+ 4731$ kg (thickening zone) $= 7375$ kg

$$\text{Clarifier depth for solids storage} = \frac{7375 \text{ kg} \times 1000 \text{ g/kg}}{7000 \text{ g/m}^3 \times 1301 \text{ m}^2}$$

$$= 0.8 \text{ m}$$

4. Compute the total depth of clarifier.

Total depth of clarifier $= 3.0 \text{ m} + 0.5 \text{ m} + 0.8 \text{ m} = 4.3 \text{ m}$

Provide average side water depth in the clarifier $= 4.5$ m (14.8 ft)

For additional safety, provide a free board of 0.5 m:

Total depth of clarifier $= 5.0$ m

Step C: Detention Time

1. Calculate the volume of the clarifier.

$$\text{Average volume of the clarifier} = \frac{\pi}{4} \times (40.7)^2 \text{ m} \times 4.5 \text{ m}$$

$$= 5855 \text{ m}^3$$

2. Calculate detention time under different flow conditions.

$$\text{Detention time under average design flow plus recirculation}^{mmm} = \frac{5855 \text{ m}^3}{0.192 \text{ m}^3\text{/s} \times 3600 \text{ s/h}}$$

$$= 8.5 \text{ h}$$

$$\text{Detention time at peak design flow plus recirculation} = \frac{5855 \text{ m}^3}{0.403 \text{ m}^3\text{/s} \times 3600 \text{ s/h}}$$

$$= 4.0 \text{ h}$$

[mmm] A detention time of 8.5 h under average design flow is high. If phosphorus release is observed in the clarifier, the return sludge should be increased to reduce the sludge blanket.

Step D: Summary of Clarifier Design Information. The design information for the secondary clarifier is summarized in Table 13-18. The layout plan of the clarifier is provided in Figure 13-29.

Step E: Influent Structure. The influent structure consists of a central feed well. An influent pipe is installed across the clarifier that will discharge into the central feed well. The influent will pass under the baffle and then distribute uniformly throughout the tank. The details of the influent structure are shown in Figure 13-29.

Step F: Effluent Structure. The effluent structure consists of effluent baffle, V-notches, effluent launder, effluent box, and a pressure outlet pipe. The details of the effluent structure are shown in Figure 13-29. General discussion on weir types, as well as configurations of rectangular and circular basins, may be found in Chapter 12.

1. Select weir arrangement and dimensions of effluent launder, effluent box, and outlet sewer.

TABLE 13-18 Summary Design Information of the Secondary Clarifier

Item No.	Design Conditions	Design Value[a] SI Units	Design Value[a] U.S. Customary Units
1	Number of clarifiers	4	4
2	Surface area, each clarifier	1301 m^2	14,004 ft^2
3	Diameter	40.7 m	134 ft
4	Water depth	4.5 m	14.8 ft
5	Freeboard	0.5 m	1.6 ft
6	Average design flow plus recirculation	0.192 m^3/s	4.4 mgd
7	Peak design flow plus recirculation	0.403 m^3/s	9.2 mgd
8	Detention period		
	At average design flow	8.5 h	8.5 h
	At peak design flow	4.0 h	4.0 h
9	Overflow rate		
	At average design flow	12.8 $m^3/m^2{\cdot}d$	314 gpd/ft^2
	At peak design flow	26.8 $m^3/m^2{\cdot}d$	658 gpd/ft^2
10	Limiting solids flux (solids-loading rate)		
	At average design flow	47.8 $kg/m^2{\cdot}d$	9.8 $lb/ft^2{\cdot}d$
	At peak design flow	100.4 $kg/m^2{\cdot}d$	20.61 $lb/ft^2{\cdot}d$

[a] 1 $m^3/m^2{\cdot}d$ = 24.5424 gpd/ft^2; 1 $kg/m^2{\cdot}d$ = 0.2048 $lb/ft^2{\cdot}d$.

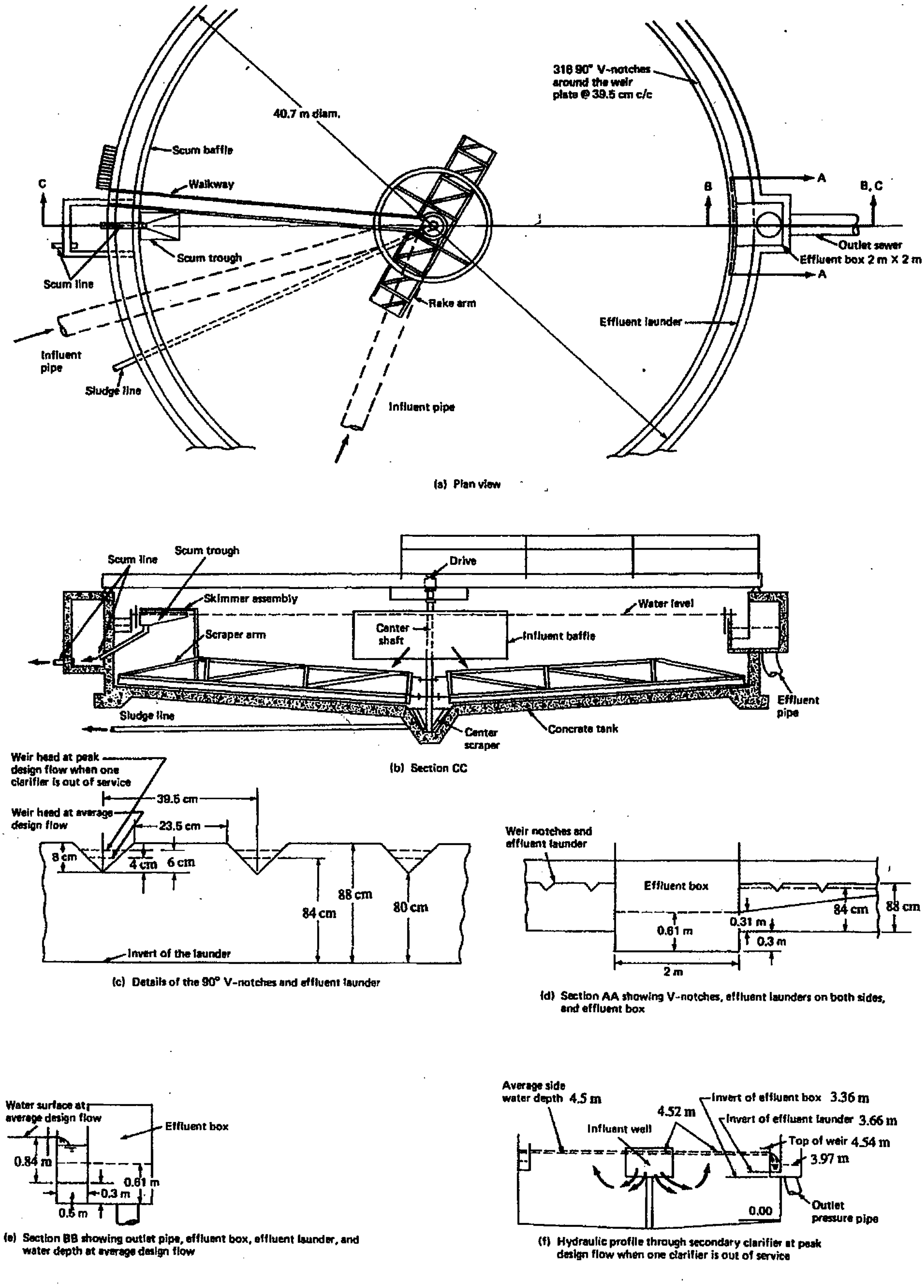


Figure 13-29 Design Details and Layout of Final Clarifier.

Provide 90° standard V-notches on the weir plate that shall be installed on one side of the effluent launder.

$$\text{Provide width of launder} = 0.5 \text{ m}$$
$$\text{Length of effluent weir plate} = \pi(40.7 \text{ m} - 1.0 \text{ m}) = 124.7 \text{ m}$$

Provide 8-cm-deep 90° V-notches at 39.5 cm center-to-center.

$$\text{Total number of notches} = \frac{124.7 \text{ m}}{39.5 \text{ cm/notch} \times (100 \text{ cm/m})^{-1}} = 316$$

2. Compute head over V-notch at average design flow.

$$\begin{array}{l}\text{Average design flow}\\ \text{from the clarifier}^{nnn}\end{array} = \text{Average design flow to aeration basin} - \text{MLSS wasted}$$
$$= 0.486 \text{ m}^3\text{/s} - [75.2 \text{ m}^3\text{/d} \times (86{,}400 \text{ s/d})^{-1}]$$
$$= 0.477 \text{ m}^3\text{/s}$$

$$\text{Flow per clarifier} = \frac{0.477 \text{ m}^3\text{/s}}{4} = 0.12 \text{ m}^3\text{/s per basin}$$

$$\begin{array}{l}\text{Flow per notch at}\\ \text{avg. design flow}\end{array} = \frac{0.12 \text{ m}^3\text{/s}}{316 \text{ notch}} = 0.00038 \text{ m}^3\text{/s/notch}$$

The head over V-notch is calculated from Eq. (12-1):

$$\text{Head over V-notch } H = \left[\frac{15}{8}\left(\frac{0.00038 \text{ m}^3\text{/s}}{0.6\sqrt{2 \times 9.81 \text{ m/s}^2} \times \tan 45}\right)\right]^{2/5}$$
$$= 0.037 \text{ m} = 3.7 \text{ cm} \approx 4 \text{ cm}$$

3. Compute head over V-notch at peak design flow when all four units are in operation.

$$\text{Flow per notch at peak design flow}^{ooo} = \frac{1.321 \text{m}^3\text{/s}}{4 \text{ clarifiers} \times 316 \text{ notch}}$$
$$= 0.00105 \text{ m}^3\text{/s per notch}$$

$$\text{Head over V-notch } H = \left[\frac{0.00105 \text{ m}^3\text{/s}}{8/15 \times 0.6\sqrt{2 \times 9.81 \text{ m/s}^2} \times \tan 45}\right]^{2/5}$$
$$= 0.056 \text{ m} = 5.6 \text{ cm} \approx 6 \text{ cm}$$

[nnn]Effluent over weir will not include return sludge, because it is returned from the bottom of the clarifier. The average design flow to aeration basin and MLSS wastes may be found in Sec. 13-11-6, Step C, 1 and Sec. 13-11-5, Step D, 2, respectively.

[ooo]Does not include return flow, and the volume of waste sludge is ignored.

4. Compute actual weir loading.

$$\text{Weir loading at average design flow} = \frac{0.12\ \text{m}^3/\text{s} \times 86{,}400\ \text{s/d}}{124.7\ \text{m}}$$

$$= 83.1\ \text{m}^3/\text{m·d} \text{ (less than the design loading of } 124\ \text{m}^3/\text{m·d)}$$

$$\text{Weir loading at peak design flow} = \frac{1.321\ \text{m}^3/\text{s} \times 86{,}400\ \text{s/d}}{4 \times 124.7\ \text{m}}$$

$$= 229\ \text{m}^3/\text{m·d} \text{ (less than the design loading of } 372\ \text{m}^3/\text{m·d)}$$

5. Compute the depth of the effluent launder.

Width of effluent launder = 0.5 m

Provide effluent box 2 m × 2 m.

Provide 0.8-m-diameter outlet pressure pipe. The pipe is an inverted syphon connected to a common junction box. The water surface elevation in the junction box is kept such that the depth of flow in the effluent box at peak design flow is maintained at 0.61 m. Provide invert of the effluent launder 0.3 m above the invert of the effluent box. The depth of the effluent launder is calculated from Eq. (11-17) (various terms in this equation have been defined in Chapter 11):

$y_2 =$ depth of water in the effluent box − invert height of effluent launder above the invert of effluent box

$$= 0.61\ \text{m} - 0.30\ \text{m} = 0.31\ \text{m}$$

$$b = 0.5\ \text{m}$$

Half of the flow divides on each side of the launder, therefore flow on each side of the launder[ppp] $= \dfrac{1.321\text{m}^3/\text{s}}{2 \times 4 \text{ clarifiers in operation}}$

$$= 0.17\ \text{m}^3/\text{s}$$

$$y_1 = \sqrt{(0.31\ \text{m})^2 + \frac{2 \times (0.17)^2}{9.81\ \text{m/s}^2 \times (0.5\ \text{m})^2 \times 0.31\ \text{m}}} = 0.42\ \text{m}$$

Provide 16 percent losses for friction, turbulence, and bends, and provide 31 cm additional depth to ensure free fall

$$\text{Total depth of the effluent launder} = (0.42\ \text{m} \times 1.16) + 0.31\ \text{m} = 0.80\ \text{m}$$

The water surface elevation in the clarifier at average design flow is kept (0.80 + 0.04) = 0.84 m above the invert of the effluent launder. See the details of effluent structure in Figure 13-29.

[ppp]Recirculation does not reach the effluent launder.

Step G: Sludge Collection System and Skimmer. The sludge collection system shall consist of a rotating rake structure with scraper blades that will scrape the settled sludge from the tank bottom to a sludge pocket located near the center of the basin. The fixed access bridge shall house the drive machinery and shall be supported by a column at the center of the tank. The skimmer shall remove the scum and deposit it into the scum trough.

Step H: Return Sludge Pumps. Four return sludge pumps each having a rated pumping capacity of 0.182[qqq] m^3/s (150 percent of design average flow per basin) shall be provided. Each pump shall be variable speed and provide independent operation of one clarifier. An identical pump (fifth pump) shall serve as a standby unit and will be cross-connected to serve all four clarifiers. A magnetic flow meter shall be provided in each force main to control the flow of the return sludge from each clarifier. A sonic sludge blanket meter shall be provided in each clarifier to control the pump speed.

The design and selection of pumping equipment is presented in Chapter 9. The flow-measuring systems are discussed in Chapter 10. The design engineers should consult these chapters to develop the specific design of return sludge and flow measurement devices mentioned above. The design is similar to that of a dry well centrifugal pump with flooded suction pipe.

Step I: Hydraulic Profile. The hydraulic profile for the secondary clarifier is shown in Figure 13-29. The profile is prepared for the peak design flow plus the recirculation. The head loss in the influent structure is small because the baffle at the central well offers little obstruction to the flow. The momentum equation [Eq. (11-18)] should be used to calculate the head loss caused by the baffle (the calculation procedure is presented in Chapter 11). The head loss calculations for the connecting pipings between the division box and the influent well are given in Chapter 21.

Step J: Effluent Quality. Based on the design criteria and equipment used in this Design Example, there is sufficient built-in safety and redundancy to perform under adverse conditions. The BNR facility will continue to meet the effluent quality standards under normal and adverse conditions. The effluent standards are given in Table 6-1 and the effluent quality is given in Table 13-13. The effluent quality is expected to be better than the values given here: BOD_5 = 10 mg/L, TSS = 10 mg/L, TN = 10 mg/L, Particulate Org.-N = 1 mg/L, Ammonia-N plus soluble Org.-N = 1 mg/L, TP = 1 mg/L.

13-12 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT ACTIVATED SLUDGE TREATMENT FACILITY

Many articles, reports, manuals, and books have been written on the control of a BNR facility. Proper control of these facilities includes a balance between anaerobic and an-

[qqq]Design flow to the aeration basin = 0.486 m^3/s (see Sec. 13-11-6, Step C, 1).

Rated pumping capacity of each pump $= \dfrac{0.486\ \text{m}^3/\text{s}}{4} \times 1.5 = 0.182\ \text{m}^3/\text{s}$.

oxic conditions, air supply, mixing, influent quality, return sludge, waste sludge, and sludge blanket maintained in the secondary clarifier. Common operation and maintenance problems that may develop at the BNR facility and procedures to correct them are summarized below.

13-12-1 Starting a New Plant

Before starting the plant, check to see that all mechanical equipment is in good working order: properly lubricated, filters clean, and all pipings free of debris. The following procedure should be followed:

1. Start treatment with a portion of incoming wastewater (one-third to one-fourth).
2. Use only a few units of the plant needed to handle the flow (one to two process trains).
3. Provide sufficient air to maintain dissolved oxygen in the mixed liquor between 2 and 4 mg/L.
4. Continue returning the entire sludge through the BNR reactors until the MLSS concentration reaches 1000–1500 mg/L. Start internal recycle when sufficient MLSS concentration builds up and nitrification starts.
5. Slowly increase the influent and bring other units into operation.
6. The normal plant operation shall be reached in 4–6 weeks. In cold climates the sludge growth may be slower than that in warmer climates.
7. If an existing activated sludge plant is available and a sufficient quantity of MLSS can be transported to the new facility, the start-up phase will be easier, and steps 1–3 can be eliminated. The reactor can be started (step 4) from the MLSS transported from the available plant.

13-12-2 Common Operating Problems and Suggested Solutions

There are many common problems that may occur in the BNR facility and the clarifier. Important problems and corrective procedures are listed below:[91,92]

1. A slightly pungent odor in the anaerobic reactor is the characteristic smell. A strong putrid, or hydrogen sulfide, odor is an indication of an excessive anaerobic zone, settling of solids, or trapped scum. Increase the return sludge and the mixer speed.
2. The ORP in the anaerobic zone should be well below −200 mv. Failure to reach low oxidation reduction potential (ORP) is an indication of insufficient anaerobic conditions. Reduce return sludge flow and adjust mixer speed to reduce surface turbulence.
3. Phosphorus release in the anaerobic reactor is essential for biological phosphorus removal. Orthophosphorus concentration well above 20 mg/L should be reached in the anaerobic reactor. Failure to reach good phosphorus release in the anaerobic reactor may be caused by (a) insufficient SCVFAs in influent (see Sec. 13-7-2), (b) the presence of an aerobic or anoxic condition, (c) insufficient HRT, and (d) low BOD_5 in the mixture of influent and return flow (see Sec. 13-7-2). Provide an anaerobic fermenter (see Sec. 13-7-2), reduce return flow, and reduce mixer turbulence.

4. An NO_3-N concentration above 0.1 mg/L is an indication of insufficient denitrification. Possible causes are high recycle ratio, insufficient HRT, excessive turbulence, low pH, and insufficient biodegradable organic carbon. Reduce the recycle ratio and adjust mixer speed to minimize surface turbulence.
5. Sludge floating to the surface of the clarifiers (bulking of sludge) may be caused by growth of filamentous organisms. Generally, the sludge volume index (SVI) is greater than 150, the food-to-MO ratio is well above 0.3/d, and microscopic examination of MLSS shows the presence of filamentous organisms (see Filamentous Growth and Sludge Bulking in Sec. 13-3-1). The following control measures may be taken: increase overall detention time in the anaerobic and anoxic zones by reducing return sludge and recycle ratios; increase DO in the aeration basin if it is less than 1 mg/L; increase pH to 7 (see Step 13 below): add nutrients (nitrogen, phosphorus, iron, etc.); add 5–6 mg/L of chlorine to the return sludge line until SVI is less than 150; add 5–6 mg/L hydrogen peroxide in the aeration basin until SVI is less than 120; or decrease food-to-MO ratio. This is achieved by (1) increasing the sludge age, (2) reducing sludge wasting, or (3) decreasing the food supply.

   Often, denitrification occurring in the secondary clarifier may be the cause of nitrogen bubbles attaching to sludge particles and sludge rising in clumps. The problem may be overcome by increasing the sludge return rate and DO in the aeration basin.
6. Turbid effluent (pin floc in effluent) but good SVI may be caused by excessive turbulence in the aeration basin or overoxidized sludge. The problem is easily overcome by reduced aeration or agitation, increased sludge wasting, or decreased sludge age. Turbid effluent may also be the result of anaerobic conditions in the aeration basin. The problem is easily solved by increasing aeration. Often, toxic shock loading may cause turbid effluent. Microscopic examination of MLSS will certainly show inactive protozoa. Reseeding from another plant may be necessary. Enforcement of industrial waste pretreatment must be done.
7. Very stable, dark tan foam on aeration tanks (which may not be broken by the sprays) may result from too long a sludge age. Increase the sludge wasting.
8. Thick billows of white sudsy foam on the aeration basin may be caused by low MLSS. A reduction in sludge wasting may help the problem.
9. Different concentrations of MLSS in different aeration basins may be the result of unequal flow distribution to each basin. Valves and gates should be adjusted to equally distribute the influent and return sludge.
10. Sludge blanket uniformly overflowing the weir may be caused by excessively high solids loading, peak flows overloading the clarifiers, unequal flow distribution on clarifiers, excessively high MLSS, and inadequate return sludge. Check the solids loading and overflow rates; distribute flows equally in all clarifiers; check return sludge pumps and pipings for malfunctioning or blockage; increase rate of return sludge to maintain at least 1 m clearwater zone in the clarifier; or increase sludge wasting.
11. Sludge blanket discharging over the weir in one portion of the clarifier may be the result of unequal flow distribution. Level the effluent weirs.
12. In diffused aeration systems, air rising in large bubbles or clumps in some areas is

an indication of faulty diffusers. Clean or replace diffusers, check air supply, and install filter ahead of blowers.

13. Low pH of MLSS (pH below 6.7) is an indication of high NH_4^+-N in the influent, causing destruction of alkalinity because of nitrification or acid wastes reaching the plant. Check the ammonia and nitrate concentrations in the effluent as well as the pH of the influent. Proper control measures include decreasing the sludge age by wasting or increasing the return sludge and internal recycle for enhanced denitrification and recovery of alkalinity, lime addition, or proper control of influent.
14. A low solids concentration in the return sludge (less than 8000 mg/L) may be caused by filamentous growth, a high return sludge rate, or excessive sludge wasting. Proper control measures include reducing sludge return rate, reducing sludge wasting, and raising DO. If microscopic examination indicates filamentous growth, follow the procedure given in Step 5.
15. Dead spots in the aeration basin may be caused by plugged diffusers or under-aeration. Check air supply and clean or replace diffusers.
16. High phosphorus in the effluent may be caused by the release of phosphorus in the sludge zone because of anaerobic condition. Increase the return sludge rate.

13-12-3 Routine Operation and Maintenance

The following procedures are necessary for routine operation and maintenance of the BNR facility and secondary clarifiers.

Anaerobic, Anoxic, and Aerobic Reactors

1. Inspect distribution boxes daily and clean baffle walls, weirs, and gates to remove solids.
2. Remove accumulations of debris from inlet channels and gates and outlet weir each day.
3. Keep a daily record of DO, phosphorus, ammonia, nitrate, pH, and MLSS concentrations in anaerobic, anoxic, and aerobic reactors; SVI; and sludge age. If any unusually high or low values are found, take corrective measures.
4. Clean all vertical walls and channels by squeegee on a daily basis.
5. Hose down and remove wastewater spills without delay.
6. Inspect gratings and exposed metal during daily cleanup for signs of corrosion and deterioration of paint.
7. Prepare lubrication chart for mechanical equipment according to manufacturer's recommendations.
8. Drain each BNR process train annually to inspect underwater portions of the concrete structure, pipings, and the like. Replace or repair all defective parts. Patch defective concrete, and repaint all clean metal surfaces as required.

Secondary Clarifier

1. Remove accumulations from the influent baffles, effluent weirs, scum baffles, and scum box each day.

2. Observe sludge return from individual clarifier, and adjust the flow rate as required from laboratory tests.
3. Determine sludge level and adjust waste sludge pump as necessary.
4. Observe operation of scum pump and provide hosing as necessary.
5. Clean daily all inside exposed vertical walls and channels by a squeegee.
6. Inspect distribution box and clean weirs, gates, and walls as necessary and remove all settled solids. Also check flow to all clarifiers.
7. Inspect effluent box, and clean weirs and walls as necessary. Measure the head over the weirs daily.
8. Hose down and remove wastewater sludge and spills without delay.
9. Check electrical motors for overall operation, bearing temperature, and overload detector twice each day.
10. Check oil level, grease reducer, and rollers on the skimmer each week.
11. Check oil level for turntable bearings each week and refill as required.
12. Grease the main bearings each week.
13. Change the oil in the gear reducer quarterly.
14. Drain each clarifier annually to inspect the underwater portion of the concrete structure and mechanism. Inspect mechanical equipment for wear and corrosion, and apply protective coating. Inspect concrete structure and patch defective areas.
15. Inspect sludge collection and other equipment annually for indication of corrosion. Clean and paint all metal works as necessary.

13-13 SPECIFICATIONS

The general specifications of pumping equipment, flow-measuring devices, and a sedimentation facility have already been presented in Chapters 9, 10, and 12. Therefore, specifications for return sludge, waste sludge and scum pumps, flow meters, and secondary clarifiers are not given in this section. The design engineer should consult Chapters 9, 10, and 12 to develop the specifications for these facilities.

The general specifications for the mixers in anaerobic and anoxic basins and aeration equipment are presented in this section. These specifications should be used only as a guide. Detailed specifications for each unit or components should be prepared in consultation with the equipment manufacturers.

13-13-1 General

This item governs the furnishing and installation of equipment required in the anaerobic and anoxic reactors and aeration basin. The mixers in anaerobic and anoxic reactors shall be vertical turbine type. The aeration equipment shall be composed of a Dacron sock diffuser. Each diffuser assembly shall be arranged for raising the entire assembly to an accessible position for servicing without dewatering the tank and shall be provided with positive means for both regulation and complete shutoff of the air supply.

The equipment manufacturer shall be experienced in the design, fabrication, installation, and operation of aeration equipment and shall submit a list of installations of this type of operation over a reasonable period of time.

13-13-2 Mixers

Provide identical mixers in anaerobic and aerobic zones. A total of six mixers will be needed in each process train. The mixer shall be vertical shaft with an electronic frequency drive for the purpose of providing variable-speed control of the electronic motor. The mixer shall be a turbine type driven by a 2.5-kW, 1200-rpm, totally enclosed electronic motor for operation on 230-V, 60-cycle, three-phase current. The frequency inverter shall be compatible and enclosed in NEMA 4 remote operation system.

The mixer shaft shall extend 6 m below the mixer mounting base and carry a 1.2-m-diameter *Laserfoil® impeller* operating at a nominal output speed of 45 rpm. The impeller will be supplied with stabilizing fins to provide shaft stability at all liquid levels. Stainless steel shall be used for fabrication of the mixer shaft and impeller assembly. The mixer driver shall be shipped completely assembled and aligned on a rectangular mounting plant, ready for mounting to the mixer support.

13-13-3 Aeration System

Diffusers. Each Dacron sock diffuser shall consist of a tubular Dacron cloth sheath having an internal dimension of 7.5 cm diameter, closed on one end and open at the other to fit over a formed rod, and clamped by a stainless steel clamp to an adapter fitting. The formed rod shall be approximately 61 cm long, 0.63-cm mild carbon steel attached to the adapter, and the whole assembly dipped in vinyl. The differential pressure across the diffuser media when the tube is discharging air at the rate of 0.2 m^3/min with 200 mm of water over the center line of the tube shall not be greater than 300 mm.

Diffuser Tube Headers. Each diffuser tube header shall comprise two lengths of galvanized fabricated steel pipe, flange connected to a T fitting, which is screw-connected to the hanger pipe. Each diffuser tube header shall have bosses drilled and tapped to accommodate the diffuser tube assemblies as detailed on the drawings.

Hanger Feed Pipes. Each hanger pipe shall comprise an upper section and a lower section of galvanized iron pipe jointed by a knee joint. The knee joint shall be equipped with wearing rings to provide an airtight connection. The hanger pipe shall serve as a support and air feed pipe to the diffuser tube header.

Swing Joint Assembly. Each swing joint assembly shall comprise a galvanized steel feeder manifold and one duplex elbow fitting swing joint with built-in valve. One feeder manifold shall serve one or two air diffusion units as detailed on the plans. The feeder manifold shall be attached to the air main by means of a flanged connection. The movable member of the swing joint shall be connected to the hanger pipe by a cast iron screwed connection. Elbow fittings, anchor, interlock, wearing rings, etc., shall be provided for stationary and swing joints and airtight connections.

Portable Hoist. One portable hoist for raising the air diffusion units from the aeration basin shall be provided.

Blowers Assembly. The contractor shall furnish and install five centrifugal blowers with motors and accessories as indicated in plans and as covered herein.

Blower. Each blower will be of the multistage centrifugal type, capable of compressing 230 m^3/min of air to a discharge pressure of 57 kPa[rrr] (8.25 lb/in.^{2} or 0.56 atm) when operating at an elevation of 500 m and an air temperature of 30°C. When volumetric capacity is reduced by at least 30 percent, each blower under specified inlet conditions shall develop at least 3.5 kN/m^2 (0.5 lb/in.^{2}) above specified discharge pressure and shall not be in surge. The manufacturer shall submit a certified test report attesting to the date and place of testing and accuracy achieved.

Motor. Five electric motors for the driving blower shall each be horizontal squirrel cage induction type 277 kW, 460 V, three-phase, 60 Hz, 1.15 service factor and shall be designed for full-voltage starting. Motor shall operate at a speed not to exceed 1720 rpm. Motor shall be subjected to standard commercial tests, and certified copies of test data, together with a certified statement of compliance with minimum specified efficiency, shall be furnished.

Each blower and motor unit shall be mounted on a single heavy-strength steel or cast iron frame, properly cross-braced to form a rigid support for the entire unit.

Air Filter. The contractor shall furnish and install an air filter of the two-stage, high-efficiency type for an air flow of 920 m^3/min. The filter shall consist of an approved renewable media filter. The overall efficiency of the filter shall average not less than 97 percent on atmospheric dust. The initial resistance to rated air flow shall not exceed 50 mm water.

Accessories. The following accessories shall be provided for each blower: a vibration-sensing device; alarm circuits, control panel, and check valve; butterfly valves; manometer (calibrated to read 0–70 kN/m^2 and suitable for wall mounting) and thermobarometer; and intake silencer of sufficient capacity to handle the air requirements without unreasonable pressure drop.

13-14 PROBLEMS AND DISCUSSION TOPICS

13-1 Using the results of the first iteration of the material mass balance analysis given in Table 13-12, perform the second iteration. Compare the results with those given in Table 13-13.

13-2 Four completely mixed bench-scale reactors were operated. The reactors were identical in design and operation. The only difference was in the mixed liquor suspended solid (MLSS) concentration that was maintained in each reactor. When the reactors reached a steady-state condition, the data collection began. Each day the following measurements were made:

(a) total volume of the effluent
(b) TVSS in the effluent
(c) soluble COD and BOD_5 of the feed

[rrr]kPa = kN/m^2.

(d) soluble COD and BOD_5 of the effluent
(e) MLSS and MLVSS in the reactor
(f) Certain volume of MLSS from each reactor was wasted to give the desired MLSS concentration that was maintained into each reactor. An equal volume of distilled water was added in each reactor.

From the results, ratios of MLVSS/MLSS, $BOD_{5,\ influent}/COD_{influent}$, and $BOD_{5,\ effluent}/COD_{effluent}$ were calculated. The 7-day average experimental results at each MLSS concentration are summarized in Table 13-19. The volume of each reactor, including the portion partitioned for settling, is 10.91. Determine the biological kinetic coefficient, k, y, k_d, and K_s based on MLVSS and BOD_5 results.

13-3 A completely mixed aerated lagoon is treating 4500 m^3/d municipal wastewater. The aeration period is 4 d. Estimate the soluble BOD_5 and total BOD_5 by considering the biological solids. Use the following data and the following equations:

$$\text{Total } BOD_{5,\ influent} = 200 \text{ mg/L}$$
$$\text{Soluble } BOD_{5,\ influent} = 150 \text{ mg/L}$$
$$TSS_{influent} = 240 \text{ mg/L}$$
$$k = 4.0\ d^{-1},\ K_s = 80 \text{ mg/L},\ Y = 0.45,\ k_d = 0.05\ d^{-1}$$

$$\frac{BOD_5}{BOD_L} = 0.68$$

$$\frac{\text{Biodegradable solids}}{TSS} = 1.42$$

$$\frac{BOD_L}{\text{Biodegradable solids}} = 0.65$$

$$S = \frac{K_s(1 + \theta k_d)}{\theta(Yk - k_d) - 1} \quad \text{and} \quad X = \frac{Y(S_o - S)}{(1 + k_d\theta)}$$

13-4 Prove the expressions representing S and X given in Problem 13-3 from any kinetic equations given in Sec. 13-2-2.

13-5 Determine (a) the dimensions of an aerated lagoon to treat 300 m^3/d wastewater having a soluble BOD_5 of 150 mg/L, (b) the power requirements of the aerators under the field condition (average temperature 25°C and pressure 1 atm), and (c) number of aerators. Effluent soluble BOD_5 is not to exceed 15 mg/L. Use the following data and the kinetic equations:

The $L:W$ ratio is 4 : 1, water depth = 4.5 m, and side slope is 1 : 1.

$$k = 5.0\ d^{-1},\ K_s = 100 \text{ mg/L},\ Y = 0.6,\ k_d = 0.05\ d^{-1}$$

Theoretical oxygen required = 1.47 × BOD_5 removed

$$N = N_o\left[\frac{C'_{sw}\beta F_a - C}{C_{sw}}\right] 1.024^{(T-20)}\alpha$$

where

N_o = kgO_2/kW·h transferred in water at 20°C and at zero dissolved oxygen
N = kgO_2/kW·h transferred under field condition

TABLE 13-19 Results of Experimental Data for Example 13-2

Reactor	MLSS Maintained (mg/L)	MLVSS/ MLSS (%)	Influent COD (mg/L)	Influent BOD_5/COD (%)	Effluent COD (mg/L)	Effluent BOD_5/COD (%)	Average Effluent Volume Each Day (L)	Total Average MLVSS at the End of Each Day[a] (mg/L)
1	2250	73.0	138.4	70.1	25.0	20	37.0	1692.3
2	1850	73.0	149.7	70.8	25.0	27	37.0	1422.5
3	1350	72.5	153.3	71.1	32.3	28	36.4	1067.2
4	1100	77.5	161.6	71.8	31.7	35	35.6	951.0

[a]Includes VSS lost in the effluent. Mass of VSS lost in the effluent was converted to the equivalent concentration in the reactor volume.

Available aerator sizes are 5, 10, and 15 kW. Other constants are defined in Eq. (13-63). (Use $C = 2.5$ mg/L, $\alpha = 0.9$, and $\beta = 0.9$, and N_o from the manufacturer's data for selected aerator is 1.2 kgO_2/kW·h. Show the aerator arrangement.

13-6 MLSS concentration in an aeration basin is 3000 mg/L. After settling it in a 1-L graduated cylinder, the settled volume is 300 mL. Calculate SVI.

13-7 2000 m^3/d flow from a dairy industry is treated in an activated sludge plant. The BOD_5 is 300 mg/L, and most of it is in soluble form. Calculate (a) the volume of aeration basin, (b) aeration period, (c) return sludge, (d) volume of waste activated sludge at sp. gr. of 1.025, food-to-MO ratio, and oxygen requirement. Use the following data:

$$\theta_c = 10 \text{ d}$$
$$Y = 0.55$$
$$k = 4.0 \text{ d}^{-1}$$
$$K_s = 95 \text{ mg/L}$$
$$k_d = 0.04/\text{d}$$
$$\text{BOD}_5 = 0.7 \text{ BOD}_L$$
$$\text{Effluent BOD}_5 = 30 \text{ mg/L}$$
$$\text{TSS in effluent} = 30 \text{ mg/L}$$

Biological solids are 72 percent biodegradable.

$$1 \text{ g of biodegradable solids} = 1.42 \text{ g utilmate BOD}$$
$$\text{MLVSS maintained} = 2800 \text{ mg/L}$$
$$\text{MLVSS/MLSS} = 0.68$$

Return sludge has 12,000 mg/L TSS.

13-8 An aerobic stabilization pond is designed to treat 500 m^3/d wastewater from a community. The BOD_5 loading is 60 kg/ha·d. Calculate the area, volume, and detention time. Average depth of the stabilization basin is 1 m. Also calculate TSS and total BOD_5 of the effluent. Assume 80 percent TSS in the effluent is caused by algae. Use the following data:

$$\text{Average algae concentration} = 100 \text{ mg/L}$$
$$\text{Biodegradable portion of algae cell} = 65 \text{ percent}$$
$$\text{Soluble BOD}_5 \text{ in effluent} = 10 \text{ mg/L}$$
$$\text{BOD}_5/\text{BOD}_L = 0.68$$
$$\text{Influent BOD}_5 = 250 \text{ mg/L}$$
$$1 \text{ g of biodegradable solids} = 1.42 \text{ BOD}_L$$

13-9 A trickling filter is designed to treat 500 m^3/d primary settled wastewater having a BOD_5 of 150 mg/L. The BOD_5 loading is 0.5 kg/m^3·d. Calculate the area and depth of the trickling filter and the recirculation ratio. Use the following data:

$$\text{Total BOD}_5 \text{ effluent} = 20 \text{ mg/L}$$
$$\text{TSS, effluent} = 20 \text{ mg/L}$$

TSS, in effluent is 60 percent biodegradable, and each gram of biodegradable solids = 1.42 g BOD_L.

$$\text{BOD}_5/\text{BOD}_L = 0.68$$
$$L_D/L_o = 10^{-KD}$$

where

L_D = soluble BOD_5 at depth D in meters, mg/L
L_o = applied soluble BOD_5 at the filter after dilution, mg/L
K = rate of BOD_5 removal. Use $K = 0.49$/m in this problem.

Filter efficiency = 75 percent

13-10 A diffused aeration basin requires 1200 kg of O_2 per day. The oxygen transfer efficiency of the diffusers is 8 percent, and water depth over diffusers is 4.5 m. Determine the power requirement of the compressor. Assume inlet temperature and pressure are 30°C and 1 atm. The efficiency of the compressor is 77 percent. Head loss in piping, air filter, silencer, diffuser losses and allowance for diffuser clogging, etc., add up to 1.7 m of water.

13-11 A BNR facility is designed for a flow of 8500 m^3/d. The influent TP is 5.5 mg/L, and required effluent TP concentration is less than 1 mg/L. The BOD_5 in influent is 150 mg/L, and the ratio of BOD_5/COD is 0.48. Calculate the achievable TP in the effluent and minimum volume of the anaerobic zone. Assume TSS in effluent is 10 mg/L or less, the TVSS/TSS ratio is 0.8, and the representative chemical formula for the VSS portion of biomass is $C_{60}H_{87}O_{23}N_{12}P$.

13-12 An anoxic reactor is designed for denitrification. Calculate the design values of HRT, SRT, and $p_{x,\,DN}$. Use the following data:

Influent TKN = 30 mg/L
Influent BOD_5 = 150 mg/L
Effluent NO_3^--N = 9 mg/L
Total effluent BOD_5 = 12 mg/L
Effluent soluble NH_4^+-N+soluble Org.-N = 1.5 mg/L

The fraction of biomass that effectively carries out denitrification in the reactor = 0.3

The MLVSS in the reactor = 3200 mg/L

Assume that all typical values of carbon and nitrate utilization given in Sec. 13-11-5, Step B, apply to this problem, except the following values: $Y_{BOD_5} = 0.55$ gVSS/gBOD_5, $Y_{DN} = 0.5$ gVSS/gNO_3^--N, and operating temperature = 12°C.

13-13 A combined BOD_5 removal and nitrification in a suspended growth reactor is designed. Calculate the design values of HRT, SRT and $P_{x,\,BOD_5}$, $P_{x,\,N}$, ratio percent of nitrification in the reactor, and concentrations of NH_4^+-N and NO_3^--N in the effluent. Use the following data:

Influent TKN = 30 mg/L
BOD_5 = 200 mg/L
Total effluent BOD_5 and TSS = 10 mg/L
Effluent soluble Org.-N = 0.5 mg/L
MLVSS = 3500 mg/L

Assume that all typical values of BOD_5 removal and nitrification given in Sec. 13-11-5, Step C, apply to this problem, except the following values:

$$Y_{BOD_5} = 0.55 \text{ VSS/gBOD}_5,\ Y_N = 0.25 \text{ gVSS/gNH}_4^+\text{-N nitrified}$$

Specific growth rate for nitrification = 0.5 d^{-1}

$$DO_{min} = 2.5 \text{ mg/L},\ T_{min} = 12°C, \text{ and } \theta_{c,N} \text{ governs the design}$$

13-14 In a denitrification facility, 28 mg/L NO_3^--N is denitrified. Calculate the theoretical concentration of methanol required.

13-15 In a biological nitrification-denitrification facility, 21 mg/L NH_4^+-N is nitrified and then 50 percent nitrate is denitrified. Calculate the alkalinity after nitrification and after denitrification. Assume that alkalinity of raw wastewater is 200 mg/L as $CaCO_3$.

13-16 The dimensions of the final clarifier in the Design Example were developed using solids concentrations and settling rate data given in Figure 13-23 and Table 13-17. The return sludge rate of 0.292 m^3/s was obtained from the aeration basin mass balance of solids in influent, return sludge, and the mixed liquor. Calculate from the clarifier data the underflow concentration at this return sludge rate.

13-17 The hydraulic profile through the aeration basin in the Design Example was developed at peak design flow plus recirculation. Develop the hydraulic profile at average design flow plus recirculation when one basin is out of service. The water depth in the effluent box is 1.04 m.

13-18 The hydraulic profile through the final clarifier in the Design Example was developed at peak design flow plus recirculation. Develop the hydraulic profile at peak design flow plus recirculation when one clarifier is out of operation. The depth of water in the effluent box is 1.04 m.

13-19 Describe various modifications of activated sludge process. List the advantages and disadvantages of each process.

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14

Disinfection

14-1 INTRODUCTION

Wastewater contains many types of human enteric organisms that are associated with various waterborne diseases. Typhoid, cholera, paratyphoid, and bacillary dysentery are caused by bacteria, and amebic dysentery is caused by protozoa. Common viral diseases include poliomyelitis and infectious hepatitis. Disinfection refers to selective destruction of disease-causing organisms in the water supply or wastewater effluent.[a] Chlorination of the water supply has been practiced since about 1850. Presently, chlorination of both water supply and wastewater effluent is an extremely widespread practice for the control of waterborne diseases. However, chlorination may result in the formation of chlorinated hydrocarbons, some of which are known carcinogens. Therefore, alternate methods of disinfection are receiving a great deal of attention.

In this chapter various alternative methods of disinfection, the advantages and disadvantages of chlorination, chlorine chemistry, ozonation, and ultraviolet radiation are presented. Step-by-step design procedures of a UV disinfection facility are covered in the Design Example.

14-2 METHODS OF DISINFECTION

Methods of disinfection broadly fall into three major categories: (1) physical, (2) radiation, and (3) chemical.

14-2-1 Physical Methods

Physical methods of disinfection include heat (pasteurization) and ultraviolet radiation. Pasteurization of milk, dairy products, and beverages has been used for decades. Pasteurization of sludge is also used. However, pasteurization is not applicable to water and wastewater because of the high energy cost in heating large volumes of liquid. Ultraviolet light is an excellent disinfectant and does not leave any known residual chemicals. Special mercury lamps have been developed that emit ultraviolet rays over thin films of water. The efficiency of ultraviolet radiation depends on the (1) depth of penetration, (2)

[a]The term *sterilization* is applied to the complete destruction of all organisms. Pasteurization is selective destruction of undesired organisms by heat.

time of contact, and (3) turbidity or suspended solids that may reduce the effective depth of penetration. Sunlight has also been effectively used for disinfection.[1]

14-2-2 Gamma Radiation

Gamma rays are emitted from a radioactive source, such as cobalt 60, and are effectively used to disinfect or sterilize water, wastewater, and sludge. These rays have high penetrating power. The method is reliable and does not leave residual chemicals in the effluent. In sludges the gamma rays alter the makeup of sludge to benefit dewaterability and stability. The disadvantages of the system include the radiation emitted as a threat to safety.

14-2-3 Chemical Agents

Many types of chemical disinfectants are used for different applications. These include (1) oxidizing agents, such as halogens (chlorine, bromine, and iodine), ozone, hydrogen peroxides, potassium permanganate; (2) alcohols; (3) phenol and phenolic compounds; (4) heavy metals; (5) quarternary ammonium compounds; (6) soaps and synthetic detergents; and (7) alkalies and acids. Among all these chemicals, oxidizing agents are commonly used for disinfection of water and wastewater. Chlorine and its compounds are most universally used for disinfection of a water supply and wastewater effluents. Bromine and iodine occasionally are used for swimming pool water but have not been used for treated wastewater.[2] Ozone is a highly effective disinfectant and is gaining popularity in spite of the fact that it does not build any residual and must be generated at the site. However, it produces less undesirable reaction products and is very effective in destroying odors and color-causing compounds in water. Hydrogen peroxide is used for odor control and to inhibit the growth of microorganisms in the collection system. It is a strong oxidant but a poor disinfectant. It neither builds residual nor produces undesirable reaction products in water. Potassium permanganate ($KMnO_4$) is also a powerful oxidizing agent; however, its use in effluent treatment is very limited.

14-3 KINETICS OF DISINFECTION

The disinfection efficiency of physical or chemical agents depends upon the following major factors: (1) concentration of dosage or intensity of disinfection agent, (2) contact time, (3) temperature, (4) number of organisms, (5) type of organisms, and (6) other impurities in the liquid (color, suspended solids, and other contaminants). A number of analytical and empirical models have been proposed to express the rate of kill of microorganisms as a function of different conditions. Among these are Chick's law and many modifications.[2-4] Many of these equations are summarized in Table 14-1.

14-4 DISINFECTION WITH CHLORINE

Chlorination has received wide application in water and wastewater treatment. It is used for many applications, including disinfection, taste and odor control, and color removal.

TABLE 14-1 Equations Used to Express the Rate of Kill of Microorganisms under Different Conditions

Eq. No.	Equation	Description
(1)	$N_t/N_0 = 1/(1 + kt_d)$	Die-off in natural body or treatment unit. The die-off coefficient for different situations (lakes, rivers, ponds, basins) may vary greatly.
(2)	$\ln N_t/N_0 = -kt$	The first-order model for die-off with time at a given concentration of disinfectant
(3)	$\ln N_t/N_0 = -kt^m$	Rate of kill may decrease ($m < 1$) or increase ($m > 1$) with time.
(4)	$C^n t_p = \text{constant}$	Relationship for a given kill and disinfectant concentration: $n > 1$, contact time is more important than dosage. $n = 1$, the effect of contact time and dosage are about equal.
(5)	$C^q N_p = \text{constant}$	Relationship on the initial number of microorganisms and desired reduction level
(6)	$k_T = k_{20}\, \beta^{(T\text{-}20)}$	Arrhenuis relationship for increasing kill at higher temperature

β = constant found experimentally, which depends upon temperature.
C = concentration of disinfectant, mg/L.
k = die-off coefficient or rate constant, t^{-1}. k_{20} and k_T are the values at 20°C and at temperature T°C.
m, n and q = constants obtained experimentally.
N_0, N_t = number of microorganisms present initially and at time t.
N_p = number of microorganisms reduced by a given percentage kill.
t_d = hydraulic detention time, d.
t_p = time required to achieve a constant percentage kill, e.g., $N_t/N_0 = 0.99$.

Source: Adapted in part from Refs. 2-7.

Chlorine is also used for oxidation of ammonia, iron, manganese, and sulfide, as well as for BOD removal. Chlorine is cheap, effective, available in large quantities, nontoxic in low concentrations to higher forms of life, and builds up a residual. The basic disadvantages of chlorine include acid generation (HCl); buildup of total dissolved salts; and formation of potentially carcinogenic halogenated organic compounds.

14-4-1 Chlorine Compounds

The most common chlorine compounds used in water and wastewater treatment plants are chlorine gas (Cl_2), calcium hypochlorite [$Ca(OCl)_2$], sodium hypochlorite (NaOCl), and chlorine dioxide (ClO_2). Important properties and common applications of chlorine compounds and of ozone and hydrogen peroxide are compared in Table 14-2. Selection of any of these compounds depends on the size of the treatment facility, objectives, economics, and safety considerations.

TABLE 14-2 Comparison of Important Properties and Common Applications of Chlorine Compounds, Ozone, and Hydrogen Peroxide

Characteristics	Chlorine	Sodium Hypochlorite	Calcium Hypochlorite	Chlorine Dioxide	Ozone	Hydrogen Peroxide
Chemical formula	Cl_2	NaOCl	$Ca(OCl)_2$	ClO_2	O_3	H_2O_2
Form	Liquid, gas	Solution	Powder, pellets, or 1 percent solution	Gas	Gas	Liquid
Containers for shipping	45.4-, 68- and 907-kg cylinders; tank cars	4.9- to 7.6-m^3 tanks; tank cars	45- to 360-kg drums	On-site generation	On-site generation	Fiberglass drums 114 to 208 tank cars
Commercial strength, percent	100	12–15	70	Up to 0.35	2	35–70
Stability	Stable	Light yellow liquid, unstable	Stable	Greenish-yellow gas, explosive	Unstable	Stable
Toxicity to micro-organisms	High	High	High	High	High	Medium

Oxidation potential at 25°C, volt	−1.36	−1.49 (HOCl)	−1.49 (HOCl)	−0.95 (aq)	−2.07	−1.76
Hazards associated with handling and use	High	Medium	Medium	High	High	Medium
Corrosion	High	Medium	Medium	High	High	Low
Deodorizing	High	Medium	Medium	High	High	High
Cost	Low	Medium	Medium	Medium	High	High
Common applications	Control of slime growth, H_2S, odor, BOD reduction, fly control, sludge bulking and foaming control, ammonia oxidation, disinfection	Control of slime growth, disinfection	Control of slime growth, disinfection	Control of slime growth and odor, disinfection	Odor control, BOD reduction, oxidation of refractory organic compounds, disinfection	Control of slime growth, H_2S, odor, sludge bulking

14-4-2 Chlorine Chemistry

Free Chlorine Residual. A number of reactions can occur when chlorine is added to the wastewater. The rate and efficiencies are dependent on temperature, pH, buffering capacity, and the form in which the chlorine is supplied. Chlorine in aqueous solution produces hypochlorus acid and hypochlorite ion [Eqs. (14-1) and (14-2)]:

$$Cl_2 + H_2O \rightleftharpoons \underset{\text{Hypochlorous acid}}{HOCl + H^+ + Cl^-} \tag{14-1}$$

$$HOCl \rightleftharpoons \underset{\text{Hypochlorite ion}}{H^+ + OCl^-} \tag{14-2}$$

The quantity of HOCl and OCl^- that is present in water is called free chlorine residual. The relative distribution of HOCl and OCl^- is important in chlorination because disinfecting power of HOCl is about 40–80 times greater than that of OCl^-. This is why wastewaters with lower pH are easier to disinfect by chlorination. The relative distribution of HOCl and OCl^- varies with temperature and pH. At 20°C, the relative distributions with temperature are reported in Table 14-3.

Free chlorine is also added in water from hypochlorite salts:

$$Ca(OCl)_2 + 2H_2O \rightleftharpoons 2HOCl + Ca(OH)_2 \tag{14-3}$$

$$NaOCl + H_2O \rightleftharpoons HOCl + NaOH \tag{14-4}$$

As seen from the chemical Eqs. (14-1)–(14-4), chlorine gas lowers the pH while hypochlorite in solution raises the pH. High pH favors the formation of OCl^-, which is much less effective than HOCl. Therefore, for an equal amount of chlorine added to poorly buffered effluents, a higher disinfection is achieved with chlorine gas than hypochlorite solutions. Hypochlorite solutions will also add additional dissolved solids to the effluent.

Combined Chlorine Residual. Chlorine reacts readily with ammonia to form three types of chloramines: mono-, di-, and trichloramines:

TABLE 14-3 Relative Distribution of HOCl and OCl^- with pH at 20°C

pH	HOCl (%)	OCl^- (%)
6.0	96.8	3.2
7.0	75.2	24.8
7.5	49.1	50.9
8.0	23.2	76.8
9.0	2.9	97.1

$$NH_3 + HOCl \rightarrow NH_2Cl \text{ (monochloramine) } H_2O \tag{14-5}$$
$$NH_2Cl + HOCl \rightarrow NHCl_2 \text{(dichloramine)} + H_2O \tag{14-6}$$
$$NHCl_2 + HOCl \rightarrow NCl_3 \text{ (trichloramine)} + H_2O \tag{14-7}$$

Combined residual chlorine is caused by chloramines. In this form chlorine has a lower disinfecting property.

Chlorine dioxide (ClO_2) has some unusual properties. It does not react with ammonia, and its disinfection properties are not fully known yet. A discussion of chlorine dioxide and its uses in water and wastewater may be found in Refs. 3 and 4.

Breakpoint Chlorination. When chlorine is added to water, it is consumed in oxidizing a wide variety of compounds present in the water. No chlorine residual can be measured until the chlorine demand is satisfied. Then chlorine reacts with ammonia, producing combined residual. Combined chlorine residual increases with additional dosages until a maximum combined residual is reached. Further addition of chlorine causes a decrease in combined residual. This is called *breakpoint chlorination* [Eqs. (14-8)–(14-11)]. At this point the chloramines are oxidized to oxides of nitrogen or other gases.

$$NH_2Cl + NHCl_2 + HOCl \rightarrow N_2O + 4HCl \tag{14-8}$$
$$4NH_2Cl + 3Cl_2 + H_2O \rightarrow N_2 + N_2O + 10HCl \tag{14-9}$$
$$2NH_2Cl + HOCl \rightarrow N_2 + H_2O + 3HCl \tag{14-10}$$
$$NH_2Cl + NHCl_2 \rightarrow N_2 + 3HCl \tag{14-11}$$

After breakpoint chlorination is reached, free chlorine residual develops at the same rate as applied dosage. A typical breakpoint chlorination curve is shown in Figure 14-1.

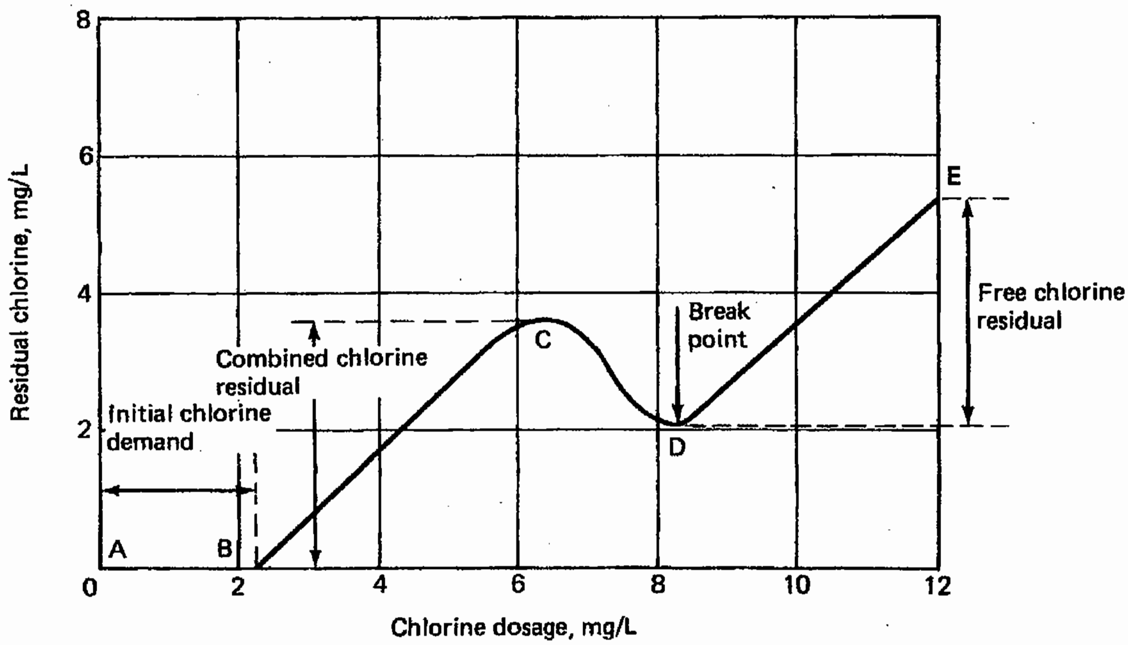


Figure 14-1 Chlorine Residuals and Breakpoint Chlorination Curve.

TABLE 14-4 Chlorine Dosages for Proper Disinfection of Wastewater Effluents

Effluent from	Dosage Range (mg/L)
Untreated wastewater (prechlorination)	6–25
Primary sedimentation	5–20
Chemical precipitation	3–10
Trickling filter	3–10
Activated sludge	2–8
Multimedia filter following activated sludge plant	1–5

Source: Adapted in part from Ref. 5.

Disinfection Efficiency. The disinfection efficiency of chlorine depends on the following factors: (1) contact time, (2) chlorine dosage, (3) temperature, (4) pH, (5) nature of liquid and suspended matter, and (6) type and number of organisms. For disinfection of wastewater, a chlorine residual of 0.5 mg/L after 20–30 min of contact period is required. The effluent quality to be disinfected must be evaluated for chemical dosage and contact period. Organic matter will react with chlorine, thus reducing its effectiveness. Furthermore, the turbidity will also reduce effectiveness by adsorption and by protecting entrapped organisms. The typical chlorine dosages for proper disinfection of wastewater effluents are summarized in Table 14-4.

The type and number of organisms also affect the effectiveness of the disinfectants. The spores of organisms are much more resistant than the organisms themselves. Also, the larger the number of organisms is longer is the time required for a given kill. Each person discharges from 100 to 400 billion coliform organisms per day, in addition to other bacteria (26–100 $\times 10^6$ per 100 mL of raw wastewater). Thus, the presence of coliform organisms in water is taken as an indication of pathogens in water.[2] The coliform bacteria are members of the family Enterobacteriacae and include many genera. The *E. coli* species appears to be most representative of fecal contamination (see Sec. 3-4-4). Total coliform remaining in primary and secondary effluents at different chlorine residuals are summarized in Table 14-5. These results are obtained after 30 min of contact time, and assuming primary and secondary effluents contain 35×10^6 and 1×10^6 total coliform per 100 mL samples.[5] Several relationships have been developed that give the number of organisms remaining at different contact time, disinfectant concentration, temperature, and so on. These relationships are given in Table 14-1. Other relationships may be found in Refs. 2–4 and 6.

Disinfection By-products. The reaction of free chlorine with many organic compounds in the effluent may produce halogenated organics or organic halides (TOX). Among these are trihalomethanes (THMs), haloacetonitriles, haloacid derivatives, chlorophenols, chlorinated furanones, and chlorinated aldehydes, ketones, and others. Many of these compounds are persistent in nature, and their reported toxicological effects in water supply are carcinogenic, mutagenic, genotoxic, hepatotoxic, neurotoxic, and like.[7]

TABLE 14-5 Total Coliform Remaining in the Effluent

Total Chlorine Residual (mg/L)	Total Coliform (Numbers/100 mL)	
	Primary Effluent	Secondary Effluent
0.5–1.5	24,000–400,000	1,000–12,000
1.5–2.5	6,000–24,000	200–1,000
2.5–3.5	2,000–6,000	60–200
3.5–4.5	1,000–2,000	30–60

Source: Adapted in part from Ref. 5.

14-4-3 Design of Chlorination System

A chlorination system for disinfection of wastewater effluent consists of four separate subsystems: (1) chlorine supply, storage, and safety; (2) chlorine feed and application; (4) mixing and contact; and (4) control systems. Design considerations of each system are discussed below.

Chlorine Supply, Storage, and Safety

Chlorine. Gaseous or liquid chlorine can be supplied in 45.4- to 68-kg (100- to 150-lb) cylinders, 907-kg (1-ton) containers, and in tank cars.[b] Selection of the size of chlorine containers depends on transportation and handling costs, available space, and quantities used. The use of a 907-kg cylinder is generally desirable for moderate-sized users.[c] Some elements of the supply system to handle 907-kg containers include scale, pipe headers for delivering chlorine at the point of use, gauges for checking pressures, and an overhead crane for handling the cylinders.

Chlorine storage and handling systems must be designed with full safety considerations because chlorine gas is very poisonous and corrosive. Many important design and safety considerations are summarized below:[2-4]

- Chlorine is a respiratory irritant detectable at 0.3 ppm (threshold odor). It causes throat irritation at 15 ppm and coughing, shortness of breath, chest pain, and possible nausea and vomiting at 330 ppm.
- The chlorination room should be near the point of application.
- Chlorine storage and chlorinator equipment must be housed in a separate building. If not, it should be accessible only from the outdoors.
- Adequate exhaust ventilation at floor level should be provided because chlorine gas is heavier than air.
- Chlorine storage should be separate from the chlorine feeders and accessories.
- The chlorinator room should have temperature control. A minimum temperature of 21°C is recommended. The chlorine supply area should be kept cooler than the chlo-

[b]Tank cars have capacities of 14,500, 27,200 and 49,900 kg (16, 30, and 55 tons).

[c]Withdrawal rates vary from 180 kg/d for gaseous to over 900 kg/d for liquid withdrawal systems.

rinator; however, the temperature in the chlorine supply area should not be allowed to drop below 10°C.

- The sun should not be permitted to shine directly on the cylinders, and heat should never be applied directly to the cylinders.
- The chlorine storage and feed system should be protected from fire hazards. Water must be available for cooling the cylinders in case of fire.
- A clear viewing window should be provided for viewing the chlorination equipment. Blower control and gas masks should be located at the room entrance.
- Wrought iron piping should be provided for liquid chlorine and chlorine gas. Tough plastic piping should be provided for chlorine solutions. Valves and pipe fittings should be specifically designed for chlorine use. Liquid chlorine has a very high coefficient of volume expansion; therefore, sufficient air cushion or expansion chambers should be provided in liquid chlorine lines.
- When confined in a container, chlorine may exit as a gas, liquid, or both, simultaneously. Gauge pressure is therefore not an indicator of chlorine amount since it is capable of changing while volume remains constant. Therefore, chlorine cylinders in use should be set on a platform scale, and loss of weight should be used for recordkeeping of chlorine dosages. Various types of scale, including beam and pipe lever types with dial indicators, are available. Hydrostatic or electronic load cells are also used for ton container storage facilities.
- If a liquid chlorine withdrawal rate in excess of 680 kg/d is used, a chlorine evaporator is generally needed.
- External pressure reducing valves should be used in long feed lines and where wide variations in temperature is expected.

Many other design guides and safety considerations for chlorine-handling equipment are provided in Ref. 4. The Chlorine Institute provides standards for chlorine-handling equipment and safety procedures.[8] All chlorine systems should conform to these standards.

A designer's checklist for ton container storage facilities should include the following:[4]

1. Cylinder weighing scales or load cells
2. Trunions for ton containers
3. Ton cylinder lifting bar
4. Chlorine gas filter
5. External chlorine pressure-reducing valve
6. Liquid chlorine expansion tank
7. Appropriate gas and liquid supply pressure gauges
8. High- and low-pressure indicating switches for alarms (high-pressure alarm is used only on liquid systems)
9. Condensate traps at inlet to chlorinators
10. Appropriate header valves and shut-off valves

The use of bulk storage tanks for chlorine storage has increased in recent years. This is due, in part, to larger chlorine tank fleets and increasing bulk chlorine deliveries by truck. The capacity of a bulk storage tank will vary from 23,000 kg up to 83,000 kg (25 to 90 tons), depending on whether the delivery is by truck or by railcar. Tank design should be of steel or iron construction and should follow recommendations included in the guidelines of the Chlorine Institute.[8] Pressure rating of the tank should be 120 percent of the maximum working pressure, but not less than 1550 kN/m^2 (225 psi).

The unloading system for bulk storage must be specifically designed for each installation but should include the following essential items:[4,9-11]

1. Unloading platform
2. Storage tank and sun shield
3. Weighing device
4. Air padding system
5. Eductor
6. Chlorine gas and liquid headers
7. Gages
8. Pressure switches and alarms
9. Expansion tanks
10. Flexible connections

Feed piping from the bulk storage tank should be Schedule 80 (minimum) black seamless steel with 906-kg (2000-lb) forged steel fittings. Extra thickness should be designed for the tank, approximately 3 mm (1/8-inch) thicker than the design code, to compensate for corrosion.[4,8,9]

Hypochlorite. The potential hazard associated with transportation, storage, and handling of chlorine has resulted in the use of hypochlorite solution. Hypochlorite is somewhat more expensive, loses strength in storage, and may be difficult to feed. However, for safety reasons alone, many larger plants in urban areas use hypochlorites.

Sodium hypochlorite solution is available in 1.5 to 15 percent strength, in 4.9- to 7.6-m^3 tanks and tank cars. Stronger solutions decompose readily by exposure to light and heat. High-test calcium hypochlorite contains at least 70 percent available chlorine. It is available in 45- to 360-kg drums as powder, granules, or compressed tablets or pellets. Hypochlorites and solutions must be stored at cool and dry places for a better shelf life. Specific information on hypochlorites and other disinfectants is compared in Table 14-1.

Chlorine Feed and Application. The chlorine feed and application system includes (1) chlorine withdrawal, (2) evaporator, (3) automatic switchover, (4) chlorinator, (5) ejector system, (6) mixing and contact, and (7) control system. The design information on these systems is provided below.

Chlorine Withdrawal. The chlorine cylinders provide chlorine to the chlorinator by either gas withdrawal or liquid withdrawal. If gas withdrawal is used, the maximum withdrawal rate for a ton container is 180 kg/d (400 lb/d) at room temperature. If the withdrawal rate is 180–680 kg/d, two or more ton containers must be manifolded together.

If chlorine is being used in this manner, the room temperature must be maintained above 18°C to provide the heat required to replace the heat of evaporation. Withdrawal rates greater than 680 kg/d should use liquid withdrawal and employ evaporators. If liquid withdrawal is used, the bottom outlet valve of the ton container is connected to the feed piping (the top outlet valve is for gas withdrawal). Liquid withdrawal has certain advantages:

1. It is not affected by ambient temperature, allowing container storage in an open structure with only a sun shield.
2. There is no danger for reliquification between container and chlorinator.
3. Fewer containers need to be connected at one time because liquid withdrawal rates are much higher than gas withdrawal rates.

Evaporators. Chlorine evaporators are generally used when the chlorine withdrawal exceeds 680 kg/d. Evaporator capacities range from 180–3600 kg/d (400–8000 lb/d). The evaporator receives the liquid chlorine from the container and vaporizes it to chlorine gas in a sealed pressurized chamber. The chamber is surrounded by a hot water bath, which provides the heat for vaporization. All evaporators should be equipped with a pressure-reducing, automatic shut-off valve to prevent liquid chlorine from entering the chlorinator. A cathodic protection system should be provided to protect against corrosion, and the exterior of the water bath should be insulated. Sizing of the evaporator can be accomplished using the equipment manufacturers' catalog data. A typical chlorine evaporator is shown in Figure 14-2.

Automatic Switchover. Provision for automatic switchover from one cylinder to another should be included to increase system reliability. There are two types of automatic switchover systems: vacuum and pressure. Both types switch the flow of chlorine from one container to the other as soon as one container becomes empty. A vacuum-type automatic cylinder switchover system, chlorinator, and ejector assembly are shown in Figure 14-3.

Chlorinator. The chlorinator receives the chlorine gas from the storage container or evaporator and regulates the flow to the ejector. Different types of chlorinators are (1) direct feed, (2) pressure type, (3) remote vacuum type, and (4) sonic flow type. A conventional chlorinator consists of the following units: an inlet pressure-reducing valve, a rotameter, a metering control orifice, and a vacuum differential regulating valve. The driving force comes from the vacuum created by the chlorine ejector. The feed rate varies from 30 to 5000 kg/d. The selection of any type of chlorinator should be based on flow rate and type of application. Chlorinators are usually designed by the equipment manufacturers to limit the concentration of chlorine solution at the injector to 3500 mg/L. The control schemes for chlorination range from simple to complex and manual to varying degrees of automation. Equipment manufacturers should be consulted for design and selection of chlorinators. A typical chlorinator details are shown in Figure 14-4.

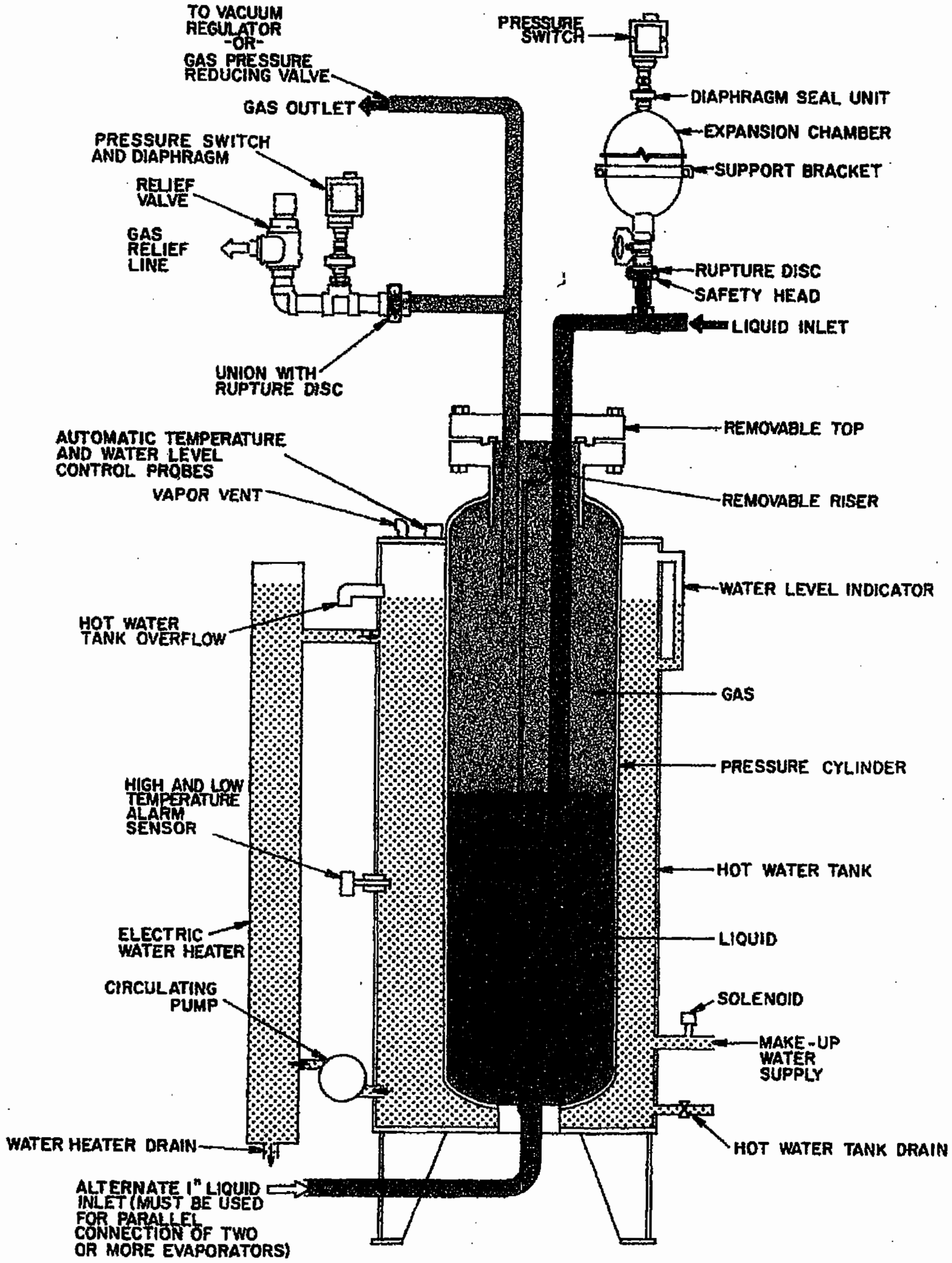


Figure 14-2 Typical Chlorine Evaporator with Relief System (courtesy Wallace & Tiernan, U.S. Filter).

Ejector System. The chlorine feed, ejector, or injector system is very essential because it provides the required dosage at the point of application. Chlorine feed systems are of two types: (1) pressure injection of gas and (2) vacuum feed. The pressure injection of gas may pose risks of gas escaping. It is normally used in small plants or in large facilities where safety precautions are rigidly followed. In vacuum feed systems, a specified vacuum is applied to an ejector to evaporate and move chlorine gas from the supply source to the chlorinator where it is mixed with water and carried to the point of application. The amount of water must be enough to (1) maintain chlorine concentration in the

(a)

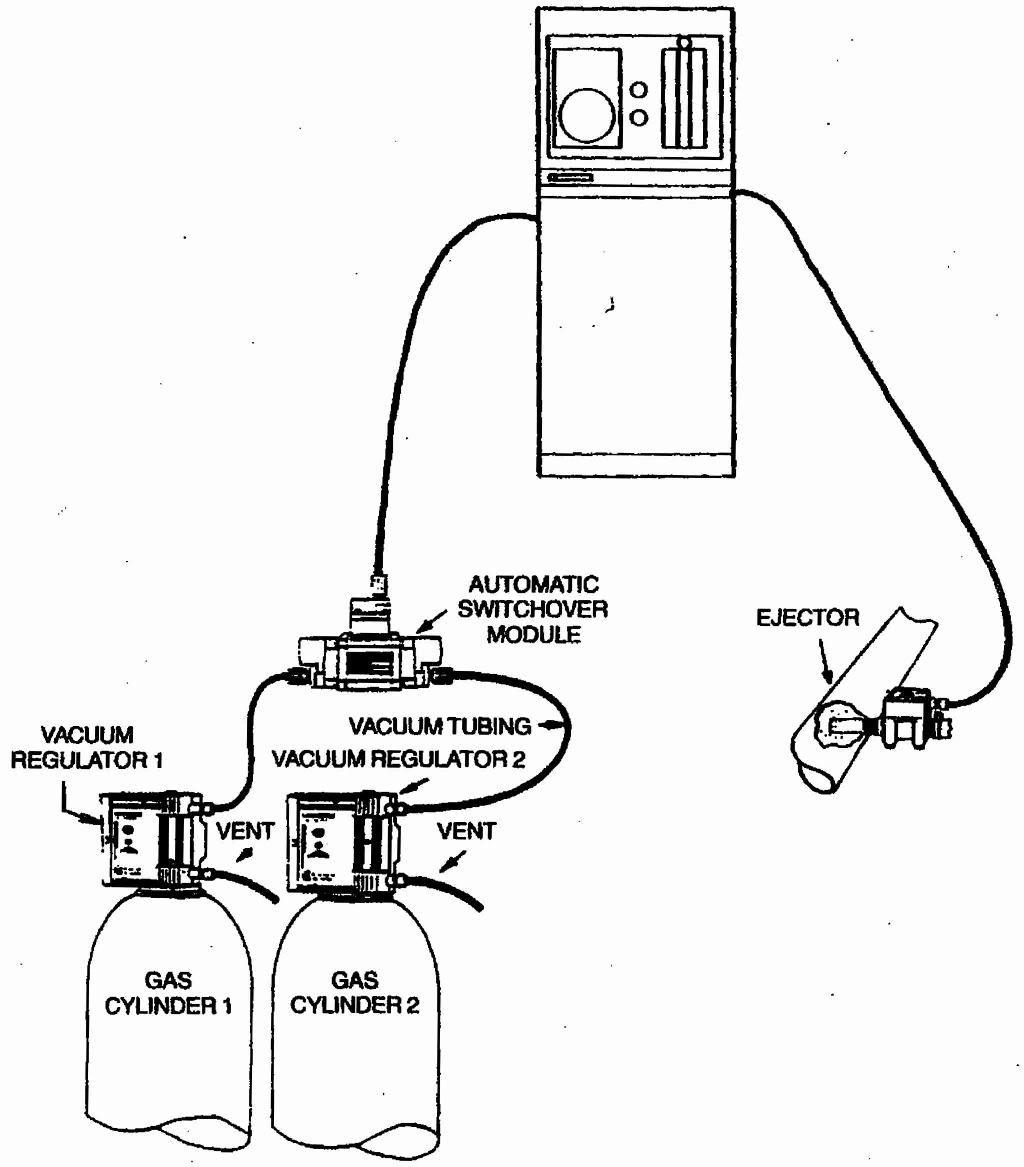
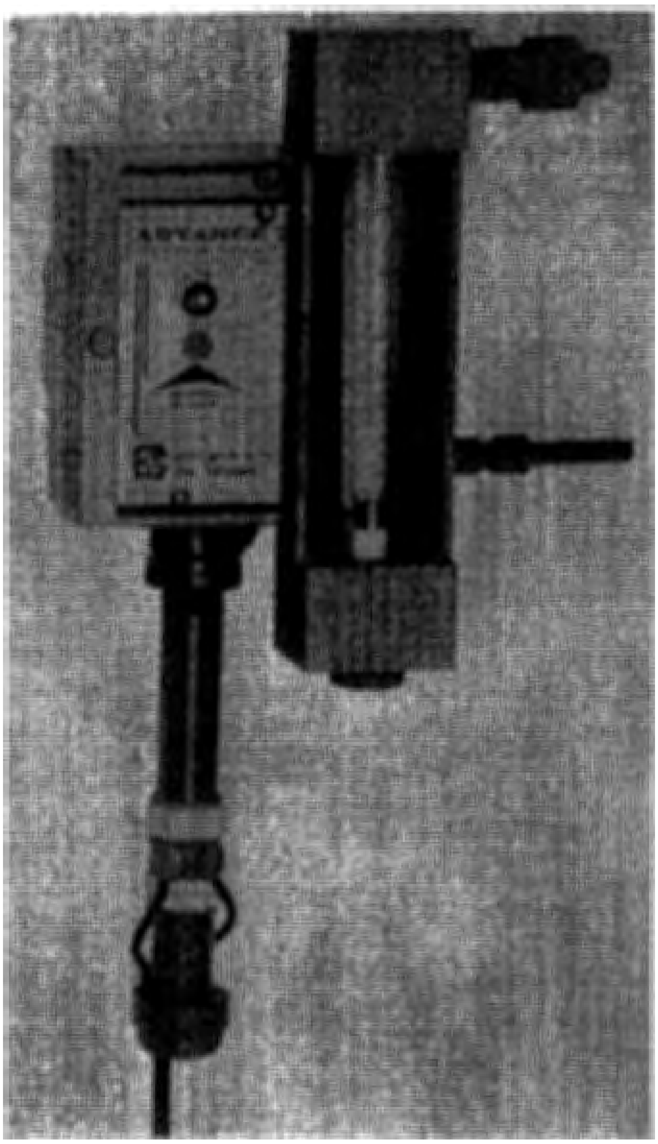


Figure 14-3 Automatic Compound Loop Chlorination System and Accessories (courtesy Capital Controls Company): (a) advance automatic switchover system using chlorine gas cylinder.

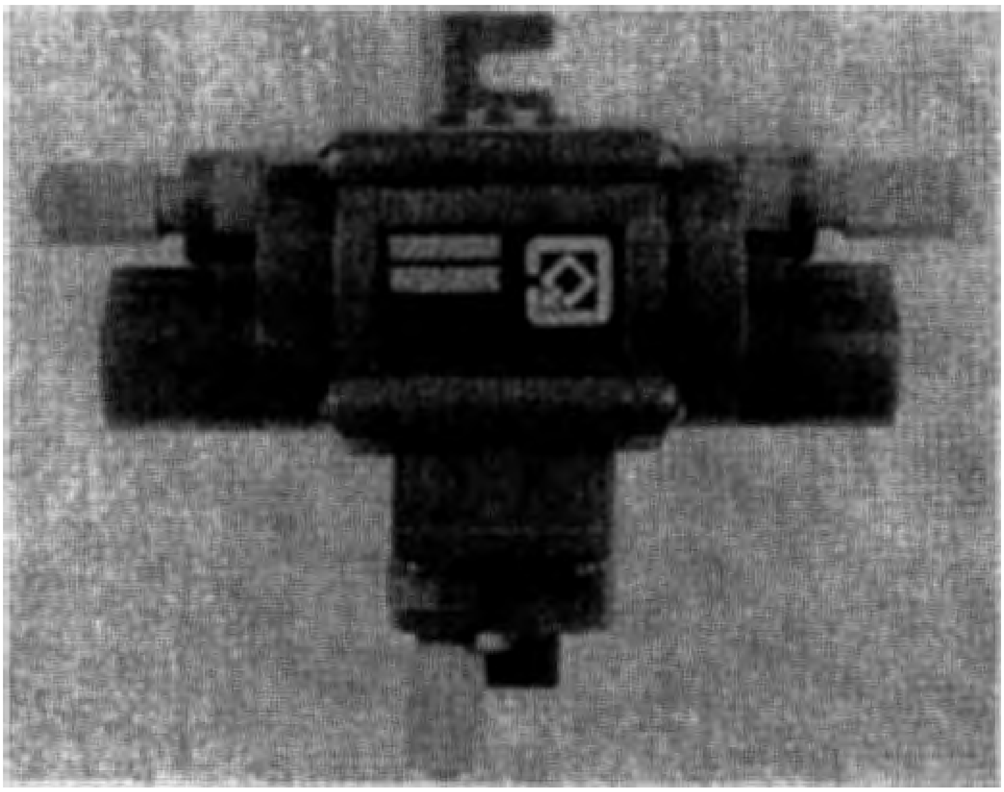
solution below saturation, which is 3500 mg/L and (2) create the required amount of vacuum in the line to the chlorinator and in all of the components of the chlorinator system. Manufacturers of chlorinator systems provide ejector operating curves that specify the amount of water and pressure required for a given amount of chlorine to be applied against a given back pressure (Figure 14-5). From the ejector the chlorine solution (in the form of hypochlorous acid) flows to the point where it is applied into the water.

The ejector system usually includes the following items:[4]

1. Solution water supply pump and piping to the injector
2. Ejector water pressure gauge
3. Ejector
4. Vacuum piping from the chlorinator
5. Ejector vacuum gauge (for remote injector installation)



(b)



(c)

Figure 14-3—cont'd (b) manifold for chlorination system, and (c) automatic switchover chlorination system.

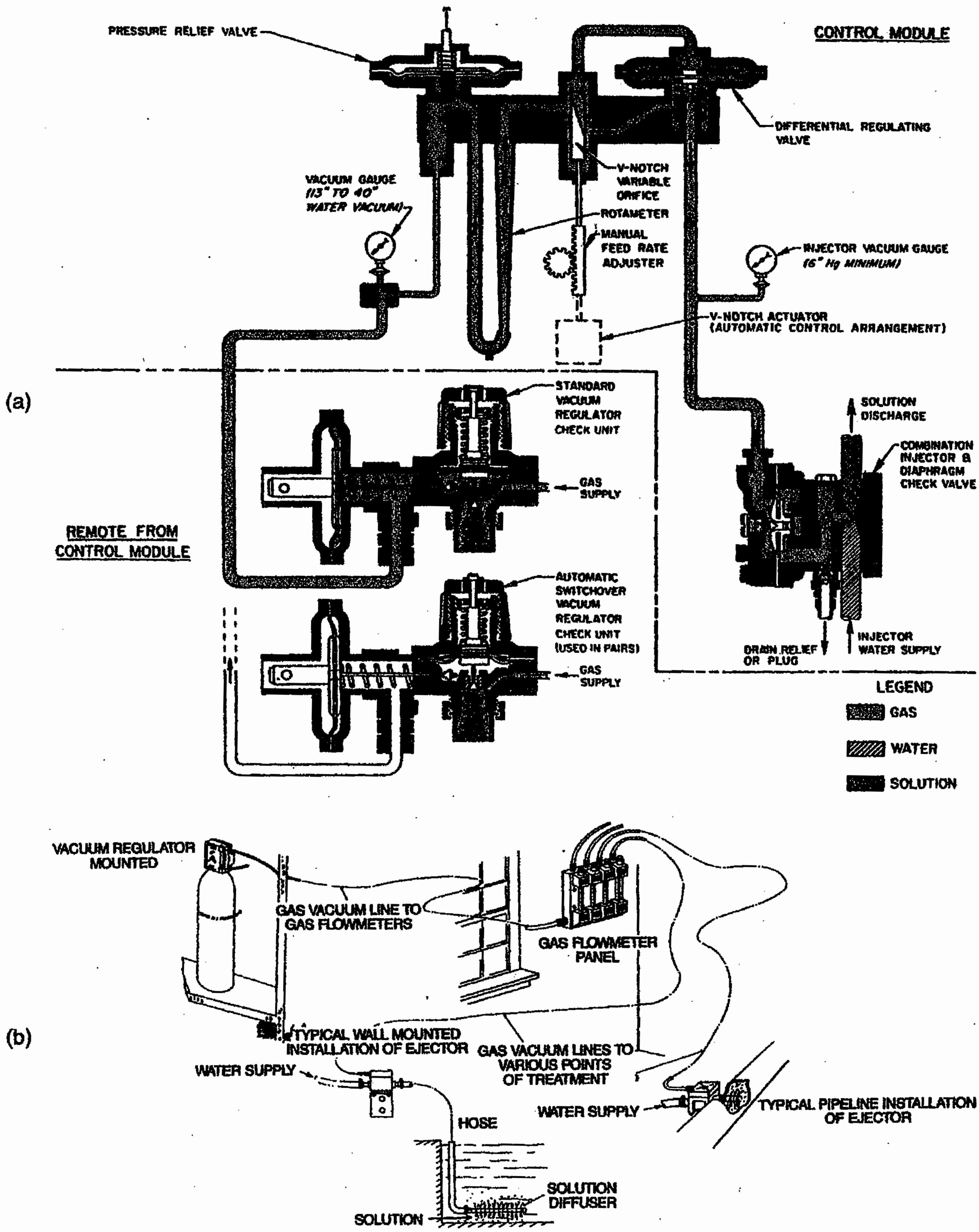


Figure 14-4 Chlorinator and Chlorine Metering and Injection System: (a) typical chlorinator arrangement (courtesy Wallace & Tiernan, U.S. Filter) and (b) typical chlorine metering and injection system (courtesy Capital Controls Company).

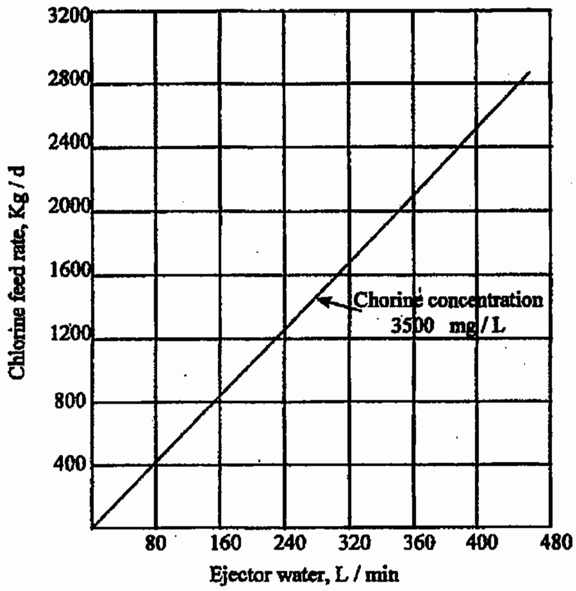


Figure 14-5 Ejector Water Flow Rate versus Chlorine Feed Rate Limiting Chlorine Concentration to 3500 mg/L.

6. Ejector vacuum line shutdown (for remote injector location)
7. The chlorine solution piping
8. Chlorine solution pressure gauge located immediately downstream from the injector (for variable throat injector only)
9. Solution water pressure switch and low water pressure alarm
10. Solution water flow meter
11. Built-in vacuum switch and alarm for both high and low vacuum
12. Compound back pressure gauge for injector discharge

A strong chlorine solution up to 3500 mg/L is generally injected in water. Negative pressure conditions should be avoided in chlorine solution piping downstream of the injectors. This could result in the undesired release of chlorine gas at the application point. An exit velocity of 7–9 m/s at the injection point will provide adequate mixing. A head loss of 2–4 m will help to maintain back pressure at the injector discharge and develop a jetting effect. Check valve and isolation valve for pressurized pipe diffusers are essential.[3,4]

Mixing and Contact. Rapid mixing of chlorine solution into wastewater, followed by a quiescent contact period, is essential for effective disinfection. The chlorine solution is provided through a diffuser system. It is then mixed rapidly by (1) mechanical means, (2) baffle arrangement, and (3) hydraulic jump created downstream of a weir, Venturi flume, or Parshall flume. Various types of chlorine diffusers and mixing arrangements are illustrated in Figures 14-6 and 14-7. A velocity gradient of above 400 s^{-1} is sufficient to provide the desired mixing. The velocity gradient to satisfy the mixing requirements may be calculated from Eq. (14-12):

$$G = (\gamma h_L / t\mu)^{1/2} \tag{14-12}$$

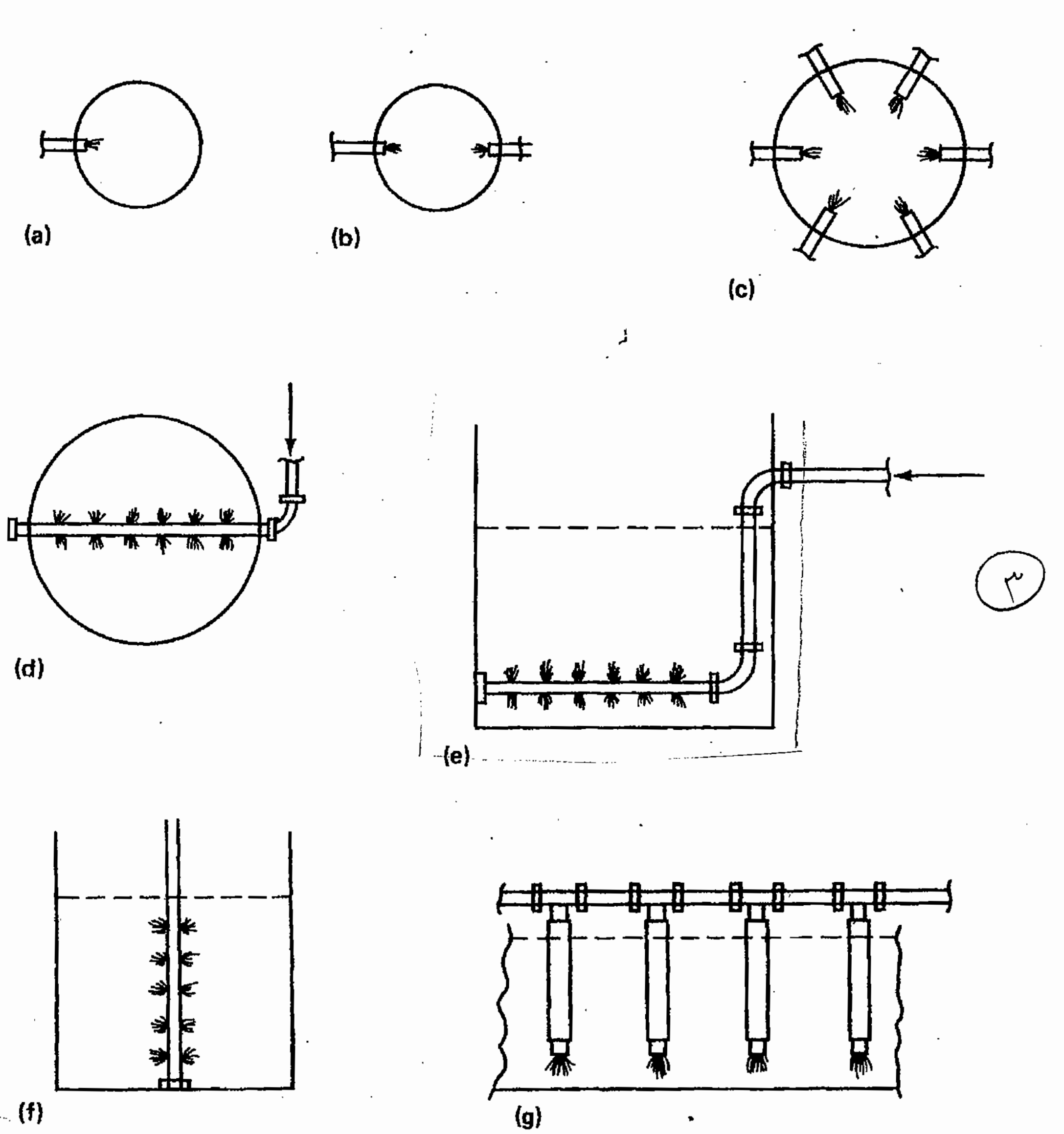


Figure 14-6 Typical Chlorine Diffusers: (a) single injector for small pipes, (b) dual injector for small pipes, (c) multiple injectors for medium-sized pipes, (d) injector system for large pipes, (e) single horizontal diffuser in open channel, (f) single vertical diffuser for open channel (for wide channels, multiple diffusers along the width may be used), and (g) typical hanging-nozzle-type multiple diffusers along the length of an open channel.

where

G = velocity gradient, s^{-1}
γ = specific weight of water, N/m^3 ($9.78\ kN/m^3$ at 25°C)
μ = dynamic viscosity of water, $N{\cdot}s/m^2$ ($0.89 \times 10^{-3}\ N{\cdot}S/m^3$ at 25°C)
h_L = total head loss through diffuser device, m
t = detention time, s

The purpose of the chlorine contact chamber is to provide the contact time necessary for the disinfecting compound to reduce the number of organisms to acceptable levels. Regulatory agencies normally specify the contact time. It may range from 15 to 30

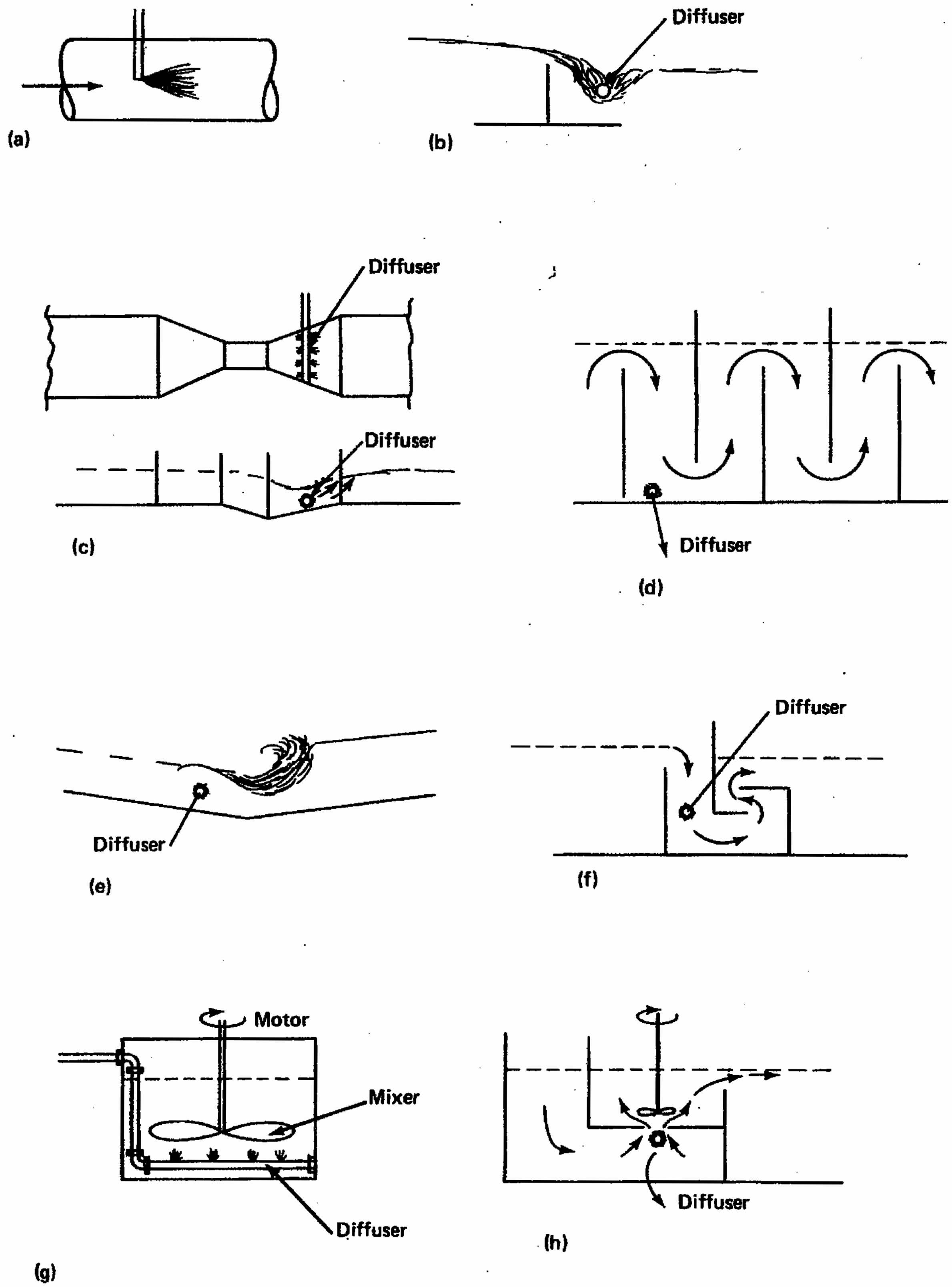


Figure 14-7 Different Types of Mixing Arrangements Used for Dispersing Chlorine Solution in Wastewater: (a) jet nozzle in a closed conduit and mixing by natural turbulence and diffusion; (b) downstream of a weir, natural mixing because of turbulence; (c) natural mixing in a Parshall flume; (d) mixing in a tank using over-and-under baffles; (e) mixing at the hydraulic jump in a channel; (f) mixing by baffle arrangement; (g) mechanical mixer in open channel; (h) mechanical mixer with baffles in open channel.

Continued

(i)

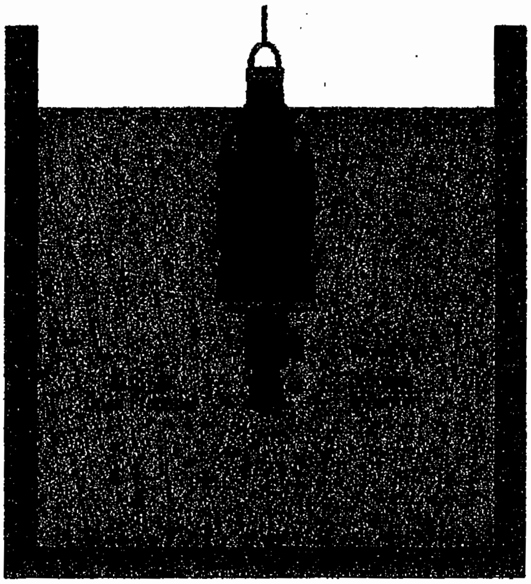


Figure 14-7—cont'd (i) WATER CHAMP® vacuum enhancer and enhanced mixing (courtesy Gardiner Equipment Co., Inc.).

min; periods of 15 min at peak flow are common.[2,3] Irregularly shaped, circular, and rectangular basins have been used. The design objectives are to (1) minimize short-circuiting and dead spaces, (2) maximize mixing for better disinfection, and (3) reduce settling of solids in the basin. Chlorination improves the settling characteristics of the solids, and accumulation of solids in the chlorine contact basin has often caused serious problems. Solids deposited in the contact chamber exert a greater chlorine demand. In addition, the deposited solids permit the growth of anaerobic organisms. The gases produced from the anaerobic areas rise and carry the solids, causing occasional high solids in the effluent.[7] Design changes were necessary in many cases to reduce this problem. Air agitation, maintaining high horizontal velocity through the basin, mechanical solids collection equipment (similar to sedimentation basin), and multiple units to take one chamber out of service for solids removal have all been attempted with some success. Many states require mechanical solids removal equipment in conjunction with chlorine contact chambers. Several solids flushing arrangements in chlorine contact units have also been proposed.

The most common design of a contact chamber utilizes longitudinal baffles with two to four passes around the ends to simulate a long, narrow channel (length-to-width ratio 10:1 or more). This design gives plug flow regime. However, lack of mixing and dead spaces around the corners result in solids accumulation. Rounded corners and additional baffles have been suggested to improve the design.[2,3] Cross-baffles having flow over and under the baffles are also used. This arrangement gives good mixing; however, solids accumulation between the baffles is a serious problem.

Control Systems. The chlorination system must maintain a given chlorine residual at the end of the specified contact time. Since wastewater flow is variable and the effluent quality to be chlorinated may also change, chlorine dosage must be adjusted frequently

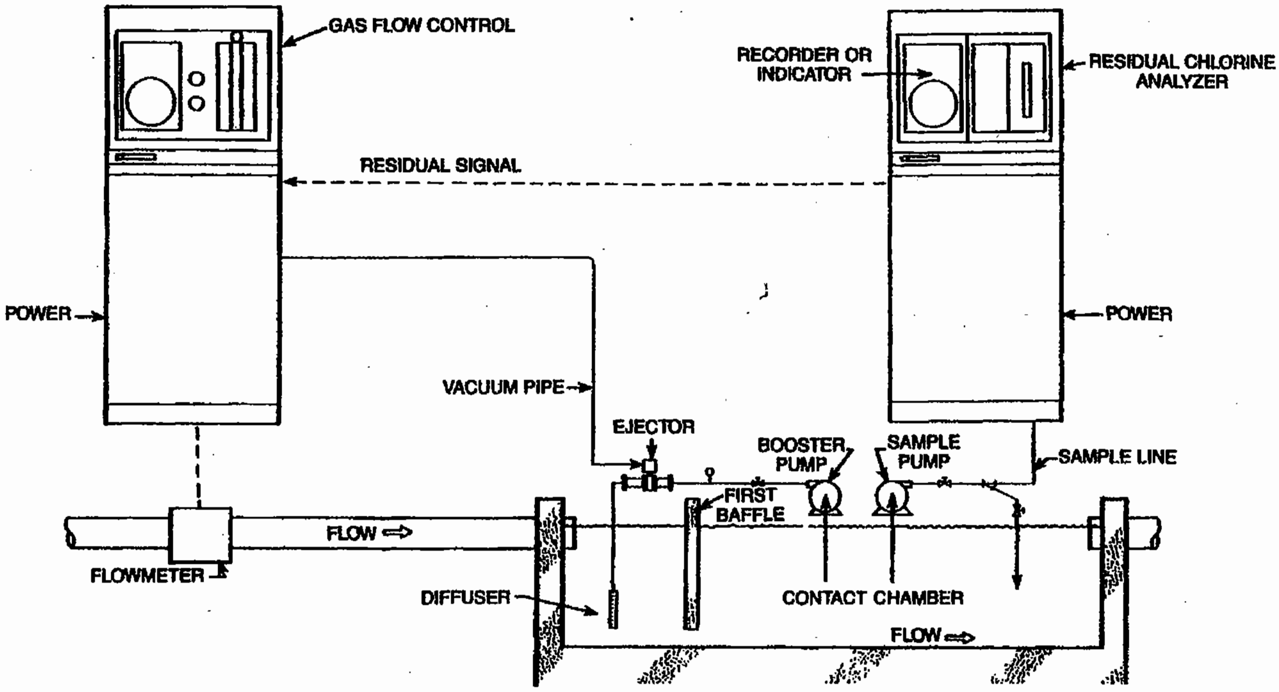


Figure 14-8 Typical Automatic Compound Loop Chlorination Assembly (courtesy Capital Controls Company).

to provide the given residual. At small installations manual control is used. The operator determines the chlorine residual and then adjusts the feed rate of chlorine solution. Simple orifice-controlled, constant-head arrangement or low-capacity proportioning pumps are used to feed the chlorine solutions. Often, constant-speed feed pumps are programmed by time clock arrangement to start and stop the pump at desired intervals.

At large facilities, complex automatic proportional control systems with recorders are used. Signals from a flow meter transmitter (Parshall flume) and/or chlorine residual analyzer (measuring the residual at one-quarter to one-half contact time) are transmitted to the chlorine feeder to adjust the feed rate to maintain a constant preset chlorine residual at all flow rates, as well as effluent quality.

Several alarms are also considered an essential part of the control system. These include high and low pressures in chlorine containers, chlorine leaks, high and low ejector vacuum, high and low temperatures for evaporator water bath, and high and low chlorine residuals.

Many types of automatic chlorine residual control loop chlorinator systems are available in the market. An automatic compound loop control chlorination system is desirable for situations where both the chlorine demand and wastewater flow rate vary. The system contains a feedback element, which senses flow readings and chlorine residuals and feeds this information back into the system for comparison with the desired level of chlorine residual. The control system consists of (1) chlorine residual analyzer, (2) chlorinator with automatic chlorine gas valve, (3) control system to receive both flow and chlorine residual signals, and (4) automatic chlorine feed system to maintain a preset chlorine residual. The system is shown in Figure 14-8.

14-5 DECHLORINATION

Chlorine reacts with many organic compounds in the effluent and produces undesired toxic compounds. Many of these compounds have long-term adverse impacts on the water environment (Sec. 14-3-2). In addition, chlorine residual may have potential toxic effects on aquatic organisms. Even a low level of chlorine residual may interfere with the biomonitoring test required to determine the toxicity in the wastewater effluent and to control the discharge of toxic pollutants in toxic amounts (Secs. 2-6-3 and 3-4-3). Therefore, dechlorination is removal of free and total combined chlorine residual from the effluent. It may be noted that dechlorination will not remove the toxic by-products that have already been produced. Dechlorination is accomplished by the reaction with a reducing agent such as sulfur dioxide, SO_2, sodium metabisulfite ($Na_2S_2O_3$), or activated carbon. Sulfur dioxide is the most commonly used chemical in plants over 3800 m^3/d (1 mgd). Sulfur dioxide and carbon adsorption systems are briefly discussed below.[2-4]

14-5-1 Sulfur Dioxide

Sulfur dioxide is an effective reducing agent for dechlorination. It successively removes free chlorine, monochlorine, dichloramine, nitrogen trichloride, and poly-n-chlor compounds.[2,12] The reactions with free and combined chlorines are given by Eqs. (14-13) and (14-14):

$$SO_2 + HOCl + H_2O \rightarrow HCl + H_2SO_4 \tag{14-13}$$

$$SO_2 + NH_2Cl + 2\,H_2O \rightarrow NH_4Cl + H_2SO_4 \tag{14-14}$$

The stoichiometric weight ratio of SO_2 and Cl_2 (free or combined residual expressed as Cl_2) in the above reactions is 0.9:1. In practice, a 1.2:1 ratio is used because organic nitrogen interferes with dechlorination reaction. An excess sulfur dioxide dose may cause deoxygenation of effluent. The reaction is slow and expressed by Eq. (14-15):

$$2\,SO_2 + O_2 + 2\,H_2O \rightarrow 2H_2SO_4 \tag{14-15}$$

Sulfur dioxide is available commercially as a liquefied gas under pressure in steel containers of 45, 68, and 907 kg. The sulfur dioxide supply, storage, feed systems, and costs are very similar to that of chlorine. The major components are cylinder delivery, unloading, storage, scales, sulfur dioxide feeders (sulfonators), evaporators, gas headers, injectors, diffusers, interconnecting pipings and valves, and alarms. The major difference is the instantaneous reaction of sulfur dioxide with chlorine and chloramines. As a result, contact tanks are not needed. Rapid mixing for 30–60 seconds at the application point is sufficient for a dechlorination reaction.

14-5-2 Activated Carbon

Granular activated carbon bed is very effective in removing, not only chlorine and chloramine, but also the other chlorination by-products. The granular activated bed is used in either a gravity or pressure filter bed. The desirable pretreatment step is regular filtration prior to carbon adsorption. The typical hydraulic loading on carbon bed is 150 $m^3/m^2 \cdot d$. The dechlorination reactions with activated carbon and free and combined chlorine residuals are expressed by Eqs. (14-16) and (14-17):[12]

$$C + 2Cl_2 + 2H_2O \rightarrow 4HCl + CO_2 \qquad (14\text{-}16)$$
$$C + 2\,NH_2Cl + 2\,H_2O \rightarrow CO_2 + 2NH_4Cl \qquad (14\text{-}17)$$

The activated carbon process for dechlorination is reliable but quite expensive. Its application becomes more cost-effective if removal of other refractory organics is desired for advanced wastewater treatment. This topic is covered in Chapter 24.

14-6 CHLORINE DIOXIDE

Chlorine dioxide has been used for many years to bleach flour, paper, and textile. Its use in wastewater effluent disinfection has been considered in recent years because it is a more powerful bactericide and viricide than chlorine, and its impacts (because of by-products) are much less adverse than those associated with chlorination.[12,13]

14-6-1 Generation of Chlorine Dioxide

Chlorine dioxide, being an unstable and explosive gas, is always generated on-site in a gaseous solution and used shortly after its preparation. Solutions up to 5000 mg/L can be stored for several days. It is generated from a solution of sodium chlorite ($NaClO_2$), which may be purchased as a 25 percent aqueous solution or a solid of 80 percent purity. The generation step involves reacting the aqueous solution of sodium chlorite with chlorine gas or hypochlorous acid. These reactions are expressed by Eqs. (14-18) and (14-19):

$$2NaClO_2 + Cl_2\,(g) \rightarrow 2ClO_2\,(g) + 2NaCl \qquad (14\text{-}18)$$
$$2NaClO_2 + HOCl \rightarrow 2ClO_2\,(g) + NaCl + NaOH \qquad (14\text{-}19)$$

The reaction is carried out in a chlorine dioxide reactor by measuring appropriate quantities of reactants. If excess chlorine solution is added, free chlorine can oxidize chlorine dioxide to form chlorate ions (ClO_3), which are difficult to remove from solution. In these reactions, 1.34 mg $NaClO_2$ reacts with 0.5 mg Cl_2. The technical grade sodium chlorite is approximately 80 percent pure; therefore, proper adjustments must be made for chemical purity.

14-6-2 Properties of Chlorine Dioxide

Chlorine dioxide is a greenish yellow gas with a strong disagreeable odor that is similar to, but stronger than, chlorine. It is toxic to humans when inhaled. Its odor is detectable above concentrations of 0.1 ppm. It makes an explosive mixture in air in concentrations above 10 percent by volume (corresponding to a solution concentration of about 12 g/L).

Chlorine dioxide is a more powerful disinfectant than chlorine, although it has a lower oxidation potential (Table 14-2). When produced in the absence of excess free chlorine, it (1) does not react with ammonia, therefore lasting longer than free chlorine residual, and (2) does not produce THMs and other chlorinated organic by-products of concern. If excess chlorine is present, many undesired effects are associated. Among these are (a) production of chlorinated organic by-products, (b) production of hypobromous acid and brominated compounds, and (c) production of chlorite and chlorate ions by disproportion with pH below 2 and above 11. Both chlorite and chlorate ions have undesired environmental effects. Free chlorine dioxide residuals have a short life and are less harmful to aquatic life than chlorine. Residuals may be destroyed by a sulfur dioxide reaction [Eq. (14-20)]:

$$5SO_2 + 6H_2O + 2ClO_2 \rightarrow 5H_2SO_4 + 2HCl \tag{14-20}$$

The sulfur dioxide requirement is 2.5 mg per mg of ClO_2. In practice, 8–10 percent excess sulfur dioxide is used. Carbon adsorption in a GAC column is also a very effective agent in reducing residual chlorine dioxide and chlorite ions into innocuous chloride ions.

14-6-3 Design Considerations

The system design for chlorine dioxide includes (1) chlorine dioxide generator; (2) supply, storage, and feed system for sodium chlorite; (3) transport of metered chlorine dioxide feed solution to the application point; and (4) dispersion of the solution in water. Chlorine dioxide generators are commercially available. The design of chlorine dioxide systems is similar to those for chlorine. If a plant has a chlorination facility, it can be fully adapted for chlorine dioxide generation.

14-7 BROMINE CHLORIDE

Bromine chloride for disinfection of effluent has received some interest because in many ways it is superior to chlorine. In this section, the generation, properties, effectiveness, and design information on bromine chloride are presented.

14-7-1 Generation of Bromine Chloride

Bromine chloride is generated by adding an equivalent amount of chlorine (as a gas or liquid) to bromine until the mixture has increased in weight by 44.3 percent. The reaction is given by Eq. (14-21):

$$Br_2 + Cl_2 \leftrightarrow 2BrCl \tag{14-21}$$

BrCl is also prepared by the reaction of bromine in an aqueous hydrochloric acid solution, chlorine in an aqueous hydrobromic acid solution, or by oxidizing a bromide salt in a solution containing hydrochloric acid. These reactions are expressed by Eqs. (14-22)–(14-24):

$$Br_2 + HCl \rightarrow BrCl + HBr \qquad (14\text{-}22)$$
$$HBr + Cl_2 \rightarrow BrCl + HCl \qquad (14\text{-}23)$$
$$KBrO_3 + 2KBr + 6HCl \rightarrow 3BrCl + 3KCl + 3H_2O \qquad (14\text{-}24)$$

In Reactions (14-21) and (14-23) the existing chlorination system at a treatment facility may be used to produce bromine chloride.

14-7-2 Properties of Bromine Chloride

The solubility of BrCl in water is 85,000 mg/L at 20°C. This is 2.5 times the solubility of bromine and 11 times that of chlorine. Bromine reportedly offers a broadly effective bactericidal effect superior to that of chlorine when compared on an equivalent dose basis. This improvement has been established for bacteria, amoebic cysts, and viruses. The following benefits have been reported in the literature:[14-18]

1. The hydrobromous acid (HOBr) is a weaker acid than hydrochlorous acid (HOCl). The dissociation constant for HOBr is $10^{-8.7}$ as compared to that for HOCl, which is $10^{-7.5}$. As a result, the effectiveness of bromine-based disinfection is retained up to a pH of 8.7, while chlorine disinfection has significantly reduced disinfecting power above pH 7.5.
2. HOBr complexion with ammonium-nitrogen incurs only a nominal reduction in bactericidal effectiveness, compared with the significant loss commonly associated with chloramine formation. This bromamine benefit would effectively obviate the necessity for either prior ammonia removal or breakpoint chlorination.
3. Bromamine species appear to decay faster than chloramine species. As a result, bromine species are short-lived and more expedient oxidizers. In this regard, the higher reactivity of the bromine and bromamine forms suggest that improved levels of disinfection could be completed with the lowest permissible halogen dosage required to achieve the desired microbial kill. With respect to halogen compounds, some studies have shown that, although the bromine forms are stronger oxidizers than chlorine forms, the chlorine species were stronger halogenators.[7] Destruction of bromine similar to dechlorination therefore may not be necessary.

14-7-3 Design Considerations

Bromine chloride is not a fully proven disinfectant as compared with chlorine. Additional work is need to varify the bromine chloride dosage, effectiveness, application methods, and field data on aquatic toxicity. Bromine chloride is a hazardous and corrosive chemical, and thus special transportation, storage, and handling precautions are required. Bromine chloride is normally shipped in liquid form in cylinders, tank cars, or 1360-kg containers. The system design is similar to that of chlorination and requires storage and handling, injectors, automatic control, solution lines and diffusers, and safety equipment. In-depth coverage of a bromine chloride system design may be found in Refs. 4, 12, and 15.

14-8 OZONATION

Disinfection of municipal wastewater effluent by ozone has received interest in recent years because of a growing concern over the formation of chlorinated organics, effluent toxicity, and the added cost of dechlorination. Ozone is an unstable gas; therefore, it must be generated on-site. Because of its high oxidation potential, it is a powerful bactericide and viricide and requires a short contact time. In this section the basic chemistry, properties, generation, and design consideration for the ozonation facility are presented.

14-8-1 Process Description and Chemistry

Ozone is a powerful oxidizing agent. Because it is a strong oxidizing agent, it is also a powerful disinfectant. Unlike chlorine, ozone produces little or no THMs [the primary disinfection by-products (DBPs) of concern]. Instead, when exposed to a neutral or alkaline environment (pH above 6), UV light, or hydrogen peroxide, it decomposes in water to produce more active hydroxyl free radicals [Eqs. (14-25) and (14-26)]. This reaction is accelerated at a pH above 8.

$$O_3 + H_2O \rightarrow 2HO_2 \tag{14-25}$$

$$O_3 + HO_2 \rightarrow HO + 2O_2 \tag{14-26}$$

The free radicals formed are HO_2 and HO (hydroxyl free radical) or a mixture of ozone and hydroxyl radicals. These are powerful oxidizing agents. The free hydroxyl radical species are more effective oxidizing agents than the molecular ozone, but they are extremely short-lived.

14-8-2 Ozone Generation

Ozone generation is an established process, but its use in wastewater disinfection is relatively new. Being a chemically unstable gas, it must be generated on-site and used quickly. It is generated by applying energy to oxygen or dried air. A high-energy electrical field causes oxygen to disassociate. The overall reaction is expressed by Eqs. (14-27) to (14-29):

$$O_2 + 2e^- \rightarrow 2O^- \tag{14-27}$$

$$2O^- + 2O_2 \rightarrow 2O_3 \tag{14-28}$$

$$3O_2 \rightarrow 2O_3 \tag{14-29}$$

This reaction is reversible, and once formed, ozone decomposes to oxygen. This reversible reaction occurs quite readily above 35°C. Therefore, an ozone generation system utilizes cooling components to dissipate heat produced during generation.

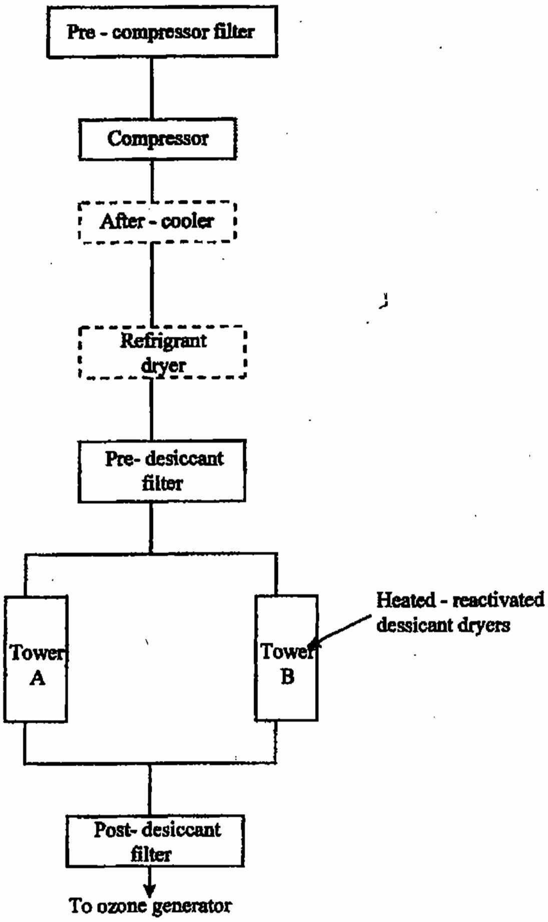


Figure 14-9 Low-Pressure Air Feed–Gas Treatment Schematic for Ozone Generation (from Ref. 7).

The ozone generation system has three elements: (1) feed gas preparation, (2) electrical power, and (3) ozone generation. All these systems are discussed below.

Feed Gas Preparation. Ozone may be generated from air or oxygen. Feed air to ozone generator must be dried to a maximum dew point of −65°C. Moisture in air reduces ozone production, causes fouling of the dielectric tubes, and increases corrosion in the ozone generator and downstream equipment.

Ambient air feed systems for ozone generation are low-, medium-, or high-operating pressure systems. Low-pressure systems operate at a partial vacuum created by a submerged turbine or other ejector device. Medium-pressure systems range from 0.7 to 1.05 kg/m^2 (10 to 15 psig). High-pressure systems operate at pressures 4.9–7.0 kg/cm^2 (70–100 psig) and reduce the pressure prior to ozone generator. The pressure desiccant dryers are also used in conjunction with compression and refrigerant dryers for generating large and moderate quantities of ozone. A typical low-pressure air feed gas preparation schematic for ozone generation is shown in Figure 14-9. Very small systems may use two desiccant dryers in series (no compression or refrigerant drying). The desiccant dryers use silica gel, activated alumina, or molecular sieves to dry air to the necessary dew point.

Feed gas can also be pure oxygen. Basic features of air feed and pure oxygen feed systems are given below. Pure oxygen generating systems have many benefits over air feed: (1) higher production density (more ozone produced per unit area of the dielectric), (2) a high concentration of ozone in the product gas (almost double), (3) less energy requirement, (4) a smaller feed gas volume for the same ozone output, and (5) less ancillary equipment. For small to medium-sized systems, oxygen may be purchased as a gas or as a liquid. For large operations, oxygen generation on-site may be necessary. There are two methods of producing oxygen on-site for ozone generation: (1) pressure swing adsorption of oxygen from air and (2) cryogenic production (liquification of air followed by fractional distillative separation of oxygen from nitrogen). Systems for production of oxygen on-site contain many of the same elements as the air preparation system since the feed gas must be clean and dry irrespective of oxygen content.[13,19]

One benefit of the oxygen generation system is peak power load shedding. The oxygen-enriched feed gas may be generated during off-peak power demand. During the period of peak power demand, the system may be switched to oxygen-enriched feed, thus reducing the power requirement for ozone generation during peak power demands.

Electrical Power Supply. The voltage or frequency of the power to the ozone generation must be varied to control the amount and rate of the ozone produced. Ozone generators use high voltages (up to 710,000 V) or high-frequency electrical current (up to 2000 Hz). Therefore, specialized power supply equipment and design considerations such as proper insulation or wiring and cooling of transformers are necessary.

Ozone Generators. Ozone can be generated by two methods: (1) UV light and (2) cold plasma or corona discharge. Ozone generation by UV light is much the same as ozone is formed in the upper atmosphere. UV light [less than 200 nanometers (nm)] is produced by arc discharge lamp and passes through dry or oxygen-enriched air. Ozone is generated by photochemical reaction; ozone generated by this method is much lower in concentration (0.25 percent) than that produced by corona discharge. This method is suited only for small-scale systems, requires low capital investment, and is relatively easy to maintain.

The most common method of ozone generation for water and wastewater treatment is corona discharge cell. The discharge cell consists of two electrodes separated by a discharge gap. High voltage potential is maintained across a dielectric material, and feed gas flows between the electrodes (Figure 14-10). Ozone concentration of 1–3.5 percent by weight is generated from cool and dry feed air, and 2–7 percent from pure oxygen.

The most common commercially available ozone generators are horizontal or vertical tubes or plates with water, air, or oil-cooled system. Operating conditions of these generators are

- low frequency (60 Hz), high voltage (>20,000V)
- medium frequency (>1,000 Hz), low voltage (<20,000V)
- high frequency (>1,000 Hz), low voltage (<10,000V)

Currently, low-frequency, high-voltage units are most common, but recent improvements in electronic circuitry make high-frequency, low-voltage units more desirable.[12,13,19]

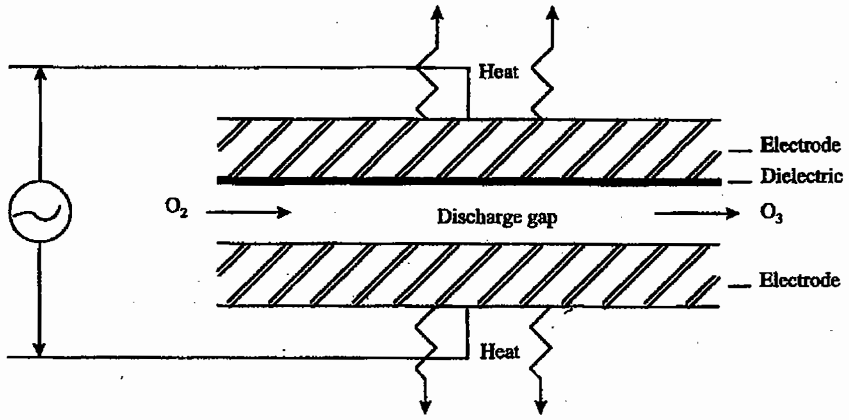


Figure 14-10 Typical Ozone-Generating Configuration for a Corona Discharge Cell (from Ref. 7).

Ozone generation at 60–70 percent of maximum generation capacity is most cost-effective. Multiple units, if selected properly, should satisfy average and peak demands, as well as the necessary standby unit for maintenance. The following example shows the selection process:

- Average ozone requirement is 40 kg/d (88 lb/d).
- Peak period ozone requirement is 60 kg/d (132 lb/d).
- Provide three ozone generators, each designed for 30 kg/d (66 lb/d).
- Two generators operating at 67 percent of maximum capacity will provide average ozone requirement (30 kg/d $\times$ 2 $\times$ 0.67 = 40.2 kg/d).
- Three generators operating at 67 percent of maximum capacity will provide maximum ozone requirement (30 kg/d $\times$ 3 $\times$ 0.67 = 60.3 kg/d).
- One standby unit will be available for maintenance during average demand.

14-8-3 Properties of Ozone and Safety

Ozone is a faintly blue, pungent smelling, and unstable gas. It is detectable even at low concentrations (0.01–0.02 ppm by volume). Higher concentrations may cause olfactory and other reactions, and much higher concentrations may be toxic in some instances. The longer the exposure to ozone, the less noticeable is the odor.

The solubility of ozone in water depends upon temperature and its concentration in feed gas as it enters the ozone contactor. The higher the concentration of ozone in the feed gas, the more soluble it is in water. Increasing pressure in the ozone contactor also increases its solubility. A solubility of 570 mg/L at 20°C has been reported.[4] The half-life of ozone in water ranges from 8 minutes to 14 hours, depending on the level of ozone-demanding contaminants in water and wastewater and temperature.

Ozone reacts with inorganic compounds primarily with nitrite, ferrous, manganous, and ammonium ions. Ozone also reacts with organic aliphatic and aromatic compounds, humic acids, and pesticides producing lower molecular organic species. Among these are aldehydes and ketones. Ozone does not produce halogenated organic matter directly. However, in the presence of bromide ions, hydrobromic acid is produced, which may encourage formation of brominated organics. It is believed that bacteria and protozoa cysts are killed because of cell wall disintegration (cell lysis). It is also a very effective viricide. Ozone does not produce dissolved solids and is not affected by ammonium ions or pH. It also does not build up residuals. For these reasons, ozonation is considered a viable alternative to chlorination, especially since, most often, dechlorination is required. Some of its reaction by-products may have toxic mutagenic or carcinogenic properties, but they are short-lived. Added benefits of ozonation may also include destruction of some harmful refractory organics and pesticides, and the dissolved oxygen level is elevated, which may eliminate the need for post-reaeration.

14-8-4 Design Considerations

The design of an ozone system has five major components: (1) feed gas preparation, (2) electrical power, (3) ozone generator, (4) ozone contactor, and (5) ozone destruction of exhaust gas, as well as some other considerations. The first three components have been discussed earlier. The other components are briefly presented below.

Ozone Contactor. Ozone is dispersed into water or wastewater in a contactor. The contactor could be a tall vertical water column, two-compartment tank, inclined packed column, static mixer, or high-speed agitator. The delivery of ozone-rich air under pressure can be made through a porous plate diffuser at the base or under negative pressure through an injector device or Venturi-type nozzle. The fine bubbles of the ozone and air mixture cause mass transfer, and oxidation and disinfection take place in the contactor. When ozone is supplied under pressure, the supply pressure must be enough to overcome the static pressure, exit pressure through the diffuser, and piping losses. If delivery is under negative pressure, the ejector device must create a vacuum action to draw the ozone mixture from the generator.

Initially, ozone applied is used up to satisfy the ozone demand, and then a residual is detected. Typical ozone residual concentrations range from 0.3–0.9 mg/L.[2-4,12] A typical ozone feed rate is 1.0–5 mg/L, depending on the purpose. Contact time is difficult to determine because of mixing and turbulence in the contact chamber. Several chambers connected in a series provide plug flow conditions. A two-compartment system is shown in Figure 14-11. The ozone dose is best determined by pilot studies. A typical schematic flow diagram of an ozone generation and application system is shown in Figure 14-12.

Ozone Destruction System. The exhaust gas from a contactor generally contains ozone concentration greater than 1 g/m^3 (500 ppm by volume). The ozone must be destroyed or recycled. Most systems, including those that use oxygen feed gas, also have an ozone destruction system because it is more cost-effective. The ozone destruction system utilizes four primary methods: (1) thermal (300–350°C for 3 seconds), (2) thermal/catalytic, (3) catalytic, and (4) moist granular activated carbon. The most common catalyst is a metal oxide. A cartridge made of granular manganese oxide is an effective catalyst for ozone destruction.

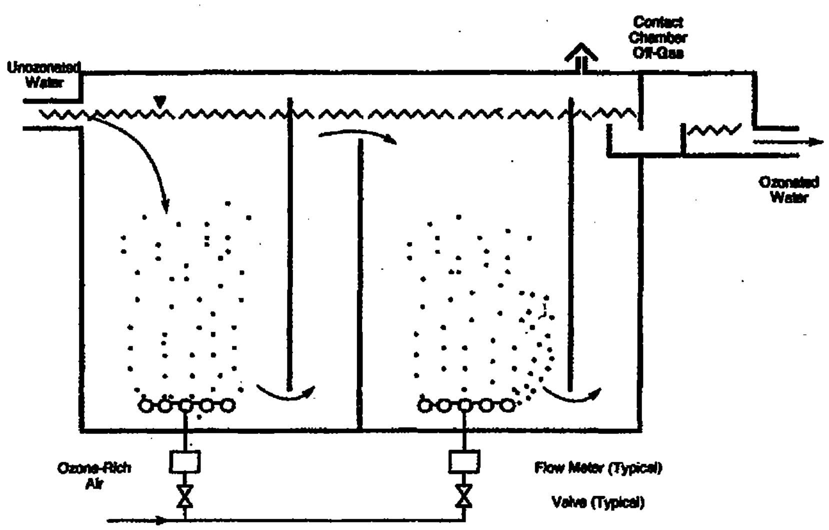


Figure 14-11 Two-Compartment Ozone Contactor with Porous Diffusers (from Ref. 12).

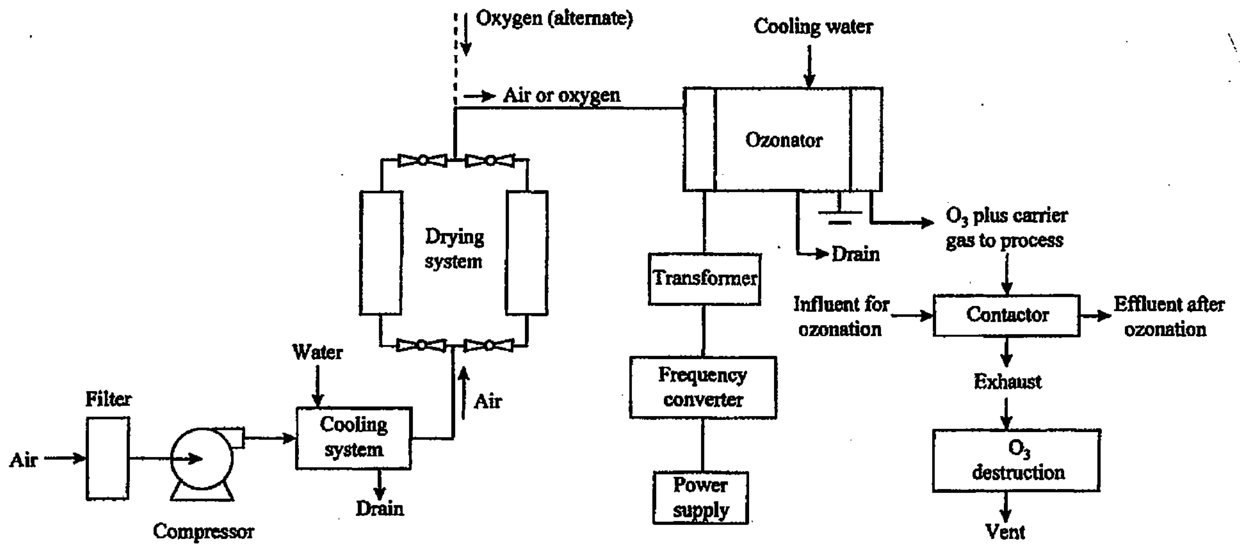


Figure 14-12 Ozonation System Schematic (from Ref. 12).

Other System Design Considerations. Materials of construction for ozone generation and application equipment should be capable of resisting the strong oxidizing and corrosive effects of ozone. Recommended construction materials are a reinforced concrete tank for the contactor, stainless steel for piping (dry gas 304-L stainless steel, wet gas 316-L stainless steel), and Teflon® for gaskets. Many valves, gauges, metering, and monitoring systems are also required. Following is a list of some of this equipment:

- gas pressure and temperature gauges or monitors at the air preparation unit
- continuous monitoring of dewpoint of feed gas to ozonator, alarm at high dewpoint, and generator shutdown
- air temperature monitoring of feed and exit gases from the ozonator, temperature of coolant media (water, oil, or air), and an automatic shutdown of ozone generator if coolant flow is interrupted or if temperature and discharge pressure exceed a set value

- flow rate, ozone concentration, temperature, and pressure monitors of ozone gas line
- ozone monitor for plant ambient air (in case of leaks)
- ozone monitoring for exhaust gas
- separate power input monitors for ozonator and air preparation units

14-9 UV DISINFECTION

The use of ultraviolet (UV) radiation for disinfection of wastewater is an emerging technology. It was first introduced almost 2 decades ago. This process has been described in the EPA Task Force Report as potentially advantageous over conventional disinfection by chlorination and ozonation.[12] In addition, it is a physical process, whereas all others are chemical processes, which leave chemical residuals. In this section the current state of knowledge on the design of the UV disinfection process is presented. Being a relatively new technology, the process design procedures are still being developed, and field data on the operation and maintenance are limited. More information will be available in the future as new plants come on-line.

14-9-1 Mechanism of UV Disinfection

Disinfection by UV radiation is a physical process that relies on the transfer of electromagnetic energy from a source (lamp) to the cellular material (protein and nucleic acids) of an organism. The basic premise of the UV disinfection process is that radiation must be absorbed by the organisms so that the energy can have a damaging effect. The UV light that is absorbed by the organisms is measured by *reflectance,* or *transmittance.* The DNA molecule of an organism is the principal target of UV photons. When UV energy is absorbed by the genetic material (DNA) of an organism, structural changes or damage occur that may prevent the propagation of the organisms. The absorbance of UV radiation by DNA molecules depends on the wavelength of the radiation. It has been shown that the most effective spectral region for germicidal effect is in the range of 250–265 nm with an optimum absorbance by nucleic acid around 254 nm.[12]

Repair of the damaged DNA molecules occurs when injured organisms are exposed to the visible range (primarily blue spectrum) subsequent to their UV exposure. This repair phenomenon has been described as *photoreactivation* and has been detected in many organisms.[20] The repair is enzymatic and requires light in the wavelength of 310–500 nm to complete the repair of damage caused by UV exposure. Among the species most often addressed by wastewater discharge permitting, the *E. coli* will photorepair while *Enterococci* cannot. Viruses do not have this ability except when in a host cell that can repair. Maximum repair occurs during summer months. A maximum of 1.5 log increase in total coliform count may occur in receiving waters after UV exposure.[21]

14-9-2 The Source of UV Light

The primary source of UV energy, at present, is the low-pressure mercury lamp. It is almost universally accepted as the most efficient and effective source of UV radiation for disinfection systems. The primary reason for its acceptance is that approximately 85 percent of its energy output is nearly monochromatic at the wavelength of 253.7 nm. This is within the optimum wavelength for germicidal effects. The lamps are tubes, typically 0.75–1.5 m in length and 1.5–2.0 cm in diameter. Some manufacturers supply shorter, thicker and high-pressure lamps. The radiation is generated by striking an electric arc through mercury vapor, which results in the emissions of UV light. Approximately 35 to 40 percent of the energy is converted to light, and approximately 85 percent of the light has a wavelength of 253.7 nm.

14-9-3 Types of UV Reactors

UV reactors are of two basic types: contact and noncontact. In both types the liquid flow may be parallel or perpendicular to the lamp. Different types of UV reactors are compared in Table 14-6. The lamp arrangement and technical details are shown in Figure 14-13.

14-9-4 General Description of UV Process

Several mathematical models have been proposed to estimate the inactivation of bacteria caused by UV radiation. These models are mostly based on first-order kinetics. The ideal

TABLE 14-6 Comparison of Contact and Noncontact UV Reactors

Reactor Type	Description	Flow Direction	Flow Type
Contact	The lamps are submerged at all times in wastewater. The lamps are encased in quartz sleeves that are slightly larger in diameter than the lamps. The lamp module or rack is encased in a sealed shell.	Parallel to lamps or perpendicular to lamps	Pressure or open channel
Noncontact	Lamps or quartz sleeves do not come in contact with the liquid. The lamps are suspended above the liquid or surround the Teflon® conduits that carry the liquid. These Teflon® conduits are transparent to UV light. The lamps are placed outside and parallel to the conduits or as an inserted removable rack (either vertically or horizontally) between the tubular rows.	Parallel to lamps or perpendicular to lamps	Pressure or gravity flow

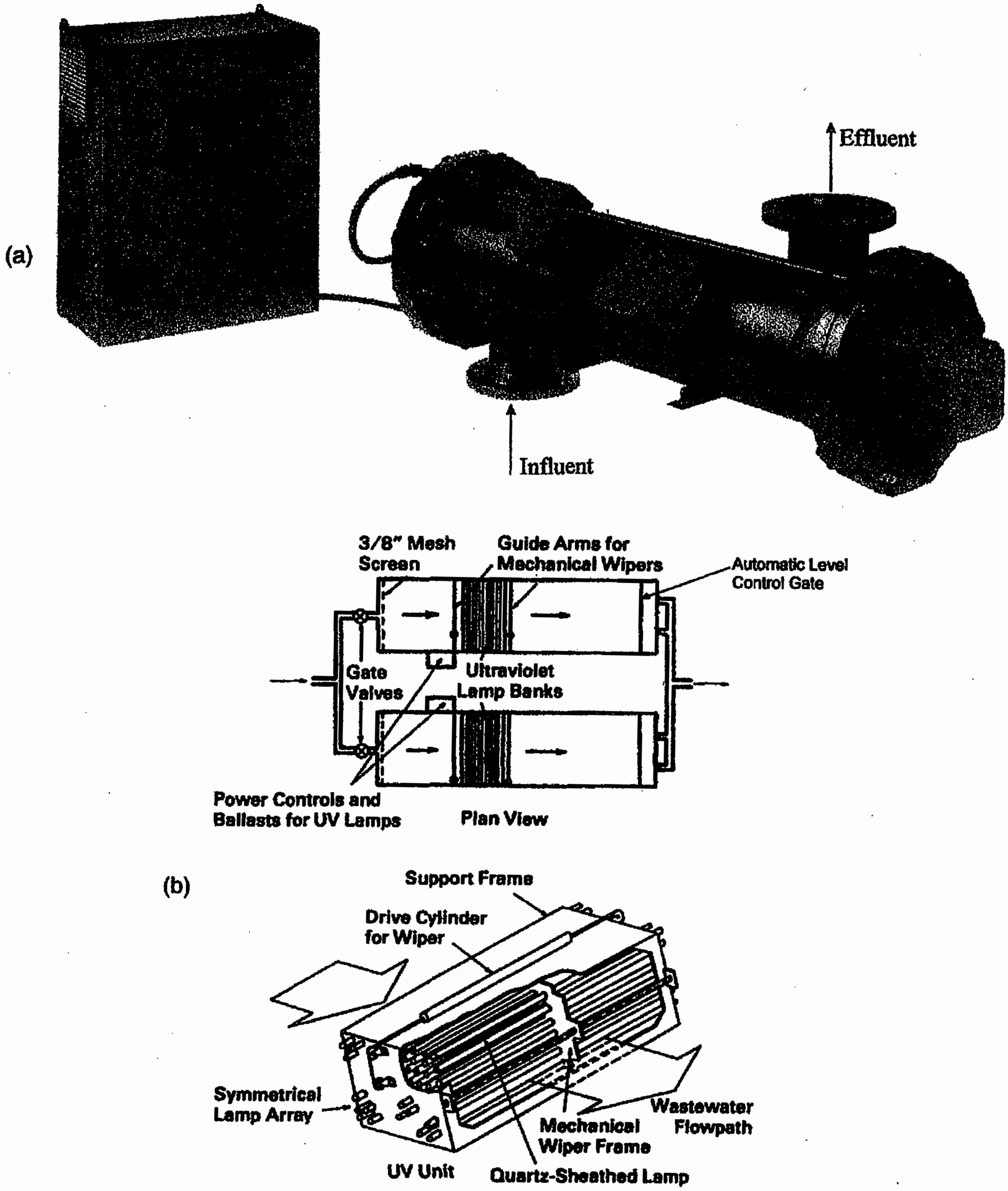
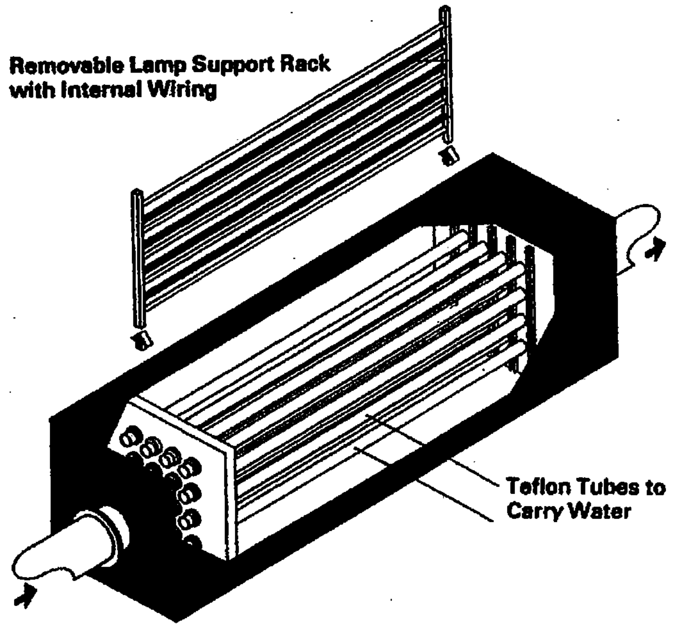


Figure 14-13 · System Components of UV Disinfection Reactors: (a) example of closed vessel UV reactor, with flow parallel to lamps (courtesy of Trojan Technologies Inc.); (b) contact-type reactor with flow direction perpendicular to the lamps, and open channel flow (from Ref. 21).



(c)

Figure 14-13—cont'd (c) example of UV system utilizing Teflon® tubes (from Ref. 12).

UV disinfection model is expressed by Eq. (14-30). The logarithmic expression of this equation yields a linear relationship [Eq. (14-31)].

$$N_t = N_0\, e^{-kIt} \tag{14-30}$$

$$\ln (N_t/N_0) = -kIt \tag{14-31}$$

where

N_t = bacterial density remaining after exposure to UV and often after photorepair, numbers per unit volume of liquid
N_0 = initial bacterial density, numbers per unit volume of liquid
I = intensity of UV radiation, $\mu W/cm^2$. A procedure to determine UV intensity is covered in Sec. 14-9-5.
t = exposure time, min
k = rate constant, t^{-1}

The intensity I is the rate at which the energy is delivered to the liquid. The product of UV intensity and time of exposure is called UV dose. The rate constant k is the slope of the line [Eq. (14-31)]. The above equations represent the idealized situations. In practice, however, deviation from the ideal model may occur because of many interfering effects, such as the following:

- Tailing effect is attributed to occulation or shadowing of bacteria by suspended particles. Filtration prior to UV disinfection is recommended to increase the efficiency.
- There is minimal or no response below a threshold dose (shoulder).

The tailing and shouldering effects are shown in Figure 14-14.

The intensity of UV radiation will attenuate as the distance from the lamps increases. This is caused by dissipation of energy in increasing surrounding volume or space. A second attenuation mechanism involves the actual absorption of the energy by physical, chemical, and biological constituents in the wastewater. This is *UV demand* and is analogous to *chlorine demand.* The UV demand of a wastewater is quantified by absorption of energy per unit depth. This is expressed by absorbance units per cm (au/cm). The au/cm is related to transmittance of UV light as measured by a spectrophotometer and is expressed by Eq. (14-32):

$$\% \text{ Transmittance} = 100 \times 10^{-(\text{au/cm})} \tag{14-32}$$

In most designs, a coefficient α is used to express UV absorbance. The coefficient α is to the base e and is directly related to au/cm. This relationship is expressed by Eq. (14-33):

$$\alpha = 2.3 \text{ (au/cm)} \tag{14-33}$$

The unit of α is cm^{-1}.

The suspended solids in the liquid reduces the efficiency of UV radiation. Experiments with mixed culture have shown that, as the UV dose is increased, the efficiency of inactivation is also increased but in a reduced proportion. The main reason is the aggregation or occulation of bacteria in particulate matter.[22-26] As a result the UV light is unable to penetrate aggregated material, and inactivation of the trapped bacteria does not occur. Thus, continued increase in the UV dose shows a reduced efficiency of inactiva-

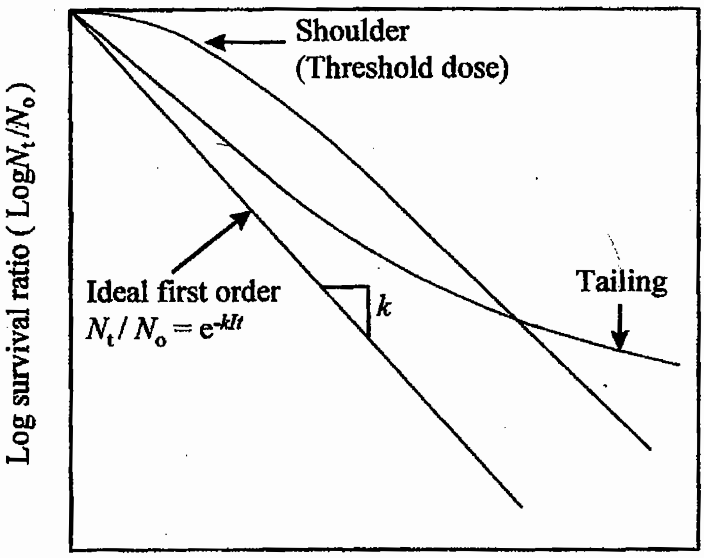


Figure 14-14 Illustration of UV Model and the Interfering Effects (from Ref. 12).

tion. This phenomenon is shown in Figure 14-15. Eq. (14-30) is modified to include this effect:

$$N_t = N_0\, e^{-kIt} + N_p \tag{14-34}$$

where

N_p = particulate bacterial density that is unaffected by UV light

The particulate bacterial density N_p is generally expressed as a function of some measurable index, such as total suspended solids (TSS) or turbidity. Total suspended solids is generally used because it is the parameter most commonly regulated in the effluent. The value of N_p is conveniently expressed as a function of TSS by Eq. (14-35):

$$N_p = c_1(\text{TSS})^{m_1} \tag{14-35}$$

where

TSS = total suspended solids, mg/L
c_1 = proportionality constant
m_1 = constant

From Eq. (14-34), it is apparent that the exposure time t is an important factor to achieve the desired level of inactivation. Thus for a UV system, one of the design objectives is to achieve a typical exposure time. In reality, however, various particles passing through the reactor will have different exposure times. In fact, there will be a distribution

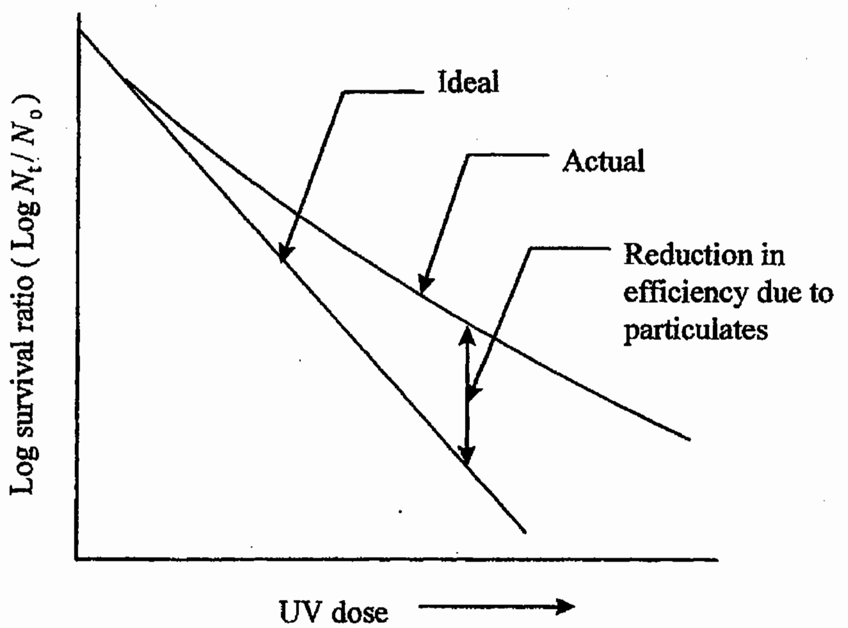


Figure 14-15 Effect of Particulate Matter on UV Disinfection Efficiency (from Ref. 12).

of time around the ideal time, which is known as the residence time distribution (RTD). The RTD is a function of the dispersion characteristics of a reactor. A disinfection model that incorporates dispersive properties of a reactor was developed by Scheible et al.[24] A generalized expression of this model is given by Eq. (14-36):

$$N_t = N_0 \, exp\left[\frac{ux}{2E}\left\{1 - \left(1 + \frac{4kE}{u^2}\right)^{1/2}\right\}\right] + N_p \tag{14-36}$$

where

x = the characteristic length of the reactor, defined as the average distance traveled by an element of water while under direct exposure to UV, cm
u = velocity of water, cm/s
E = dispersion coefficient (cm^2/s). E can be estimated from the RTD curve of a particular reactor system (more information on this subject is given in the Design Example)
k = bacterial inactivation rate, s^{-1}
N_t, N_0 and N_p have been defined earlier.

The performance curve of a UV reactor is generally based on the nonparticulate effluent fecal coliform density, N'. Since the particulate fecal coliform density N_p is additive, Eq. (14-36) can be modified in terms of N' and is expressed by Eq. (14-37):

$$N'/N_0 = \exp\left[\frac{ux}{2E}\left\{1 - \left(1 + \frac{4kE}{u^2}\right)^{1/2}\right\}\right] \tag{14-37}$$

where

$$N' = \text{nonparticulate effluent coliform density} = N_t - N_p$$

The inactivation rate k is expressed as a function of the UV intensity. Thus, for a given exposure time, k will increase, depending upon the light intensity I. A linear relationship between k and I is expressed by Eq. (14-38):

$$k = a\,(I_{avg})^b \tag{14-38}$$

where

a and b = slope and intercept of the linear regression developed from log function of the equation
I_{avg} = average intensity of light in the reactor, $\mu W/cm^2$

Eqs. (14-36) and (14-38) are combined to give a generalized UV design model. This model is expressed by Eq. (14-39):

$$N_t = N_0 \exp\left[\frac{ux}{2E}\left\{1 - \left(1 + \frac{4Ea(I_{avg})^b}{u^2}\right)^{1/2}\right\}\right] + c_1(\text{TSS})^{m_1} \tag{14-39}$$

All terms in Eq. (14-39) have been defined earlier.

14-9-5 Determination of UV Intensity

Determination of UV intensity at any point in a complex lamp reactor is not straightforward. At the present time there is no commercially available detector that can measure the true intensities in such a system. Moreover, different lamp configurations will yield different nominal intensities in the reactor. Calculations are performed for a number of designs and subsequently reduced to show the intensity as a function of UV density of the reactor and UV energy absorbed.

The UV density, D is defined as the total nominal UV power (at 253.7 nm) available within a reactor divided by the liquid volume of the reactor (D = total UV output/liquid volume). The density is directly related to the spacing of the lamps; the closer the spacing, the higher is the UV density in the reactor. Typical spacing configurations are as follows:

- uniform array: The lamps are arranged in even, horizontal rows and vertical columns. The centerline spacings are generally equal in both directions [Figure 14-16(a)].
- staggered uniform array: The array is similar to uniform array, except that the alternating vertical rows are offset by one-half of vertical spacing. The staggered effect is designed to induce turbulence [Figure 14-16(b)].
- centric array: The lamps are arranged in concentric circles [Figure 14-16(c)].
- tubular array: The lamps are suspended outside and parallel to a Teflon® conduit. The lamps and tubes are stacked vertically in alternating rows. The equivalent vertical and horizontal centerline spacing between lamps is generally kept equal [Figure 14-16(d)].

Generalized graphical relationships have been developed for four arrangements of tubes to obtain average nominal intensity from UV absorbance coefficient and UV density for different lamp arrangements. These relationships are given in Figure 14-17. It is important to note that, in Figure 14-17, the intensity of UV radiation is calculated by assuming that the quartz sleeve or Teflon® tube transmits 100 percent of the energy emitted by the lamps. That is why it is expressed as calculated nominal average intensity. Under actual operation and for design purposes, the nominal average intensity of lamps must be adjusted to account for the aging of the lamps and consequent reduction of UV output caused by losses of energy as it passes through the quartz sleeves or Teflon® walls. These losses are caused by fouling of the inner side of the Teflon® tubes and the outer surface of the quartz sleeves because of the contact with wastewater. Thus, to es-

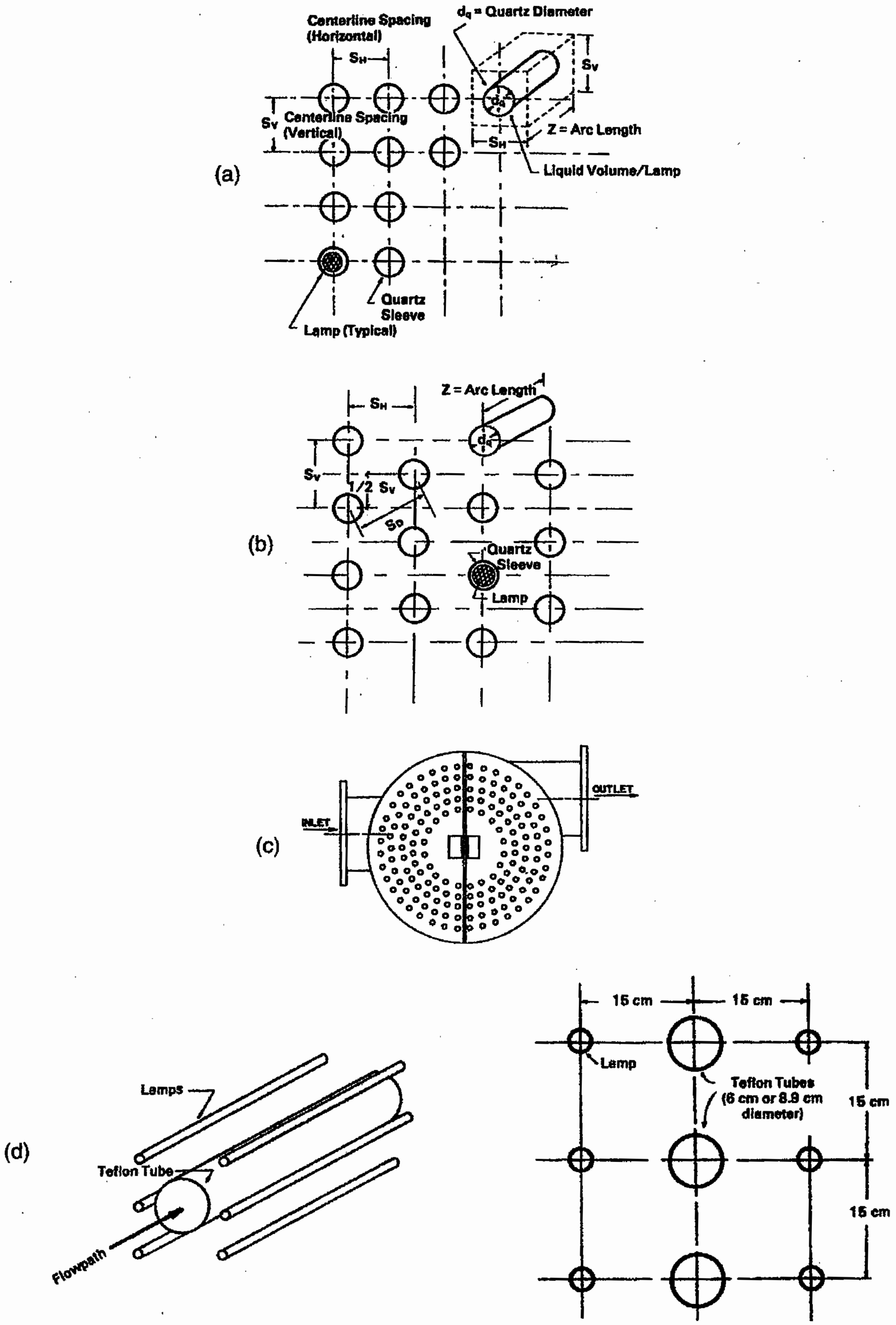


Figure 14-16 Schematic of Different Lamp Arrangements (from Ref. 12): (a) uniform array, (b) staggered uniform array, (c) concentric array, and (d) tubular array.

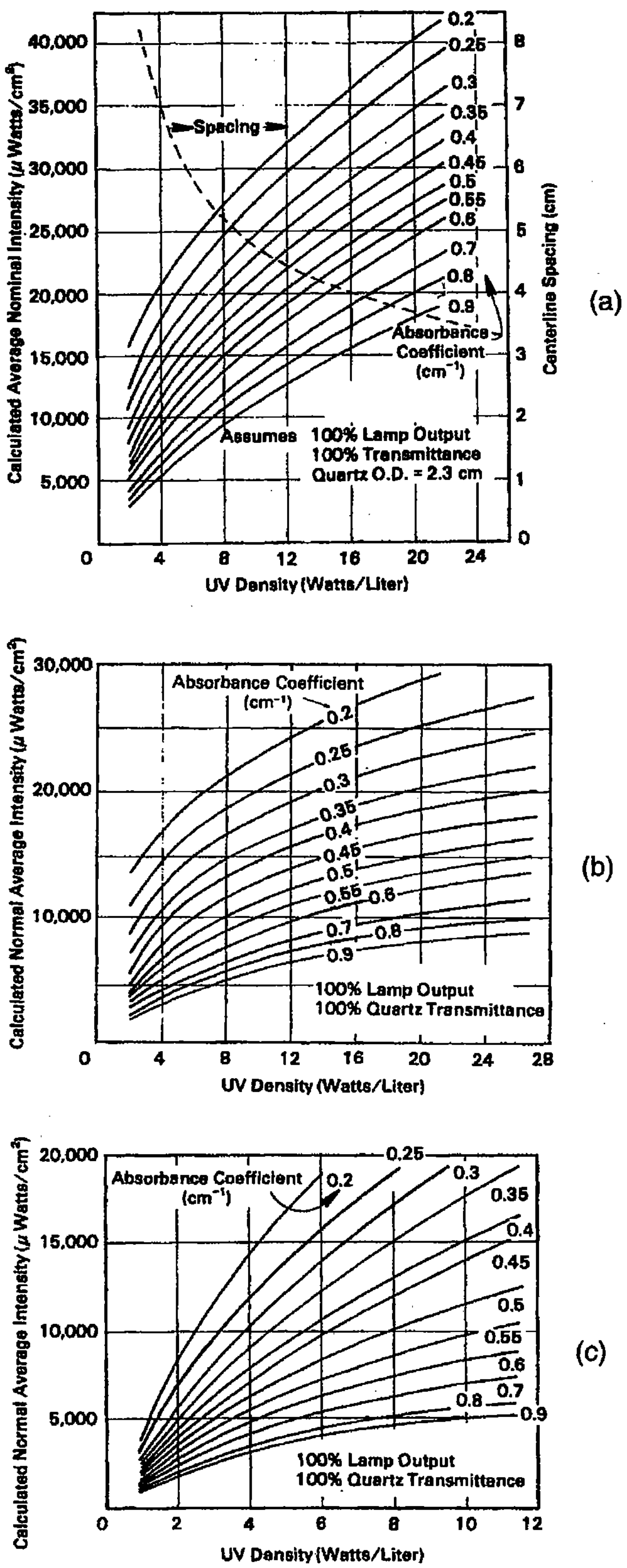


Figure 14-17 The Average Nominal Intensity as a Function of the Reactor UV Density and UV Absorbance Coefficient for Different Lamp Arrangements (from Ref. 12): (a) uniform array, (b) staggered uniform array, (c) concentric array.

Continued

(d)

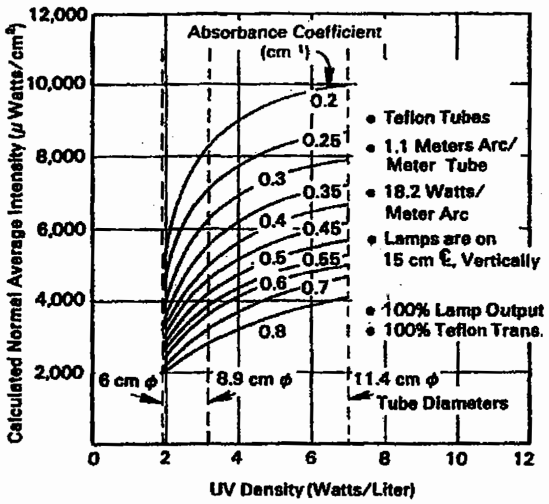


Figure 14-17—cont'd (d) tubular array.

timate the actual intensity under a given set of field conditions, it is necessary to adjust the nominal intensity. Eq. (14-40) is normally used for this purpose.

$$I_{\text{avg}} = (\text{Nominal } I_{\text{avg}}) \times F_p \times F_t \tag{14-40}$$

where

F_p = ratio of the actual output to the nominal output of the lamps
F_t = ratio of the actual transparency of the quartz sleeve or Teflon® tubes to the nominal transparency (100%) of the enclosures

When designing a new unit, the system should be designed at an average F_p of 0.8, which represents a lamp inventory output at approximately one-half of its operating life. An F_p of 0.65 is expected at the end of the operating life. A reasonable minimum F_t would be 0.7 for a quartz system and 0.6 for a Teflon® system.

14-9-6 Hydraulic Design of UV Reactor

There are three basic design objectives of an effective UV disinfection reactor: (a) the unit should be a plug flow reactor (PFR), and (b) the flow motion should be turbulent radially from the direction of flow. This allows each element to receive the same overall average intensity of radiation in the nonuniform intensity field that may exist in the reactor. The trade-off in this requirement is that some axial dispersion will be introduced, yielding a dispersive or nonideal plug flow reactor, and (c) maximum use must be made of the entire volume of the reactor. Dead spaces must be minimized so that the effective volume is very close to the actual volume available. The basic factors for a UV reactor design are discussed below.

Residence Time Distribution (RTD). The RTD provides key information on the actual or anticipated hydraulic behavior of a reactor. Dye-tracer study should be conducted to establish the actual residence time.

Dispersive Characteristic. A key goal of hydraulic design of the UV reactor is to minimize the dispersion and maximize the contact time. Therefore, the average velocity profile should be established at different flow conditions.

Turbulence and Head Loss. An important consideration in the hydraulic design of a UV reactor is fluid turbulence. Some turbulence is desired for uniform distribution of UV energy in the nonuniform intensity field of the reactor. But turbulence is associated with head loss, which is a function of velocity. Head loss is the controlling factor in the design of a reactor. This topic is discussed in the Design Example.

Effective Volume. Maximum use of the reactor volume is another important design consideration. The lamp battery volume is the portion of the total vessel volume that is occupied by UV lamps. It is therefore important that the reactor is designed to utilize the maximum vessel volume. Dead zones or short-circuited areas would also mean ineffective use of lamp power. Both components of the system (reactor and lamp battery) comprise a major portion of the capital and operating costs.

14-10 EQUIPMENT MANUFACTURERS OF A UV DISINFECTION FACILITY

A number of equipment manufacturers supply UV disinfection systems. Names and addresses of many manufacturers are given in Appendix D. The design engineer should work closely with the local representatives of the equipment suppliers. A good performance record of UV disinfection equipment at other installations should be reviewed before equipment selection is made. Responsibilities of the design engineer for proper equipment selection are given in Sec. 2-10.

14-11 INFORMATION CHECKLIST FOR DESIGN OF A UV DISINFECTION FACILITY

The following information must be obtained and decisions made before the design engineer should proceed with the design of a UV disinfection facility:

1. Effluent TSS (must be typically less than 20 mg/L)
2. Peak wet weather, peak dry weather, average design, and minimum initial flows
3. Treatment plant design criteria prepared by the concerned regulatory agency
4. Specified contact time and corresponding flow condition (peak or average)
5. UV absorbance or transmittance of the effluent to maintain the desired level of germicidal effect; color caused by industrial wastes can be serious.
6. Equipment manufacturers and equipment selection guide
7. Information about existing facility if plant is being expanded
8. Head loss constraints through the unit; the automatic level control gate requires a minimum of 0.4 m of head loss.
9. Existing site plan with contours and location of the disinfection system
10. Influent and effluent seasonal coliform count for specific flows (annual average,

maximum 7-day average, maximum 30-day average, peak dry weather, and peak wet weather)
11. Amount of redundancy required
12. Attitude and capability of operating and maintenance staff towards cleaning the lamps and providing continued operation under adverse conditions
13. Likely future expansion of the plant and anticipated design changes such as module size, number of banks, and number of channels

14-12 DESIGN EXAMPLE

14-12-1 Design Criteria Used

The following design criteria shall be used for the design of the UV disinfection system.

Step A: General

1. The plant's permit calls for year-round disinfection.
2. The influent to the UV disinfection system is the effluent from the secondary clarifiers. Many manufacturers recommend a coarse screen in the channel ahead of the UV disinfection facility.
3. Provide a Parshall flume ahead of the UV disinfection facility to measure the flow. The flow data are necessary to operate the required number of channels and lamp modules.
4. The following flow shall be used for the design of the UV system:
   - average daily wastewater flow = 0.440 m^3/s (26,400 Lpm) (Table 6-9)
   - maximum 7-day average flow = 0.660 m^3/s (39,600 Lpm). This flow is based on 1.5 × (avg. daily flow)
   - maximum 30-day average flow = 0.484 m^3/s (29,040 Lpm). This flow is based on 1.1 × (avg. daily flow)
   - peak dry weather flow = 0.917 m^3/s (55,000 Lpm) (Table 6-9)
   - peak wet weather flow = 1.321 m^3/s (79,300 Lpm) (Table 6-9)
5. The following TSS values shall be used for the influent to the plant:
   - average daily TSS = 10 mg/L
   - maximum 7-day average TSS = 20 mg/L
   - maximum 30-day average TSS = 10 mg/L
6. Influent fecal coliform density:
   - average daily = 5×10^5 org[d]/100 mL
   - maximum 7-day = 2×10^6 org/100 mL
   - maximum 30-day = 1×10^6 org/100 mL
   - peak dry weather = 1×10^6 org/100 mL
   - peak wet weather = 0.5×10^6 org/100 mL
7. Required effluent fecal coliform density:
   - average daily = 100 org/100 mL
   - maximum 7-day average = 200 org/100 mL
   - maximum 30-day average = 100 org/100 mL

[d]Organisms.

- peak dry weather flow = 200 org/100 mL
- peak wet weather flow = 400 org/100 mL

8. UV transmittance (at 253.7 nm wavelength of UV)
   - daily average flow = 70%
   - maximum 7-day average flow = 60%
   - maximum 30-day average flow = 65%
9. Assume UV transmittance at peak wet weather flow = 70% and at peak dry weather flow = 65%.

Step B: Reactor. The disinfection models presented earlier will be used to determine the optimum design for the stated application. Several design scenarios and unit configurations will be evaluated to maximize the loading to the system while still meeting the performance goals.

The basic assumptions for the reactor design are

1. Provide uniform lamp array.
2. The center to center spacing of the lamps shall be 6.0 cm.
3. The lamps shall be 1.5 m long with an effective arc length of 1.47 m; the nominal UV output is approximately 18.2 W (watts)/m arc.
4. Each lamp is sheathed in a quartz enclosure with an outer diameter of 2.3 cm.
5. The lamps shall be configured axially parallel to one another, and the flow path will be parallel to the lamps.
6. The values of the energy loss factors, F_p and F_t [Eq. (14-39)], are 0.8 and 0.7, respectively.
7. The coefficient a, b, c_1, m_1, and E in Eqs. (14-35), (14-36), and (14-38) are generally developed experimentally because they are site-specific. The typical values of these coefficients for disinfection of secondary effluent from a POTW are used here.[12] These typical values are

$$\begin{aligned} a &= 1.45 \times 10^{-5} \\ b &= 1.3 \\ c_1 &= 0.25 \\ m_1 &= 2.0 \end{aligned}$$

   E varies in the range of 120–1300 cm^2/s in proportion to the flow.

14-12-2 Reactor Type and Arrangement

The UV system designed in this example utilizes four open channels arranged in parallel. Each channel has two banks of UV lamps in series. Each bank contains several UV modules in a *uniform array.* The flow is parallel to the UV lamps. The flow from a Parshall flume enters a common influent division channel that divides the flow into the UV disinfection channels. A sluice gate and identical rectangular weir at the head of each disinfection channel divide the flow equally into each channel. The entire system consists of UV modules, power distribution centers, and all necessary interconnecting cables, along with a host of accessory components. The effluent structure has an automatic control flap gate level controller. The flow from the flap drops into a common collection channel. The system configuration for the Design Example and layout of other UV system installation supplied by equipment manufacturers are shown in Figure 14-18.

14-12-3 Design Calculations

Step A: Disinfection Reactor Design. The reactor design involves UV lamp selection, arrangement and cleaning, module and bank arrangement, channel sizing, and head loss calculations.

1. Calculate the UV density of the reactor.
   The UV density is calculated from the volume of liquid exposed to light. Two banks of the UV module are placed in series in each channel.
   a. Calculate the volume of liquid (V_v) exposed per lamp.
      The lamp spacings in both vertical and horizontal directions are 6 cm (Figure 14-19). The volume of liquid exposed per lamp is calculated from Eq. (14-41):

$$V_v/\text{lamp} = (S^2Z) - (\pi d_q^2/4)Z \tag{14-41}$$

where

S = center to center spacing between the lamps = 6 cm
Z = arc length of the lamp = 147 cm
d_q = diameter of quartz sleeve = 2.3 cm

$$\text{Therefore, } V_v/\text{lamp} = \{(6.0\text{ cm})^2 \times (147\text{ cm})\} - \{\tfrac{\pi}{4}(2.3\text{ cm})^2 \times 147\text{ cm}\}$$
$$= 4700\text{ cm}^3 \text{ or } 4.7\text{ L}$$

   b. Calculate the UV density.

$$\text{The UV density, } D = \text{total UV output per lamp/liquid volume per lamp}$$
$$= (1.47\text{ m arc} \times 18.2\text{ W/m arc})/4.7\text{ L}$$
$$= 5.7\text{ W/L}$$

   c. Calculate the absorbance unit (au/cm), absorbance coefficient α, nominal UV intensity (nominal I_{avg}), and adjusted UV intensity (adjusted I_{avg}).
      (i) At average daily flow
      The absorbance unit (au/cm) is calculated from Eq. (14-32). At average daily flow, the UV transmittance at 253.7 nm = 70% (see Design Criteria)

$$70 = 100 \times 10^{-\text{au/cm}}$$
$$\text{au/cm} = 0.155$$

      The absorbance coefficient α is calculated from Eq. (14-33):

$$\alpha = 2.3 \times (0.155/\text{cm})$$
$$= 0.35/\text{cm}$$

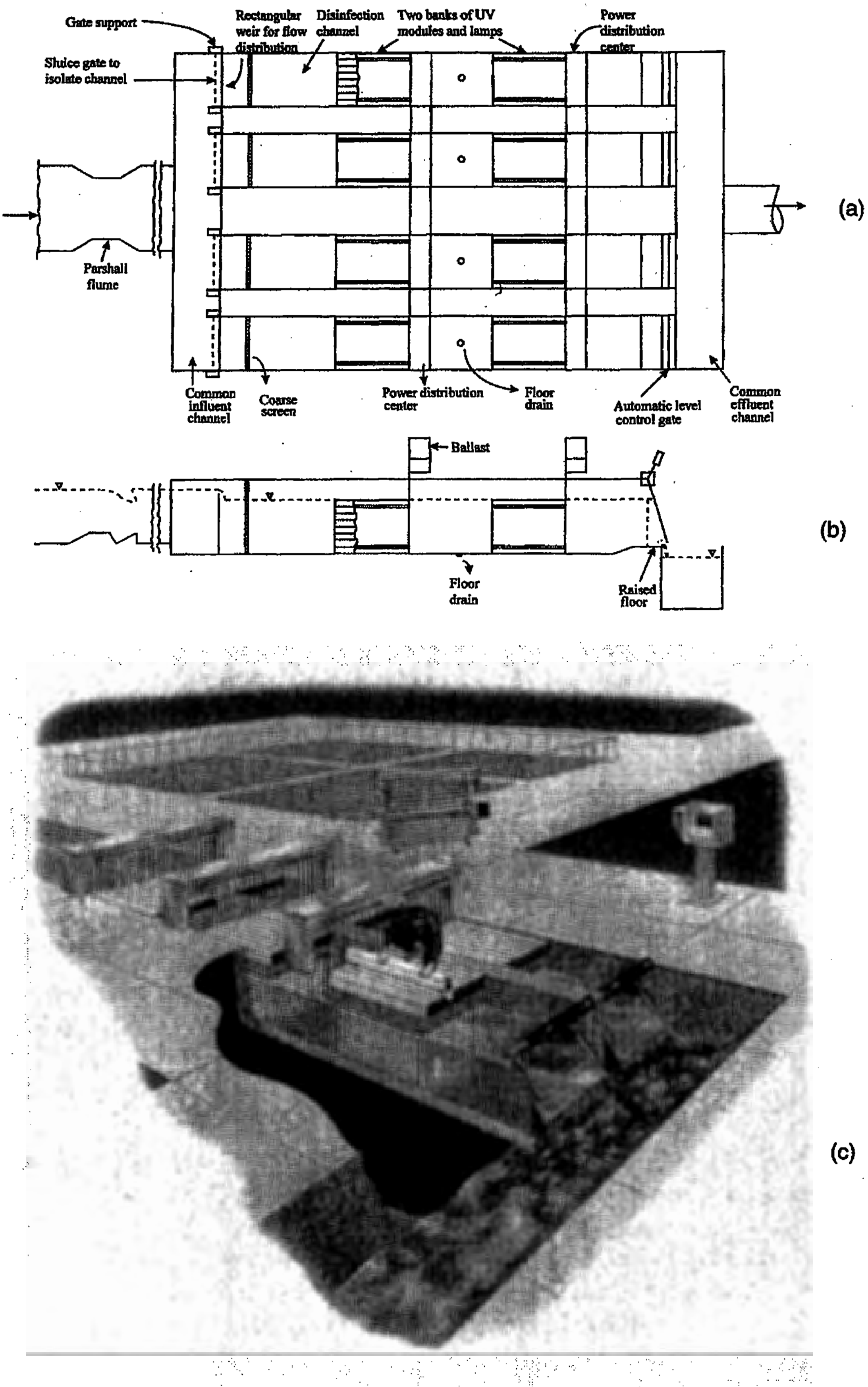


Figure 14-18 UV Disinfection System Layout: (a) plan view, (b) longitudinal section of proposed facility for the Design Example, and (c) a photograph of UV system with two channels and three banks (courtesy Trojan Technologies, Inc.).

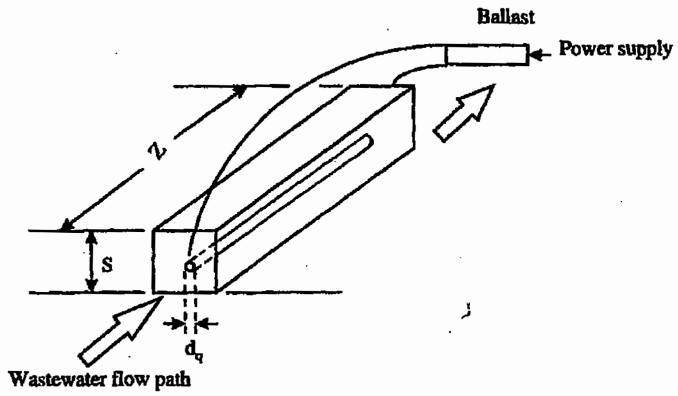


Figure 14-19 One UV Lamp and Flow Path (from Ref. 12).

At UV density = 5.7 W/L and α = 0.35/cm, the nominal I_{avg} is obtained from Figure 14-17(a):

$$\text{Nominal } I_{avg} = 17{,}300\ \mu\text{W/cm}^2$$

Adjusted I_{avg} is calculated from Eq. (14-40):

$$\begin{aligned}\text{Adjusted } I_{avg} &= 17{,}300\ \mu\text{W/cm}^2 \times 0.8 \times 0.7\\ &= 9700\ \mu\text{W/cm}^2\end{aligned}$$

These results are summarized in Table 14-7.

(ii) Maximum 7-d average and maximum 30-d average.
The above procedure is repeated to calculate the absorbance unit (au/cm), absorbance coefficient α, nominal UV intensity (nominal I_{avg}), and adjusted UV intensity (adjusted I_{avg}) for above flows. These values are also summarized in Table 14-7.

d. Calculate the inactivation coefficient k at different flows.
The inactivation coefficient k at average daily flow is calculated from Eq. (14-38). The values of coefficients a and b are 1.45×10^{-5} and 1.3, respectively (Design Criteria, Sec. 14-12-1, Step B, 7):

$$k = 1.45 \times 10^{-5}(9700)^{1.3} = 2.21\ \text{s}^{-1}$$

Similarly, the k values for the maximum 7-d average and maximum 30-d average flows are calculated. These values are summarized in Table 14-7.

2. Develop the performance curve.
The performance curve is developed from UV loading and exposure time.
a. Calculate the UV loading.
UV loading is defined as the volume of water exposed to the nominal UV wattage:

$$\begin{aligned}\text{UV loading} &= \frac{4.7\ \text{L/lamp}}{1.47\ \text{m arc/lamp} \times 18.2\ \text{W/m}\cdot\text{arc}}\\ &= 0.176\ \text{L/W}\end{aligned}$$

TABLE 14-7 *Adjusted Average UV Intensity and Inactivation Coefficients at Different Flow Conditions*

Design Conditions	UV Transmittance at 253.7 nm (%)	UV au/cm	UV Absorbance Coefficient α (cm^{-1})	Nominal I_{avg} ($\mu W/cm^2$)	Adjusted I_{avg} ($\mu W/cm^2$)	Inactivation coefficient k (s^{-1})
Average daily flow	70[a]	0.155	0.35	17,300	9700	2.21
Maximum 7-d avg	60[a]	0.220	0.51	13,000	7300	1.53
Maximum 30-d avg	65[a]	0.190	0.43	15,100	8450	1.85

[a]Values obtained from the Design Criteria (Sec. 14-12-1 Step A, 7).

b. Calculate nominal exposure time.
The nominal exposure time t_n is the average time the water is exposed to UV light.
For each lamp t_n is calculated from volume of water exposed per lamp (V_v) and flow rate per lamp (q):

$$t_n = \frac{\text{volume of water exposed/lamp}}{\text{flow rate/lamp}} = \frac{V_v, \text{liter (L)}}{q, \text{ liter per min (Lpm)}}$$

Divide numerator and denominator by W_n (watt):

$$t_n = \frac{V_v/W_n}{q/W_n}$$

To develop the UV performance curve, assign different values to variable q/W_n. For each assigned value of q/W_n, calculate the corresponding value of t_n. As an example, at $q/W_n = 0.5$ Lpm/W.

$$t_n = \frac{0.176 \text{ L/W}}{0.5 \text{ L/min·W}} \times 60 \text{ s/min} = 21.12 \text{ s}$$

Similarly, calculate t_n for other assigned values of q/W_n (q/W_n = 1.0, 2.0, 3.0, and 4.0 L per min per Watt). These values are summarized in Table 14-8.

c. Calculate the velocity through UV lamps in the channel.
The velocity through the UV banks is obtained from the nominal exposure time and the liquid exposure length under the UV lamps. There are two UV banks per channel, and flow is parallel to the lamps. The length of each lamp is 150 cm. Therefore, total length of UV exposure is 300 cm. The nominal exposure time t_n for $q/W_n = 0.5$ Lpm/W is 21.12 s. The corresponding velocity u is calculated below:

$$u = 300 \text{ cm}/21.12 \text{ s} = 14.20 \text{ cm/s}$$

Similarly, the u values for other assumed g/W_n values are calculated and summarized in Table 14-8.

d. Calculate the dispersion coefficient.
The dispersion coefficient E relates to the hydraulic regime, mixing, or the residence time distribution in a reactor. Experimental determination of RTD using a conservative (nonreactive) dye tracer is the most reliable technique. Dispersion models are used to evaluate or diagnose the flow regime in a specific reactor. Equation (14-42) is generally used to define the dispersion behavior in a reactor:

$$d = \frac{E}{ux} \tag{14-42}$$

where

d = dispersion number
E = dispersion coefficient, cm^2/s

TABLE 14-8 Calculated Performance Values log N'/N_0 at Daily Average, Maximum 7-d Average, and Maximum 30-d Average Flows

q/W_n	t_n (s)	x (cm)	u (cm/s)	k^a (s^{-1})	E (cm^2/s)	log N'/N_0
Average Daily Flow						
0.5	21.12	300	14.20	2.21	127.84	−11.37
1	10.56	300	28.41	2.21	255.68	−6.88
1.5	7.04	300	42.61	2.21	383.52	−5.02
2	5.28	300	56.82	2.21	511.36	−3.98
2.5	4.22	300	71.02	2.21	639.20	−3.31
3	3.52	300	85.23	2.21	767.05	−2.83
3.5	3.02	300	99.43	2.21	894.89	−2.48
4	2.64	300	113.64	2.21	1022.73	−2.20
4.5	2.35	300	127.84	2.21	1150.57	−1.98
5	2.11	300	142.05	2.21	1278.41	−1.80
Maximum 30-d Average						
0.5	21.12	300	14.20	1.85	127.84	−10.04
1	10.56	300	28.41	1.85	255.68	−6.01
1.5	7.04	300	42.61	1.85	383.52	−4.35
2	5.28	300	56.82	1.85	511.36	−3.43
2.5	4.22	300	71.02	1.85	639.20	−2.84
3	3.52	300	85.23	1.85	767.05	−2.43
3.5	3.02	300	99.43	1.85	894.89	−2.12
4	2.64	300	113.64	1.85	1022.73	−1.88
4.5	2.35	300	127.84	1.85	1150.57	−1.69
5	2.11	300	142.05	1.85	1278.41	−1.54
Maximum 7-d Average						
0.5	21.12	300	14.20	1.53	127.84	−8.76
1	10.56	300	28.41	1.53	255.68	−5.18
1.5	7.04	300	42.61	1.53	383.52	−3.73
2	5.28	300	56.82	1.53	511.36	−2.92
2.5	4.22	300	71.02	1.53	639.20	−2.41
3	3.52	300	85.23	1.53	767.05	−2.05
3.5	3.02	300	99.43	1.53	894.89	−1.79
4	2.64	300	113.64	1.53	1022.73	−1.58
4.5	2.35	300	127.84	1.53	1150.57	−1.42
5	2.11	300	142.05	1.53	1278.41	−1.29

[a]These values are taken from Table 14-7.

The variables u and x are defined earlier.

True plug-flow and complete-mix reactor regimes are reached when d approaches 0 and $\propto$, respectively. Performance of a disinfection system is high under plug flow conditions. At $d = 0.03$, low to moderate dispersion exists, and E values are calculated for different flow velocities. Sample calculations for $q/W_n = 0.5$ are given below:

$$0.03 = \frac{E}{14.2 \text{ cm/s} \times 300 \text{ cm}}$$

$$\text{or } E = 27.8 \text{ cm}^2\text{/s}$$

Similarly, the values of E for other assumed q/W_n values are calculated and are summarized in Table 14-8.

e. Calculate UV performance values, $\log N'/N_0$.

The UV performance values $\log N'/N_0$ for average daily, maximum 7-d average, and maximum 30-d average flow conditions are calculated from Eq. (14-37). Sample calculations for $q/W_n = 0.5$ and average daily flow are shown below. It may be noted that if the effect of photoreactivation in the effluent or receiving water is considered, then the procedure for developing the performance goal should be modified.

$$N'/N_0 = \exp\left[\frac{ux}{2E}\left\{1 - \left(1 + \frac{4kE}{u^2}\right)^{1/2}\right\}\right]$$

The values of u and E are given in Table 14-8, $x = 300$ cm, and $k = 2.21 \text{ s}^{-1}$.

$$N'/N_0 = \exp\left[\frac{14.2 \text{ cm/s} \times 300 \text{ cm}}{2 \times 127.8 \text{ cm}^2\text{/s}}\left\{1 - \left(1 + \frac{4 \times 2.21 \text{ s}^{-1} \times 127.8 \text{ cm}^2}{(14.2 \text{ cm})^2}\right)^{1/2}\right\}\right]$$

$$= \exp(-26.17)$$

$$\ln N'/N_0 = -26.17$$

$$\log N'/N_0 \times \ln 10 = -26.17$$

$$\log N'/N_0 = -26.17/2.3 = -11.37$$

Similarly, the performance values for the three flow conditions are calculated and arranged in Table 14-8. The performance curves ($\log N'/N_0$ versus q/W_n) for the three flow conditions are developed and are shown in Figure 14-20.

3. Establish the performance goals.

The performance goals of the UV disinfection facility are developed from the permit requirements of the fecal coliform. The values of N_p are calculated from Eq. (14-35), with the coefficient c and m_1 equal to 0.25 and 2.0, respectively (see Design Criteria). These values are then subtracted from the permitted effluent fecal coliform densities to yield the design performance goals (N'org/100 mL). This value is then used to compute $\log (N'/N_0)$ for the design purposes. The goals are established for

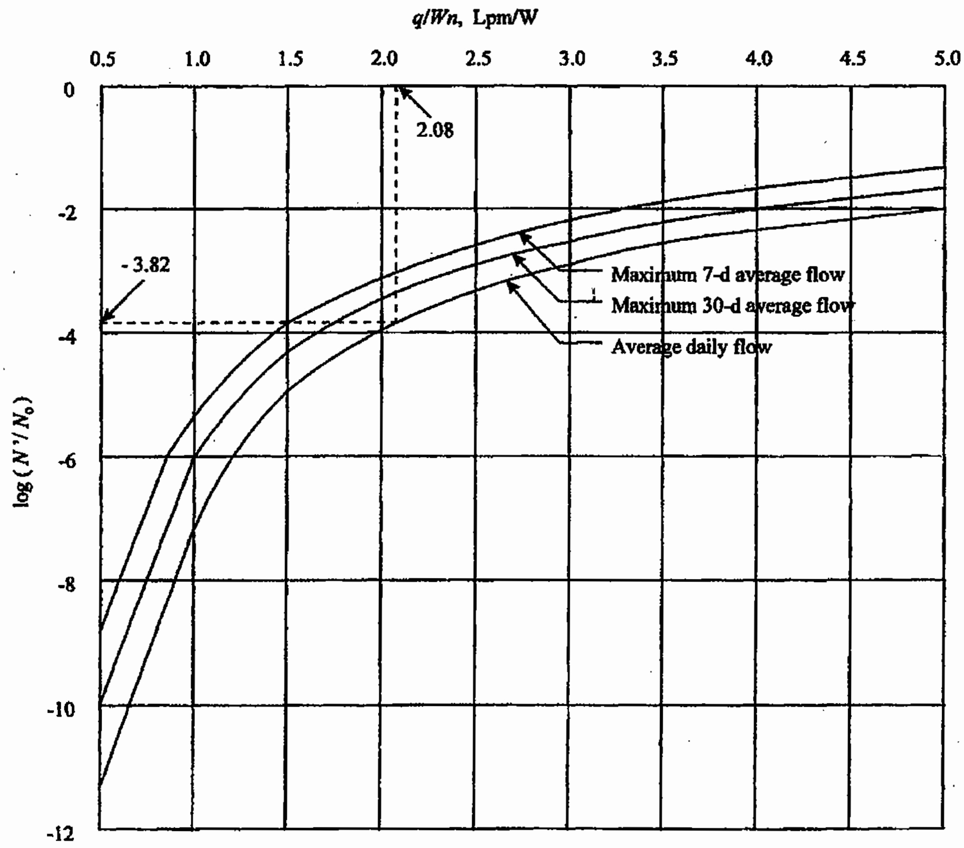


Figure 14-20 Performance Curves.

daily average flow, maximum 7-d average, and maximum 30-d average. Various values are summarized in Table 14-9.

4. Determine UV-loading (q/W_n) to achieve performance goals.
   The performance curves are used to obtain the required UV-loading (q/W_n) to achieve the performance goals. The UV-loading (q/W_n) for average day, maximum 7-d average, and maximum 30-d average are read directly from the corresponding performance curves in Figure 14-20, to achieve the performance goals established in Table 14-9. For example, for average day flow and the required removal of log (N'/N_0) = -3.82, the UV-loading (q/W_n) is obtained from Figure 14-20. This value is 2.08. Similarly, the values for maximum 7-d average and maximum 30-d average are also obtained from Figure 14-20. These values are summarized in Table 14-10. The performance curves for design peak dry weather and design peak hour flows are not developed. The UV-loadings q/W_n for these flows are estimated from the closest performance curves (Figure 14-20). The UV-loading values for these flows are also summarized in Table 14-10.
5. Calculate the nominal exposure time t_n.
   The value of nominal exposure time t_n is the time required to achieve the required disinfection. It is obtained from the equation,

$$t_n = \frac{0.176}{q/W_n} \times 60\ s$$

TABLE 14-9 Estimate of Reactor Performance Requirement for the Design Example

	Daily Average	Maximum 7-Day Average	Maximum 30-Day Average	Peak Dry Weather	Design Peak Hour
Initial fecal coliform density,[a] N_0 (org/100 mL)	5×10^5	2×10^6	1×10^6	1×10^6	0.5×10^6
Total suspended solids (mg/L)	10	20	10	20[b]	20[b]
Particulate coliform density, N_p (org/100 mL)	25[c]	100	25	100	100
Effluent coliform density, N (org/100 mL)	100[d]	200[d]	100[d]	200[d]	400[b]
Performance goals N' (org/100 mL)	75[e]	100	75	100	300
Required UV performance value $\log (N'/N_0)$	−3.82[f]	−4.3	−4.12	−4.0	−3.22

[a]Values are given in the Design Criteria (Sec. 14-12-1, Step A, 5)
[b]Assumed values.
[c]$N_p = c(\text{TSS})^m = 0.25\,(10)^{2.0} = 25$
[d]Obtained from effluent standards (Table 6-6).
[e]$N' = N - N_p = 100 - 25 = 75$
[f]$\log (N'/N_0) = \log \left(\frac{75}{5 \times 10^5}\right) = -3.82$

The values of nominal exposure time for different flow conditions to achieve the required performance levels are given in Table 14-10.

6. Calculate the number of lamps required.
The number of lamps required is calculated from Eq. (14-43):

$$\text{Number of lamps} = \frac{\dfrac{Q}{q/W_n}}{W/\text{Lamp}} \tag{14-43}$$

The required UV loading q/W_n for different flow conditions and for needed performance are provided in Table 14-10.

TABLE 14-10 The Maximum UV Loading and Number of Lamps Required as Different Flow Conditions to Achieve the Required Performance Levels

	Daily Average	Maximum 7-Day Average	Maximum 30-Day Average	Peak Dry Weather	Design Peak Hour
Required UV performance value, log (N'/N_0)	−3.82[a]	−4.3	−4.12	−4.0	−3.22
Maximum q/W_n (Lpm/W), from performance curve	2.08[b]	1.28[b]	1.58[b]	1.40[c]	1.83[c]
Nominal exposure time t_n, s	5.1	8.3	5.9	8.3	5.8
Flow (Lpm)	26400	36900	29040	55000	79300
Lamp requirement	475	1078	687	1468	1620

[a]For daily average flow the required performance, log $(N'/N_0) = -3.82$ is obtained from Table 14-9.
[b]The maximum UV-loading, q/W_n is obtained from performance curves (Figure 14-20).
[c]The performance curve for maximum 7-d average flow is used.

$$\text{Number of lamps at average daily flow} = \frac{\dfrac{26{,}400\ \text{Lpm}}{2.08\ \text{Lmp/W}}}{\dfrac{1.47\ \text{m.arc}}{\text{Lamp}} \times \dfrac{18.2\ \text{W}}{\text{m.arc}}}$$

$$= 475\ \text{lamps}$$

Similarly, the number of lamps at all other flow conditions are calculated and summarized in Table 14-10.

7. Design the UV disinfection channel.

The maximum number of UV lamps (1620 lamps) are needed for design peak hour flow. There are four disinfection channels and each channel has two banks.

Number of lamps per channel = 405 lamps
Number of lamps per bank = 203
Provide 12 lamps per module.
Number of modules per bank = 16.9 or 17
Total number of lamps per bank = 204
Total number of lamps provided = 1632
Water depth in the channel = 12 lamp × 6 cm spacing/lamp
= 72 cm or 0.72 m
Provide a free board = 0.6 m
Width of the channel = 17 modules × 6 cm spacing/lamp
= 102 cm or 1.02 m

This arrangement gives lamp spacings 6 cm center to center in vertical and horizontal directions, 3 cm from the bottom and sides of the channel to the center of the lamp, and 3 cm from the free water surface to the center of the lamp. Different manufacturers may specify different spacings. Designers must work closely with the manufacturer for proper clearance requirements in the channel. The design details and lamp arrangement is shown in Figure 14-21. This arrangement gives a total of 408 lamps per channel.

8. Location of UV bank in the channel
The placement of influent and effluent structures relative to lamp arrays is critical to achieve uniform flow. Measurement of velocity profiles in full-scale systems show that a minimum of 2 m (6 ft) should be allowed between inlet/outlet structures and the closest lamp array.[27]
9. Select the head loss equation.
The head loss through a UV bank causes a drop in the free water surface in the channel. This may cause serious operational problems in the disinfection process. If the liquid level is set such that the downstream free surface is coincident with the top of the irradiated zone, then some liquid on the upstream end will pass through a region of low intensity. Conversely, if the free surface is set in accordance with the upstream lamps, then the portions of the lamps in the lower end of the bank may not be immersed. With diurnal fluctuations in the flow, some lamps may experience alternate immersion and dryness, which may cause fouling of quartz sleeves and irregular heat distribution that may shorten the life of the lamps. Some designers have used the sloping bottom of the channel to step down the subsequent banks.

The head loss through the UV banks depends on the number and arrangement of the UV lamps, system geometry, and velocity through the channel. Most manufacturers of the UV system have experimentally developed a relationship between head loss and velocity through the net section of UV bank. The functional relationship between the head loss and velocity may be expressed by Eq. (14-44):

$$\Delta h/L = au + b\rho u^2 \qquad (14\text{-}44)$$

where

Δh = head loss, cm
L = length of chamber over which Δh is expressed, cm
u = approach velocity, cm/s
ρ = liquid density, g/cm^3
a, b = empirical constants measured by field test

Under laminar flow conditions, only the first term in the equation is important. Under turbulent conditions, only the second term is important. Experimental results have shown that in a UV system the second term accounts for most of the head loss.

The designers are more familiar with Eq. (7-9) for calculation of head loss through valves, fittings, elbows, contractions, expansions, and so on. Blatchley experimented head loss across UV banks arranged in series in a UV channel. He reported the experimental results as head loss per bank as a function of approach ve-

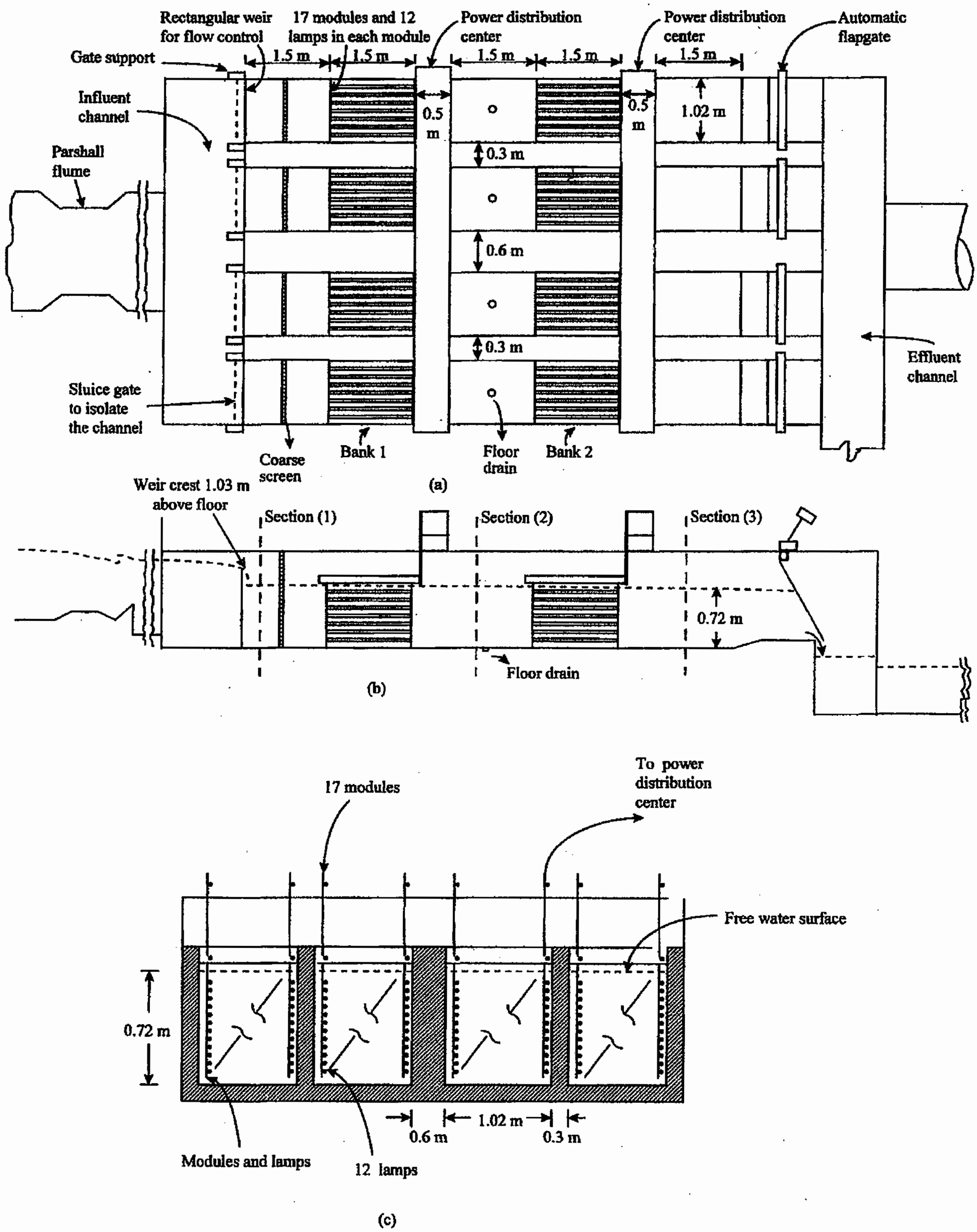


Figure 14-21 Design Details of the UV System: (a) plan, (b) longitudinal section, (c) cross section.

Continued

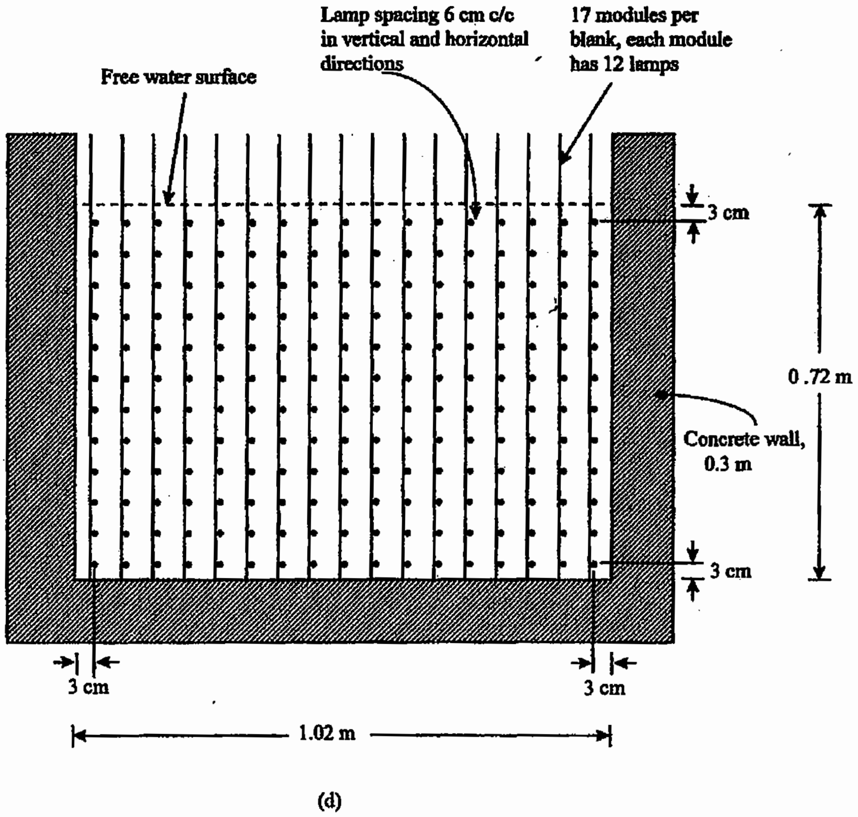


Figure 14-21—cont'd (d) details of modules and lamp arrangement.

locity.[27] From these experimental data the head loss coefficients K for each bank as a function of approach velocity are calculated. These K values are shown in Figure 14-22. These K values shall be used in Eq. (7-9) to determine head loss across each UV bank in series. Eq. (7-9) for UV application is given below:

$$h_L = Kv^2/2g$$

10. Calculate the head loss through the UV system at average daily flow.
The procedure for calculating head loss through the UV system involves the use of the energy equation at downstream and upstream sections of the UV bank. Apply the energy equation at section (3) and (2) as shown in Figure 14-21(b).
   a. Calculate head loss across the downstream bank.

$$\text{Flow per channel}^{e} = 0.11 \text{ m}^3/\text{s}$$

The depth of liquid above the top of UV lamp at the downstream end of the second bank

[e]Average flow = 0.440 m^3/s. Flow per channel = $\frac{0.44}{4}$ m^3/s = 0.11 m^3/s.

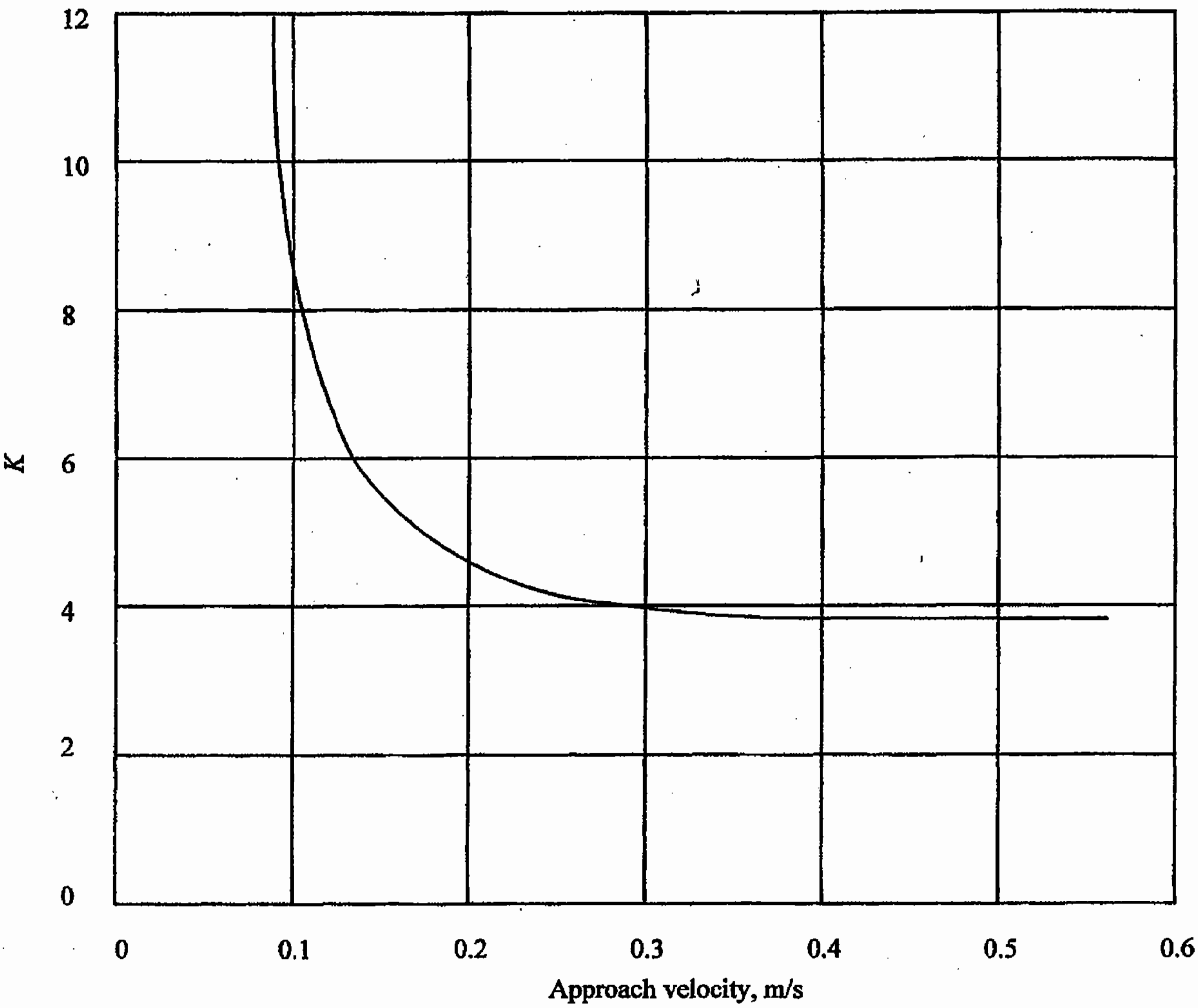


Figure 14-22 The Head Loss Coefficient through Two Banks Arranged in Series.

$$\text{section } (3)^{\text{f}} = \left(\frac{6\text{ cm}}{2} - \frac{2.3\text{ cm}}{2}\right) = 1.85\text{ cm}$$

Channel width = 1.02 m

Liquid depth in the channel at section (3) = 0.720 m

Velocity of flow, v_3 at section (3) $= 0.11\text{ m}^3\text{/s/ }(1.02\text{m} \times 0.72\text{ m}) = 0.150\text{ m/s}$

Applying energy equation between sections (2) and (3) [Figure 14-21(b)]

$$z_2 + d_2 + v_2^2/2g = z_3 + d_3 + v_3^2/2g + h_L$$

$$0 + d_2 + v_2^2/2g = 0 + 0.72\text{ m} + \frac{(0.150\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} + Kv_2^2/2g$$

$$d_2 + (1 - K)\, v_2^2/2g = 0.721\text{ m}$$

[f]The center-to-center spacing of the lamps is 6 cm, and the outer diameter of quartz sleeve is 2.3 cm.

Using the trial-and-error method,

$$\text{Assume } d_2 = 0.725 \text{ m}$$

$$v_2 = \frac{0.11 \text{ m}^3/\text{s}}{0.725 \text{ m} \times 1.02 \text{ m}} = 0.149 \text{ m/s}$$

From Figure 14-21, $K = 4.6$.

Solving for d_2 and v_2 from the above equation,

$$d_2 + (1 - 4.6) \times \frac{(0.149 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2} = 0.721 \text{ m}$$

$$\begin{aligned} d_2 &= 0.725 \text{ m} \\ h_L &= 0.725 \text{ m} - 0.720 \text{ m} \\ &= 0.005 \text{ m or } 0.5 \text{ cm} \end{aligned}$$

b. Calculate head loss across the upstream bank.
Applying the energy equation between sections (1) and (2) [Figure 14-21(b)]

$$z_1 + d_1 + v_1^2/2g = z_2 + d_2 + v_2^2/2g + h_L$$

$$0 + d_1 + v_1^2/2g = 0 + 0.725 \text{ m} + \frac{(0.149 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2} + Kv_1^2/2g$$

$$d_1 + (1 - K)\, v_1^2/2g = 0.726 \text{ m}$$

Using the trial-and-error solution, $d_1 = 0.730$ m, $v_1 = 0.148$ m/s, and $K = 4.61$.

$$\begin{aligned} \text{Head loss across the upstream bank} &= 0.730 \text{ m} - 0.725 \text{ m} \\ &= 0.005 \text{ m or } 0.5 \text{ cm} \\ \text{Total head loss} &= 0.730 \text{ m} - 0.720 \text{ m} \\ &= 0.01 \text{ m or } 1.0 \text{ cm} \end{aligned}$$

c. Calculate the head loss across upstream and downstream banks for different flow conditions. These head losses are summarized in Table 14-11.

11. Determine the head loss through the influent structure.
The influent structure to the UV channel is controlled by a rectangular weir. The length of the weir is the same as the width of the UV channel. A free fall of 0.08 m at average design flow is provided in the UV channel. The head over weir at average and peak design flows are calculated below:

$$H_{avg} = \left[\frac{3}{2} \times \frac{0.11 \text{ m}^3/\text{s}}{0.6 \times 1.02 \text{ m} \times \sqrt{2 \times 9.81 \text{ m/s}^2}}\right]^{2/3} = 0.16$$

$$H_{peak} = \left[\frac{3}{2} \times \frac{0.33 \text{ m}^3/\text{s}}{0.6 \times 1.02 \text{ m} \times \sqrt{2 \times 9.81 \text{ m/s}^2}}\right]^{2/3} = 0.32$$

12. Determine the height of influent weir above the floor of UV channel.

TABLE 14-11 Head Loss across UV Banks at Different Flows

Flow Condition	Flow (m^3/s)	h_L Across for Upstream Bank (cm)	h_L Across Downstream Bank (cm)	Total h_L for Two Banks (cm)
Average daily	0.11	0.5	0.5	1.0
Maximum 7-d average	0.17	0.9	0.9	1.8
Maximum 30-d average	0.12	0.5	0.5	1.0
Peak dry weather	0.23	1.4	1.6	3.0
Peak wet weather	0.33	3.0	3.0	6.0

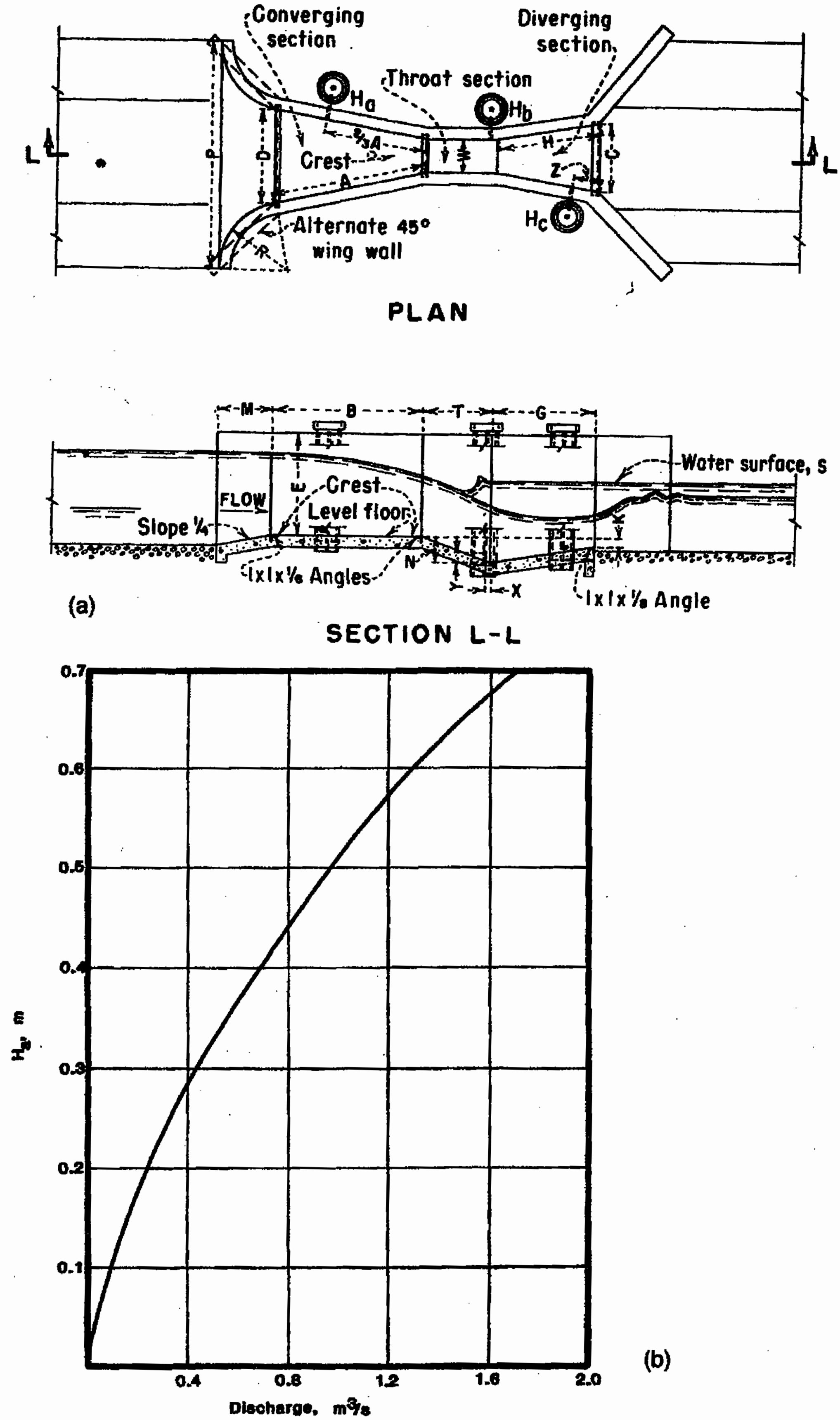
$$\begin{aligned}\text{Height of influent weir} &= (\text{water depth downstream of second UV bank at average design flow}) + (\text{head loss in both UV banks}) + (\text{free fall at the weir}) \\ &= 0.72 \text{ m} + 0.01 \text{ m} + 0.08 \text{ m} \\ &= 0.81 \text{ m}\end{aligned}$$

13. Determine the head loss in the influent channel.
    The head losses in the influent channel will be encountered because of excessive turbulence, friction, flow distribution, and change in direction. Therefore, a total of 0.37-m head loss is assumed in the influent channel.
14. Determine the head loss at the effluent structure.
    The effluent structure has a raised floor on which an automatic flap gate level control structure is installed. The flap-type gate with counterweights keeps the liquid level near the effluent structure at the exact depth. A free fall at the downstream of the gate is necessary for proper operation of the flap gate. As a result, the entire head because of channel depth is lost at the gate. In this design, the channel floor is raised 0.3 m to conserve some head. Therefore, a total of 0.42 m (0.72 m − 0.30 m) head will be lost at the gate. Provide a free fall of 0.20 m downstream of the gate.

Step B: Parshall Flume Design. A standard Parshall flume is designed as an integral part of the UV disinfection system. The flow measured at the Parshall flume will be divided into the operating UV disinfection channels equally. The design details of the Parshall flume are shown in Figure 14-23.

1. Select the dimensions of the channel upstream of the Parshall flume.

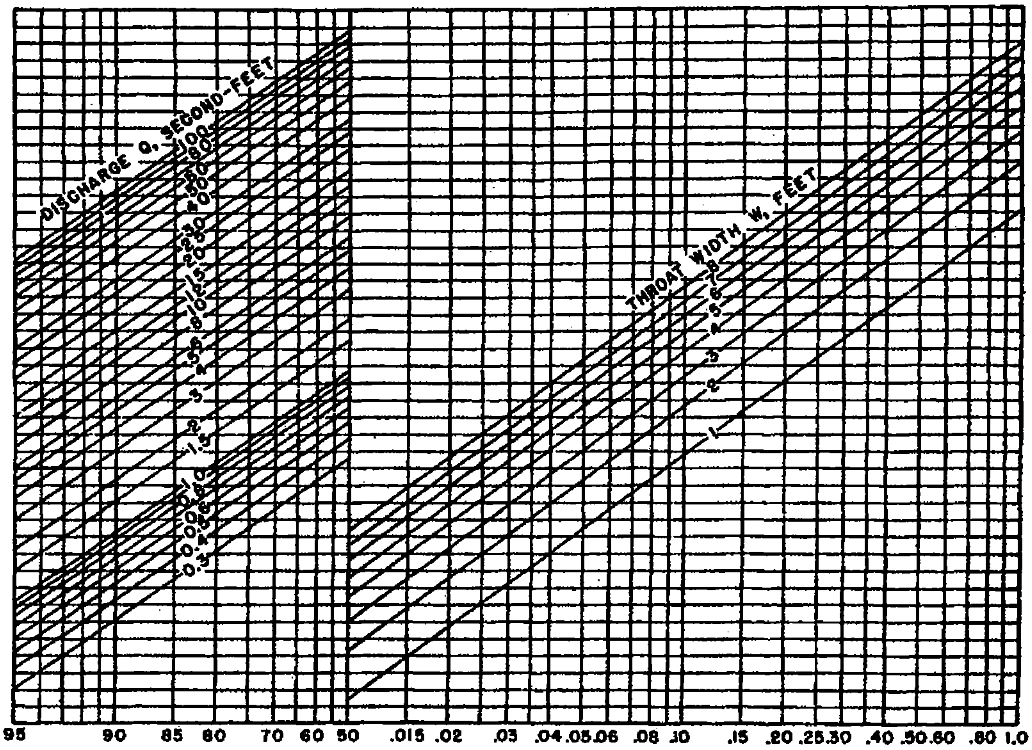
$$\begin{aligned}\text{Channel section} &= \text{rectangular} \\ \text{Width of the channel} &= 2.0 \text{ m} \\ \text{Depth of flow at peak design flow, } y_1 &= 0.8 \text{ m}\end{aligned}$$



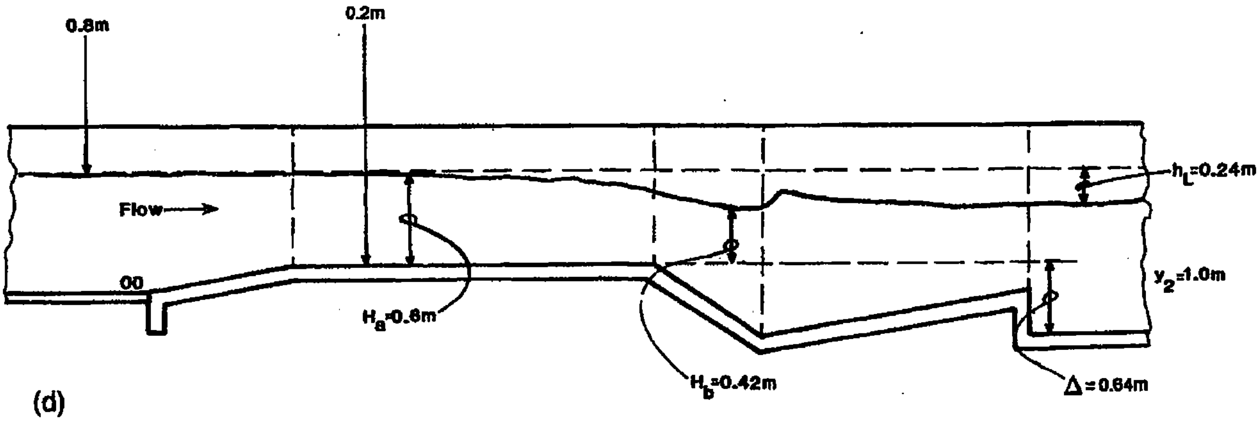
Standard dimensions of 1.22 m (4ft) Parshall flume

W	1.22m
A	1.83m
B	1.79m
C	1.52m
D	1.94m
E	0.91m
T	0.61m
G	0.91m
H	-
K	0.08m
M	0.46m
N	0.23m
P	2.66m
R	0.61m
X	0.05m
Y	0.08m
Z	-
Capacity, min.	36.81l/s
Capacity, max.	1922.71l/s

Figure 14-23 Design Details of Parshall Flume Used for Flow Measurement in the Design Example: (a) standard dimensions of Parshall flume (from Refs. 28 and 29); (b) calibration curve of 1.22 m (4 ft) Parshall flume.



(c)



(d)

Figure 14-23—cont'd (c) head loss through Parshall flume, second-ft = ft³/s. (from Refs. 28 and 29); and (d) water surface profile through Parshall flume at peak design flow.

The slope of the channel is obtained from the Mannings Equation [Eq. (7-1)].

$$\text{Cross-sectional area} = 2.0 \text{ m} \times 0.8 \text{ m} = 1.60 \text{ m}^2$$

$$R = \frac{\text{area}}{\text{perimeter}} = \frac{1.60 \text{ m}^2}{\text{width} + 2 \text{ depth}}$$

$$= \frac{1.60 \text{ m}^2}{2.0 \text{ m} + 2 \times 0.8 \text{ m}}$$

$$= 0.44 \text{ m}$$

$$1.321 \text{ m}^3/\text{s} = 1.60 \text{ m}^2 \times \frac{1}{0.013} \times (0.44 \text{ m})^{2/3} S^{1/2}$$

$$S = 0.000344$$

2. Select the dimensions of the rectangular channel downstream of the Parshall flume.

Width	= 2 m
Depth of flow at peak design flow y_2	= 1 m

The slope of the channel is obtained from Eq. (7-1).

$$\text{Cross-sectional area} = 2.0 \text{ m} \times 1 \text{ m} = 2.0 \text{ m}^2$$

$$R = \frac{2.0 \text{ m}^2}{2 \text{ m} + 2 \times 1 \text{ m}} = 0.5 \text{ m}$$

$$1.321 \text{ m}^3/\text{s} = 2.0 \text{ m}^2 \times \frac{1}{0.013} \times (0.5 \text{ m})^{2/3} \times S^{1/2}$$

$$S = 0.000186$$

3. Select the dimensions of the Parshall flume.

Throat width	= 1.22 m (4 ft)
Submergence at peak design flow	= 70 percent

The dimensions of various components of the Parshall flume are given in Figure 14-23(a).

4. Select the discharge equation for the Parshall flume.
At submergence below 70 percent, the flow through a 1.22-m Parshall flume is essentially the same as for free flow conditions.[28,29] Free flow discharge for a Parshall flume is given by Eq. (14-45):

$$Q = 4WH_a^{1.522W^{0.026}} \tag{14-45}$$

where

Q = free flow, cfs
W = throat width, ft
H_a = depth of water at upstream gauging point, ft (Figure 14-23)

5. Compute H_a and H_b (depth of water at upstream and downstream gauging points).
H_a at peak design flow of 1.321 m^3/s (46.7 cfs) is calculated from Eq. (14-45):

$$46.7 \text{ cfs} = 4 \times 4 \text{ ft} \times H_a^{1.522(4 \text{ ft})^{0.026}}$$

Solving this equation,

$$H_a = 1.97 \text{ ft } (0.6 \text{ m})$$

$$H_b \text{ at 70 percent submergence} = 0.7 \times 1.97 \text{ ft} = 1.38 \text{ ft } (0.42 \text{ m})$$

6. Compute head loss H_L through Parshall flume at peak design flow.
   The head loss is computed from Figure 14-23(c). At peak design flow of 46.7 cfs (1.321 m^3/s) and 70 percent submergence, the head loss is 0.77 ft (0.24 m).
7. Compute the downstream channel bottom from the flume crest Δ.
   The water surface in the flume at the H_b gauge is essentially level with the surface in the downstream channel. Therefore, $\Delta = y_2 + H_L - H_b = 1.00 \text{ m} + 0.24 \text{ m} - 0.60 \text{ m} = 0.64 \text{ m}$.
8. Prepare water surface through the Parshall flume at peak design flow.
   The water surface profile is shown in Figure 14-23(d).
9. Prepare the calibration curve for the Parshall flume.
   The calibration curve for the Parshall flume is prepared from Eq. (14-45). At lower flows the water depth in the contact chamber will be reduced, and therefore, the submergence at the downstream side of the Parshall flume will increase. The calibration curve is shown in Figure 14-23(b).

Step C: Head Losses and Hydraulic Profile. The head loss calculations through the UV channel and Parshall flume were provided earlier. Following is the summary of head losses that are encountered through each unit at peak design flow when all four UV disinfection chambers are in service. The hydraulic profile for such a condition is illustrated in Figure 14-24.

Head loss through Parshall flume	= 0.24 m
Head loss in the influent channel due to free fall friction, turbulence, entrance, etc. (assume)	= 0.37 m
Differential head between upstream and downstream of influent weir of UV disinfection channel	= 0.35 m
Head loss through UV disinfection channel	= 0.06 m
Head loss through level control gate	= 0.42 m
Free fall downstream of gate	= 0.20 m
Total	= 1.64 m

A total head loss of 1.64 m at the Parshall flume and UV disinfection facility is disproportionately large; however, such head losses are not uncommon. In situations where adequate head is not available, free fall allowances could be reduced, and further raising the channel floor at the level control gate may be considered.

Step D: Design Details. The design details of the UV disinfection facility and Parshall flume are provided in Figures 14-21, and 14-23.

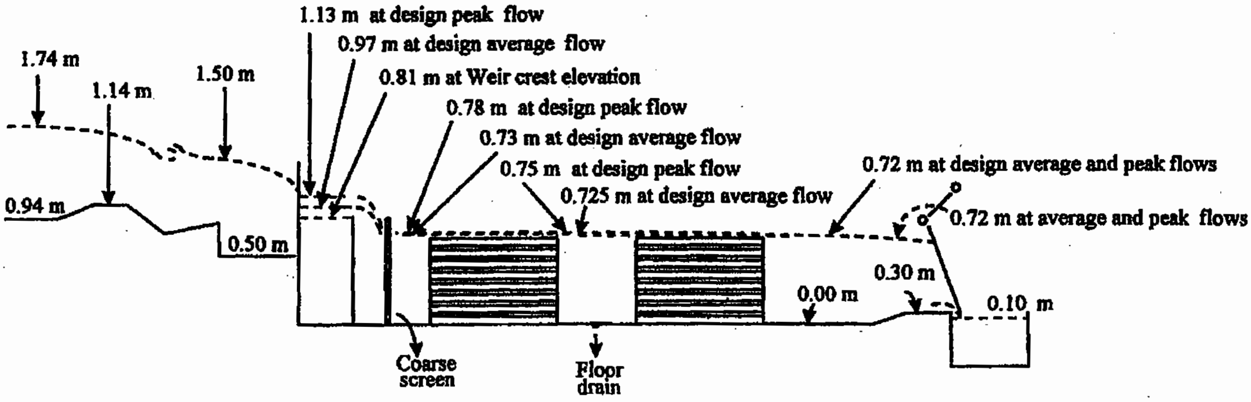


Figure 14-24 Hydraulic Profile Through the Parshall Flume and UV Disinfection Channel at Peak Hour Wet Weather Flow.

14-13 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT UV DISINFECTION FACILITY

Operation and maintenance of a UV disinfection system requires well-trained operators familiar with the UV disinfection system and personal safety. It is also important that the operators know various components of the UV system. Therefore, a summary of major components of a UV system are presented first, followed by operation and maintenance and troubleshooting sections. The hydraulic profile through the UV disinfection channel at average and peak hour flows are shown in Figure 14-23.

14-13-1 Major Components

The major components of a UV system include disinfection channel, UV banks, and electrical supply system. The effluent flows by gravity into four open channels for disinfection. Each channel consists of two banks with 17 modules per bank and 12 lamps per module. Each bank contains 204 lamps. In all, eight banks contain a total of 1632 lamps. Figure 14-21 shows the UV system components.

The UV system disinfects water as the effluent flows through the banks. A power cord leads from each UV module to its power control (PC) module, located in the control panel. PC modules supply and monitor electrical power to the UV lamps. The UV modules and PC modules can be removed separately for maintenance. Downstream from the UV modules is the automatic level controller (a flap-type gate with counterweights), which keeps the effluent at the correct depth. The banks of UV modules can be operated singularly or in series. The number of channels can be varied to meet the disinfection requirements of a given flow rate and effluent quality.

The stainless steel UV modules hold the ultraviolet lamps. The lamps are enclosed individually in quartz sleeves and submerged in the effluent channel. UV modules are held in the support frame with the module handle above the water. Each UV module is connected to a power cable that leads through the wireway to a PC module in the control panel. The lamps are 147 cm long, and there are 12 lamps in each module.

The PC Modules are located in the control panels. Each PC module contains the ballasts for the UV lamps and a printed circuit board that monitors current to the lamps.

Attached to the circuit board and visible on the front of the PC module are light emitting diodes (LEDs). They indicate the status of each lamp and are arranged in the same order as the lamps they monitor. One additional LED monitors the power to the PC module. The number and type of ballast, type of circuit board, and power requirements of the PC module depend on the number and type of lamps in the UV module.

A total of eight control panels house the PC modules. A power cable leads from each PC module to the UV module in the effluent channel. The monitoring LEDs are visible through a window in the lockable front door. The PC modules connect to the main power supply through ground fault circuit interrupters. Lamp ballasts are kept at operating temperature by cooling fans located near the ballasts. The service entrance for the control panel is located at the main disconnect.

The automatic level controller is a device that keeps the effluent at the proper depth. It is positioned in the effluent channel downstream from UV modules. It consists of a baffle balanced by weights. The baffle automatically swings partially open in proportion to the flow and releases the excess discharge. This keeps the effluent level constant, regardless of flow rate. The position of the balance weights can be adjusted to control the liquid level in the channel. The automatic level controller can be opened completely and rested on a stop chain to lower the depth in the channel to a minimum level. This permits maintenance of the UV modules while they are in the channel.

The UV sensor probe measures the intensity of UV light at 254 nm and converts this measurement into a UV dose reading at the meter. The meter is also equipped with a UV dose remote alarm or running light. A drop in meter reading can be caused either by reduced light output or by a change in the effluent quality. Either of these conditions results in a lower bacteria kill. The meter reading is a good indicator of the system's disinfecting power.

The UV disinfection system includes the following equipment items:

Item Description	Quantity
UV modules	136
PC modules	136
Stainless steel control panel	8
Stainless steel UV rack assembly	8
Automatic level controller	4
Cable wireway tray stainless steel	4
Elapsed time meters	8
UV module cleaning rack, cleaning basins and jib crane	2

14-13-2 Routine Operation and Maintenance

Routine operation consists of operating the UV disinfection channels in parallel. During a peak design wet weather flow of 1.32 m^3/s, all four channels and eight banks must be in operation. During the average flow condition, only two channels or one bank in four channels will provide the required degree of disinfection. One UV bank may be removed for cleaning and maintenance any time except during peak wet weather flow condition.

The following operation and maintenance steps are necessary to keep a trouble-free operation:

1. Start up: To start up a UV channel, turn on the lamps for that channel. Then start the water flow through the channel by operating the gate over the weir. Verify that the drain valves are closed. Lamps require approximately 20 minutes to warm up.
2. Shut down: To shut down a channel, close the slide gate to that channel and open the drain valve. After the channel is drained, turn off the UV lamps.
3. Operation: Check daily and record the meter reading of UV dose. Also check the LED display on the UV control cabinet. If the lamps or ballasts are burned out, replace them.
4. Dirty quartz sleeves and lamps can reduce disinfection efficiency. Turn off the lamp module and electric supply before removing the module or bank. Do not overtighten the sensor clamp on the quartz sleeves because this may break the sleeve.
5. Do not operate near UV lamps without proper eye, face, and body protection. Monitor the water surface and effluent gate to keep the top of the lamp near the gate completely submerged and to keep the water surface about 2 cm above the lamp.
6. Clean the UV module as needed in accordance with the manufacturer's instructions.
7. Regularly monitor the fecal coliform in the influent and effluent of the UV disinfection channel. Also record the number of banks and modules on-line and the flow rate. This information is used to determine the actual dose being applied. Additionally, the information is useful over time, to identify trends, predict cleaning intervals, and provide design information for future expansions.
8. Measure TSS and turbidity in the effluent. Both TSS and turbidity affect the performance of the UV system. If TSS or turbidity is higher than normal, all standby modules should be utilized to maintain the required degree of disinfection.
9. Insects are generally attracted by the UV light. Vacuum the area daily around the UV disinfection channels if an insect problem occurs.
10. Replace lamps after 8700 hours of actual operation.

14-13-3 Troubleshooting

The following troubleshooting guide should be utilized:

1. If power LED is on but two or more LEDs are out, the probable cause may be
   a. Malfunctioning of one or more lamps: A single faulty lamp can cause more than one LED to go out. Replace the appropriate lamps one at a time.
   b. If the LED is still off, check the connections in the PC module. The problem may be caused by a malfunctioning ballast. Identify the faulty ballast by the lamp number written on it and replace the ballast. If necessary, replace the circuit board of the entire module.
2. If all LEDs are on but the PC module is out, make sure that the main switch is on and the PC module and related UV module are properly connected. Press the RESET switch located on the ground fault receptacle. If it does not solve the problem, plug the PC module into a different ground fault receptacle and press the RESET switch. The following troubleshooting guide may apply:

   a. If the receptacle switches off immediately, the probable cause may be an electrical leak (short) through ground connections. Look for and repair any cracked sleeves, loose connections in the UV module, and faults in the power cable.
   b. If the receptacle stays on but the LED is still out, the probable cause may be a fault in the PC module. Check connections inside the receptacle. Replace the circuit board or the entire PC module if necessary.
   c. If LEDs come on but the system is not operable, check connections to the original ground fault receptacle. Replace if necessary.
3. If the sensor indicates that the UV dose is below the desired level, the probable causes of the problems and solutions are as follows:
   a. A sudden drop in measured dose may be caused by dirt or debris on the sensor window. Clean if necessary.
   b. A low measured dose reading may be caused by a faulty lamp being monitored. Try a different lamp.
   c. Decrease in effluent quality may cause low readings in other sensors. If readings of other sensors are acceptable, the sensor in question may be faulty.
   d. A gradual drop in sensor reading may be caused by a coating on the sleeve. Clean the sleeve.
   e. A low measured dose after elapse of a reasonable amount of time since installation of the lamp may be caused by normal depreciation of the lamp output. If sufficient time has not been elapsed, the lamp is faulty. Replace the lamp in both cases. Follow the instructions for lamp replacement.

14-14 SPECIFICATIONS

Specifications on a UV facility are briefly presented in this section. The purpose of these specifications is to describe many components that could not be fully covered in the design. The design engineer should use these specifications only as a guide. Detailed specifications for each unit or component should be prepared in consultation with the equipment manufacturers.

14-14-1 General

The contractor shall provide all labor, materials, tools, and equipment required to furnish and install a complete, tested, and ready-for-continuous-service ultraviolet disinfection system in an open channel as shown on the design plans.

14-14-2 Channels and Housing

The contractor shall provide four UV disinfection channels complete with foundations, concrete channels, handrails and stairs, piping, channel drains and valves, solid metal grating over the UV banks to confine light exposure within the banks (to minimize the insect attraction), walkway grating, channel isolation gates, lifting hoist, and main electrical power systems.

14-14-3 UV Equipment

The UV equipment shall include the following components:

1. Eight UV banks, two banks in series in each channel, with uniform array in horizontal rows, and flow parallel to lamps, 17 modules in each bank, and 12 lamps in each module
2. Power distribution center, control panel, interconnecting cables to modules and alarms
3. Lamp identification and monitoring system
4. UV intensity monitoring systems
5. Module cleaning station liner
6. Energy conservation flow pacing system
7. Automatic level control
8. UV eye shields and safety equipment
9. Spare parts
10. Startup, testing, and personnel training

14-14-4 Quality Assurance

1. The UV disinfection system shall be capable of disinfecting the specified flows based on the minimum effluent quality.
2. Provide a 5-year history of successful installations. Evidence of previous performance shall include the operational data documented by a laboratory independent of the manufacturer.
3. Provide a reference list of a minimum of five wastewater treatment plants in which similar UV disinfection systems have been installed. The list shall include operator names and current telephone numbers.

14-14-5 Performance Requirements

1. The lamp output must be at least 65 percent of initial levels after 1 year of operation and with no fouling on the lamp sleeves.
2. The head loss through two series banks at peak design flow shall not exceed 6 cm, this being confirmed by measurements in the field after startup.
3. The system shall be designed for energy conservation and partial system shutdown by automatic flow pacing of modules in a manner that minimizes any loss of disinfection capacity. The system specified shall be able to continue providing disinfection of at least 90 percent of peak design flow when the largest single bank of lamps are out of service for cleaning or while replacing UV lamps, quartz sleeves, ballasts, or electronic circuit boards.

14-14-6 UV Module

1. Each horizontal UV module shall consist of 12 UV lamps, each enclosed in an individual quartz sleeve. The sleeve shall be sealed with a UV-resistant double seal with

stainless steel backup. The closed end of the quartz sleeve shall be held in place by means of a UV-resistant retaining cup. The quartz sleeve shall not come into contact with any stainless steel in the frame. The quartz sleeve shall provide a minimum of 90 percent transmission at a 254-nm wavelength and shall have a nominal wall thickness of 1.0 mm.

2. Each module shall be labeled with the module number permanently attached to the module and shall be constantly visible without requiring the grating to be removed. The module enclosure shall be suitable for continuous outdoor operation.
3. Each module shall include an integral air scrub system during the cleaning process. Provision for compressed air shall be provided.
4. Wiring exposed to UV light shall be Teflon® coated.
5. Each UV lamp module shall include a safety interlock that shall automatically disconnect the power to the UV lamps when the module enclosure is opened for service.
6. Each module shall be protected by a panel-mounted thermal magnetic circuit breaker. The rating of each individual circuit breaker shall not exceed 30 amps. The circuit breakers shall be located adjacent to the UV channel in a proper load center.
7. Ultraviolet lamps and individual electronic lamp controllers shall be arranged so that each may be easily and safely tested in place.

14-14-7 Ultraviolet Lamps

Lamps shall meet the following requirements:

1. The lamp shall be low-pressure, mercury vapor UV lamps. Each lamp shall produce UV light with 90 percent of the UV emission at a 253.7-nm wavelength.
2. The UV lamp output shall not be less than 26.7 UV watts. Maximum power consumption per lamp shall be 70 watts. The UV lamp intensity at a distance of 1 m in air shall be 190 microwatts/cm^2.
3. Minimum UV lamp arc length shall be 147 cm.
4. Lamps shall be rated not to produce ozone.
5. The lamp base shall be of a durable construction resistant to UV. The lamp design shall prevent electrical arcing between connections in moist conditions.

14-14-8 Instrumentation and Controls

The power distribution center (PDC) shall be of type NEMA 3R and wall-mounted for indoor/outdoor installation. It will be provided with necessary LED displays, alarms, and controls.

14-14-9 Automatic Flap Gate Level Controller

An automatic level control shall be placed at the discharge end of the channels to ensure that the UV lamps are properly submerged, regardless of the plant flows. The level of the water shall be maintained at an appropriate level at all design flow conditions by simple adjustment to a counterbalance tank. The level control gates shall include a hinged baffle with a counterbalance.

14-15 PROBLEMS AND DISCUSSION TOPICS

14-1 Calculate the dimensions of a chlorine contact basin that has a four-pass-around-the-end baffled arrangement. The contact time at 0.5 m^3/s flow is 20 min. The clear width of each pass and the openings at the baffles are 2.0 m, and depth is 3 m. The total length of the basin is the centerline around the baffle.

14-2 A Parshall flume is designed for measuring the flow at a wastewater treatment facility. The expected peak design and minimum initial flows are 1.2 m^3/s and 0.3 m^3/s, respectively. The throat is 1.22 m. Both the influent and effluent channels to the Parshall flume are 3 m wide and have a slope of 0.000054. The Parshall flume has a maximum submergence of 70 percent. Determine the head loss at peak and minimum flows. Draw the hydraulic profiles through the flume.

14-3 Calculate the number of chlorinators, number of chlorine containers attached to the heads, and the number of containers required for a 3-week chlorine supply. Use the following data:

Max. and average flow = 2.00 and 0.67 m^3/s
Maximum chlorine feed rate = 9 mg/L

The chlorinators are 450 kg/d capacity.

Gaseous chlorine withdrawal rate per container = 180 kg/d

Use a 1-ton container size.

14-4 Hydraulic profile at peak design flow was developed in the Design Example. Draw the hydraulic profile through the Parshall flume and UV disinfection facility at average design flow when all four chambers are in operation. The average design flow is 0.44 m^3/s.

14-5 The residual chlorine and chlorination data are given below. Plot the chlorination curve. Obtain the break point chlorination dosage. What will be the initial demand and total chlorine demand (kg/d) to give a free chlorine residual of 1.2 mg/L? The flow is 1800 L/s.

Chlorine Dosage (mg/L)	Chlorine Residual (mg/L)
1	0
2	0.8
3	1.4
4	1.0
5	1.1
6	2.0
7	3.0

14-6 The reduction of organism in a chlorination process is expressed by Eq. (2) in Table 14-1. Using the midpoint chlorine dosage of 3 mg/L and 130 coliform organisms per 100 mL remaining in the secondary effluent after 30 min contact time, calculate k. $N_0 = 10^6$ coliform per 100 mL. Also calculate the number of coliform organisms remaining after 20 min of contact time.

14-7 Calculate the volume of a chlorine contact basin and the quantity of chlorine needed in kilograms per day. The average design flow is 0.2 m^3/s, contact time is 18 min, total chlorine demand is 17 mg/L, and chlorine residual maintained is 1.5 mg/L.

14-8 Define the following terms: free chlorine residual, combined chlorine residual, total chlorine residual, pasteurization, and disinfection.

14-9 The laboratory data for a chlorination study are given below. Using Eq. (2) in Table 14-1, calculate the contact period to reduce the coliform count from 10^5 org/100 mL in the influent to 50 org/100 mL in the effluent. The chlorine dosage is the same as that used in the laboratory study.

Contact Time at Chlorine Dosage of 10 mg/L, min	Number of Coliform Organisms Remaining per 100 mL
0	10^5
5	5000
10	250
15	12

14-10 Write the advantages and disadvantages of ozonation over chlorination.

14-11 What is the significance of breakpoint chlorination in effluent disinfection? Compare the chlorine dosage requirements to reach the breakpoint chlorination for two effluent samples: one has high ammonia nitrogen, and the other is well nitrified.

14-12 A UV disinfection facility is designed to reduce the fecal coliform number. The expected number of fecal coliform and TSS in the influent are 10^5 org/100 mL and 15 mg/L, respectively. The adjusted $I_{avg} = 9000\ \mu W/cm^2$. The UV system has two banks; each bank uses 150-cm-long lamps. The flow velocity through the channel is 20 cm/s. The dispersion coefficient is 325 cm^2/s. The experimental constants $a = 1.45 \times 10^{-5}$, $b = 1.2$, $c_1 = 0.27$, and $m_1 = 2.1$. Calculate the fecal coliform count after disinfection.

14-13 Draw the performance curves for peak dry weather and design hour flows in the Design Example similar to the curves for the flows given in Figure 14-20. Calculate the number of lamps needed for both conditions.

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15 Effluent Disposal

15-1 INTRODUCTION

The proper disposal of treatment plant effluent is an essential part of planning and designing wastewater treatment facilities. In most cases the ultimate effluent disposal or reuse requirements dictate the selection of the wastewater treatment process train. In Chapter 6 several methods of ultimate disposal and reuse of secondary effluents were compared. These methods included (1) natural evaporation, (2) urban reuse, (3) industrial reuse, (4) agricultural reuse, (5) recreational and aesthetic reuse, (6) habitat restoration/enhancement, (7) groundwater recharge, (8) augmentation of potable water supplies, and (9) discharge into natural waters.[1,2] In this chapter, each of these methods of effluent disposal is briefly discussed. Also effluent disposal into natural waters and the design procedure for river outfalls are discussed in considerable detail.

15-2 METHODS OF EFFLUENT REUSE AND DISPOSAL

Effluent reuse is increasing. Even effluent discharge into natural waters is being considered as an indirect reuse of effluent for municipal water supply. The overriding consideration, however, for developing a reuse system is the quality of reclaimed water for its intended use. Following is a general discussion on various types of effluent reuse and disposal methods partly adapted from Ref. 1.

15-2-1 Natural Evaporation

The process involves large impoundments with no discharge. The amount of evaporation from the water surface depends on temperature, wind velocity, and humidity. There are substantial variations in the average daily evaporation rate from month to month and year to year. However, depending on the climatic conditions, large impoundments may be necessary if precipitation exceeds evaporation over several months. Therefore, consideration must be given to net evaporation, storage requirements, percolation into ground, and groundwater pollution. Natural evaporation has been used for industrial wastewater in arid climates, particularly for disposal of brines. This is particularly beneficial where recovery of residues is desirable. Impervious liners are necessary in many instances.

15-2-2 Urban Reuse

Urban reuse systems provide reclaimed water for many nonpotable purposes within an urban area. These uses include (1) irrigation of public parks and recreation centers, golf courses, athletic fields, school yards and playgrounds, highway medians and shoulders, and landscaped areas surrounding public buildings and facilities;[3] (2) irrigation of the landscaped areas of single-family and multifamily residences, general washdown, and other maintenance activities; (3) irrigation of landscaped areas surrounding commercial, office, and industrial developments; (4) irrigation of golf courses; (5) commercial uses, such as vehicle washing facilities, window washing, and street washing; (6) ornamental landscape uses and decorative water features, such as fountains, pools, and waterfalls; (7) dust control and concrete production on construction projects; (8) fire protection; and (9) toilet and urinal flushing in commercial and industrial buildings.[1]

The large consumption of urban reuse occurs in highway medians, parks, playgrounds, athletic fields, golf courses, and recreational facilities; major water-using industries or industrial complexes; and a comprehensive combination of residential, industrial, and commercial properties through dual distribution systems. In dual distribution systems, the reclaimed water is delivered to the customers by a parallel network of distribution mains separated from the community potable water distribution system.[1,2]

In the design of an urban reclaimed water system, the most important considerations are the reliability of service and protection of public health. Treatment to meet appropriate water quality and quantity requirements, and system reliability are very important. The following safeguards must be considered during the design of any urban reuse system: (1) assurance that the reclaimed water delivered to the customer meets the water quality requirements for the intended uses, (2) prevention of improper system operation, (3) prevention of cross-connections with potable water lines, and (4) prevention of improper use of nonpotable water.

To avoid cross-connections, all equipment associated with reclaimed water systems must be clearly marked. National color standards have not been established, but the accepted practice by manufacturers and many cities is to use purple.[4] In general, the urban water reuse systems have two major components: (1) water reclamation facilities for reclaimed water production and (2) a reclaimed water distribution system, including operational storage and high-service pumping facilities.

15-2-3 Industrial Reuse

Industrial reuse represents an enormous potential market for reclaimed water in the United States and other developed countries. The reclaimed water is ideal for industries where processes do not require water of potable quality. Since industries are often located near populated areas, the effluent from municipal wastewater treatment facilities is an available resource. Currently, the number of industrial uses of treated wastewater effluent is increasing rapidly.

Reclaimed water for industrial reuse may be derived from in-plant recycling of industrial wastewaters and/or municipal water reclamation facilities.[1,2] Recycling within an industrial plant is usually an integral part of the industrial process and must be developed on a case-by-case basis. Most industries treat and recycle their own wastewater either to conserve water or to meet or avoid stringent regulatory standards for effluent discharges. Industrial uses for reclaimed water include (1) once-through or evaporative cooling water; (2) boiler-feed water, (3) process water, and (4) irrigation and maintenance of ground and landscape around the industrial plant.[5-7]

The most frequent water quality problems in cooling water systems are scaling, corrosion, biological growth, fouling, and foaming. The boiler-feed make-up water requires extensive treatment to reduce hardness or even demineralization. The industrial process water quality is dependent upon the manufacturing process and must comply with the strict water quality requirements. The overall deciding factors for effluent reuse by industry include (1) availability of natural water, (2) quality and quantity of effluent and the cost of processing, (3) pumping or transport cost of effluent, and (4) industrial process water that does not involve public health considerations.

15-2-4 Agricultural Irrigation

The use of municipal effluents for agricultural irrigation is an acceptable practice in many parts of the United States. Irrigation has been practiced primarily as a substitute for scarce natural waters or sparse rainfall in arid areas. Health regulations may govern the types of crops that are irrigated with effluents.[8,9]

Agricultural irrigation presents a very viable alternative for effluent reuse. In California and Florida, agricultural irrigation accounts for approximately 63 and 34 percent, respectively, of the total volume of reclaimed water used within the states. The major design consideration for agricultural irrigation systems are demand, quality of reclaimed water, reliability, and costs. The demand for agricultural irrigation varies from month to month throughout the year. Major factors are climatic considerations (such as temperature, precipitation, evapotranspiration, etc.), crop type and stage of plant growth, storage requirements, and method of irrigation being applied. The quality is important because there are some constituents in reclaimed water that have a special significance in agricultural irrigation: elevated total dissolved solids (TDS) levels, toxic chemicals because of industrial discharges, and saltwater (chloride) infiltration into the sewer system in coastal areas.[10]

In addition to supply, demand, and quality issues, there are many other factors that are equally important. Availability of suitable farm land and pumping costs for delivering effluent to cultivated acreage where demand may exist are perhaps the main factors for selection of this method. Use of reclaimed water may require modification in current irrigation practice. Users may be reluctant to abandon the existing facilities for the opportunity to use reclaimed water. These factors, along with system reliability, site use control (buffer zone), groundwater monitoring, runoff control, incentives to change from an established source of irrigation to effluent reuse, and irrigation equipment, should all be evaluated during the planning stages of the project.

15-2-5 Recreational and Aesthetic Reuse

Reclaimed water is extensively used for recreational and environmental purposes. Common uses include maintenance of landscapes and aesthetic impoundments; water-based recreational lakes for swimming, fishing, and boating; ornamental fountains; snow making; and rearing of freshwater sport fish. The extent of reuse projects in this category will depend upon water demand and availability of reclaimed water of desired quality.[1,2]

Recreational and aesthetic impoundments may serve the needs of aesthetics, noncontact uses, boating and fishing, and swimming. The required level of treatment, however, will vary with the intended use of water. As the potential for human contact increases, the required treatment levels will also increase.[11,12] For nonrestricted recreational use, treatment of secondary effluent is to be followed by coagulation, filtration, and disinfection to achieve a total coliform count of less than 3 per 100 mL.[1] The primary purpose of the coagulation step is to reduce TSS, precipitate phosphorus, improve appearance, and improve efficiency of virus removal by disinfection.[13,14]

Reclaimed water impoundments can be easily developed in urban areas. For example, landscaping plans for golf courses and residential developments commonly integrate water traps or ponds, which are also used as storage facilities for irrigation water. Examples of such development projects may be found in many states.[1]

Commercial fish production in reclaimed water impoundments is widely used in several countries. Large-scale fish production with reclaimed water is currently being investigated in the United States and has the potential for providing a significant future reuse of reclaimed water. The reclaimed water quality aspects should be thoroughly assessed for possible bioaccumulation of toxic contaminants through the food chain.[1,2]

15-2-6 Habitat Restoration and Enhancement

Over the past 200 years, approximately 50 percent of the wetlands in the United States have been destroyed for such diverse uses as agriculture, mining, forestry, and urbanization. Wetlands serve many beneficial environmental functions. Among them are flood attenuation, habitat for wildlife and waterfowl, productivity to support the food chain, aquifer recharge, and water quality enhancement. The application of reclaimed water to wetlands is beneficial because wetlands provide maximum nutrient assimilation, have aesthetic and environmental improvement, and enhance the water quality of reclaimed water by cost-effective treatment through natural systems. In addition to these, wetlands maximize wildlife habitats. The basic design considerations include selection of appropriate plant and animal species, salinity evaluation, and a balance in hydraulic and constituent loadings with the needs of the ecosystem.[15,16]

A number of states provide regulations that specifically address the use of reclaimed water in wetland systems. Natural wetlands are protected under the NPDES Permits and Water Quality Standards. Effluent quality-equivalent of a secondary level or higher may be permitted. On the other hand, the constructed wetlands are built and operated for treatment only.

Aquaculture, or the production of aquatic organisms (both flora and fauna), has been practiced for centuries, primarily for the production of food, fiber, and fertilizer. Water hyacinth, duckweed, seaweed, alligator weed, midge larvae, *Daphnia,* and a host of other organisms have been cultured. Water hyacinths are large, fast-growing, floating aquatic plants that thrive on raw, as well as on partially treated, wastewaters.[11,15,17] Wastewater treatment by water hyacinths for BOD, solids, nutrients, and heavy metals removal has been considered. Other potential applications of water hyacinth culture include upgrading of lagoons, rennovation of small lakes and reservoirs, pretreatment of surface waters for domestic water supply, stormwater treatment, and recycling of fish culture water. Water hyacinths harvested from these systems have been investigated as a fertilizer/soil conditioner after composting, animal feed, and a source of methane when anaerobically digested. Heavy metals generally accumulate in water hyacinths and limit their suitability as a fertilizer or feed material. Lagoons are used for aquaculture, although artificial and natural wetlands are also being considered. The greatest concern, however, is the uncontrolled spread of water hyacinths because the flora can clog waterways and ruin water bodies. In many cases where a significant volume of water is withdrawn for municipal water supply and irrigation uses, stream augmentation may be desirable to maintain stream flows and to enhance the aquatic and wildlife habitat, as well as to maintain the aesthetic value of the watercourse. Reclaimed water can be used to maintain flow. This will also provide enhancement of aquatic and wildlife habitat and will maintain the aesthetic value of the stream.

15-2-7 Groundwater Recharge

In many areas excessive pumping of groundwater for municipal, industrial, and agricultural uses has resulted in the lowering of the groundwater table. Adverse effects such as subsidence, saltwater intrusion, and in general, shortage of water have indicated a need for artificial recharging of the aquifer. Methods of groundwater recharge include rapid infiltration by effluent application or impoundment, intermittent percolation, and direct injection. In all cases, risks for groundwater pollution exist. Furthermore, direct injection implies high costs of treating effluent for injection and high costs of injection facilities. Also, information regarding the underground environment and geological risk levels of injection must be evaluated. Oil companies have used effluent for recharging oil-bearing strata to increase oil yield.

Considerable improvement in the quality of reclaimed water may occur by percolation through the soil, unsaturated zone, and aquifer.[18] This is called soil-aquifer treatment (SAT). A high reduction in BOD_5; nitrification-denitrification; and removal of synthetic organic compounds, microorganisms, and other chemical constituents may occur during their passage through suitable soils. The major removal processes are microbial decomposition, adsorption, filtration, ion exchange, volatilization, dilution, chemical oxidation and reduction, chemical precipitation, chelation, and complexation. The degree of contaminants removal is dependent upon the ability of the soil to remove different constituents. Heavy metal removal ranges from very small to more than 90 percent, depending upon soil properties and on speciation of the influent metals. There are indications that, once heavy metals are adsorbed, they are not easily desorbed, although desorption may depend in part on buffering capacity, salt concentration, and oxidation-reduction potential.[19,20]

Two methods of groundwater recharge are commonly used: (1) surface spreading and (2) direct injection. Surface spreading utilizes flooding, ridge and furrow, constructed wetlands, and infiltration basins.[1] Direct injection involves pumping of reclaimed water directly into the groundwater zone.[21] While there are obvious advantages with groundwater recharge, there are possible disadvantages.[1,22-24] Among these are

(1) extensive land areas may be needed for spreading basins; (2) the high cost of energy and injection wells for recharge may be prohibitive; (3) recharge may increase the danger of aquifer contamination; (4) aquifer remediation is difficult, expensive, and may take years to accomplish; (5) not all added water may be recoverable; (6) the area required for operation and maintenance of a groundwater supply system is generally larger than that required for a surface water supply system; and (7) sudden increases in water supply demand may not be met because of the slow movement of groundwater.

15-2-8 Augmentation of Potable Water Supply

Water is a renewable resource. Nature purifies it through an unending hydrological cycle, whereby water evaporates from the ocean and land and then is returned as pure rainwater. Technology is now available to treat wastewater to any extent the usage may require. For direct potable reuse, the federal drinking water quality standards must be met. Indirect potable reuse requires treatment of wastewater to a level that it can be discharged into a body of water or underground storage for later reuse as a water supply source. Both direct and indirect potable water reuse are being used or under consideration for many communities. Both uses are briefly presented below.

Direct Potable Water Reuse. Direct potable water reuse on an experimental basis has been attempted in Denver, Colorado; Tempa, Florida; and San Diego, California.[1,25] The reports clearly show that the product water is of better quality than that produced from many surface water sources. The unit processes used to produce potable water from wastewater are well understood, effective, and very reliable on a relatively large scale. However, the cost of monitoring the trace contaminants that are regulated in drinking water is quite high. The direct reuse of wastewater for human consumption will therefore depend upon economies, government regulations, and public acceptance.[24] Many public attitude surveys show that the public will accept and endorse many types of nonpotable reuse while being reluctant to accept potable reuse. Direct reuse of treated wastewater therefore is practicable only on an emergency basis.

Indirect Potable Water Reuse. Indirect potable water reuse is more acceptable to the public than direct potable water reuse. Many natural bodies of water that are used for a municipal water supply are also used for effluent disposal. In this way, the natural purification processes that are continually at work in natural waters result in further purification of wastewater effluents. Thus, water treatment plants would employ only the essential treatment technologies to produce potable water supply. In many instances, these water supply sources are less expensive and less easily developed than the upland unpolluted surface water or underground sources. Such practice may be in line with the basic philosophy of water supply resource selection; that is, priority should be given to selection of the purest source. The polluted source should not be used unless other sources are economically unavailable.

Infiltration gallery, riverbank or dune filtration are also indirect potable water reuse methods. The polluted surface water or reclaimed water is infiltrated into the groundwater zone through the riverbank, infiltration gallery percolation from spreading basins, or percolation from drain fields of porous pipes.[26,27] This method is distinctly different from groundwater recharge of high-quality reclaimed water. Infiltration gallery, riverbank, or dune filtration of surface or reclaimed water provides partial treatment during its passage through the soil until it is pumped out from a gallery or extraction well.

15-2-9 Discharge into Natural Waters

Discharge of effluents into waters is the most common disposal practice. The self-purification, or assimilative, capacity of the natural waters is thus utilized to provide the remaining treatment. The 1972 and 1977 Clean Water Acts established certain effluent limitation requirements—and deadlines for reaching them—in order to restore and maintain the physical, chemical, and biological integrity of the nation's waters.[28,29] All states of the United States were required by the government to adopt water quality standards.

As a part of the overall water quality management and setting of the effluent standards for conservative and nonconservative pollutants, various types of water quality models are widely employed.[a] These models are used to provide predictive answers to such questions as[30-34]

- What will be the net effect of a given level of treatment on the water quality of a receiving body of water?
- How will a particular watershed management practice affect the water quality?
- What are the long- and short-term effects of a certain course of action?
- Is a certain course of action worth the costs of implementation?

Water quality models utilize constituents such as total dissolved solids, total suspended solids, coliform organisms, nutrients, oxidation of carbonaceous and nitrogenous compounds, and dissolved oxygen. Many excellent references are available on this subject.[1,9-11] Relationships between contaminants and their probable impacts are summarized in Table 15-1.

Effluent disposal in natural waters is the most common practice. Therefore, the remainder of this chapter is devoted to the design aspects of the outfalls in the receiving waters. The deoxygenation in rivers because of effluent discharge and requirements and design factors for river, lake, estuarine, and ocean outfalls are briefly discussed. The design procedure for a river outfall and deoxygenation effects are presented in more detail in the Design Example.

Requirements of Outfall. The outfall structures are designed to properly disperse the effluent into the receiving waters. The design of outfall is related to the characteristics of the receiving waters whether they are rivers, lakes, estuaries, or coastal waters of seas and oceans. In all cases consideration must be given to proper dispersion, to avoid a localized pollution problem, for utilization of total available assimilative capacity, and for

[a]Conservative pollutants are those that do not decay. Examples of such pollutants are dissolved salts and heavy metals. The nonconservative pollutants are those that exhibit time-dependent decay, such as BOD, ammonia nitrogen, and radioactive wastes.

TABLE 15-1 Relationship between Contaminants and Their Probable Impacts on Water Quality

	Aesthetics	DO Depletion	Sediment Deposits	Excessive Aquatic Growth	Public Health Threats	Impaired Recreational Value	Ecological Damage	Reduced Commercial Value
Floatables and visual contaminants	X					X	X	
Bacteria, virus					X	X		
Degradable organics		X					X	
Suspended solids	X		X			X		
Nutrients	X			X				
Dissolved solids	X							X
Toxic materials					X		X	

Source: Ref. 30.

avoidance of shore contamination. Requirements of river, lake, estuarine, and ocean outfalls are given below. In most cases the discharge is made below the water surface through one or multiple outlets to reduce foam formation and achieve good dispersion.

River Outfall. Shoreline releases of effluents into the rivers are poorly dispersed and hug the banks for long distances with little mixing. Installation of diffusers across the width of the river is preferred.[b] Sewage effluent is generally warmer than most receiving waters, and therefore it rises while being swept along by dominant currents. Formation of telltale signs such as sleek or detergent froth should be avoided.

Lake Outfall. In lakes an attempt should be made to achieve the best possible dispersion to avoid shore contaminations. Shallow lakes and reservoirs are subject to significant mixing because of wind-induced currents, and effluents are dispersed well. In stratified lakes there is a possibility that the effluent may deplete the dissolved oxygen in the lower stratum.[13] Warmer effluents may also destroy the thermoclines.[c]

Estuarine Outfall. The effluents discharged into the tidal rivers and estuaries may oscillate due to freshwater flow, and tide- or wind-produced seiches. The mixing is caused by dispersion. If stratification occurs because of salinity or temperature, the effluent plume will rise to the surface.

Marine or Ocean Outfall. The marine outfall consists of long pipes to transport the waste long distances from the shore and then release it. The function of a diffuser is to mix the low-density waste with high-density seawater at the bottom. The plume will rise under prevailing currents until the density of the mixture is equal to that of seawater below the surface. At this point the plume will spread horizontally and disperse with little or no visual effects. Some excellent references on marine outfalls are available in the literature.[12,35-37]

Deoxygenation in Rivers Caused by Effluent Discharge. As effluent is discharged into a stream, it mixes with the river water. The enhanced microbiological activity utilizes the dissolved oxygen (DO) in the river to stabilize the biodegradable organic matter. The DO is replenished mainly by atmospheric reaeration, photosynthesis, and additional downstream dilution. A minimum DO of 5 mg/L is necessary for healthy aquatic life. The concentration of conservative pollutants are reduced mainly by dilution. The amount of reaeration is proportional to the DO deficit and oxygen supplied by photosynthesis. Several equations based on the Streeter and Phelps model have been developed to express many relationships.[38-42] These equations [Eqs. (15-1)–(15-7)] are summarized below. If the effect of photosynthesis, algal respiration, and benthic oxygen demand is small, then Eqs. (15-2) and (15-7) become identical.

[b]In navigable rivers, construction of pipelines across the bottom is subject to approval by the U.S. Army Corps of Engineers.

[c]Thermocline is a zone of significant temperature change and is extremely resistant to mixing.

$$C_o = \frac{Q_r C_r + q_w C_w}{Q_r + q_w} \tag{15-1}$$

$$D_t = \frac{KL_o}{K_2 - K}(e^{-Kt} - e^{-K_2 t}) + D_o e^{-K_2 t} \tag{15-2}$$

$$t_c = \frac{1}{K_2 - K} \ln\left[\frac{K_2}{K}\left(1 - \frac{D_o(K_2 - K)}{KL_o}\right)\right] \tag{15-3}$$

$$D_c = \frac{K}{K_2} L_o e^{-Kt_c} \tag{15-4}$$

$$x = tv \text{ or } x_c = t_c v \tag{15-5}$$

$$K_T = K(\theta)^{T-20},\ (K_2)_T = K_2\,(\theta)^{T-20} \tag{15-6}$$

$$D_t = \frac{KL_o}{K_2 - K}(e^{-Kt} - e^{-K_2 t}) + D_o e^{-K_2 t} + \frac{S + R - P}{K_2}(1 - e^{-K_2 t}) \tag{15-7}$$

where

C_o = initial concentration of constituent in the mixture at the point of discharge, mg/L
C_r = concentration of constituent in river before mixing, mg/L
C_w = concentration of constituent in wastewater effluent, mg/L
D_o = initial oxygen deficit of the mixture at the point of waste discharge (time $t = 0$), mg/L
D_c = critical dissolved oxygen deficient at point X_c or at time t_c, mg/L
D_t = oxygen deficit at time t, mg/L
$(K)_{20}$ = first-order reaction rate constant at 20°C, d^{-1} (base e). The value of K_{20} for domestic wastewater effluent may be in the range of 0.1–0.2 d^{-1}. More information on this topic may be found in Chapter 3.
$(K_2)_{20}$ = reaeration constant at 20°C, d^{-1} (base e). The value of $(K_2)_{20}$ depends upon the characteristics of stream for different types of stream conditions. The typical values at 20°C are: small ponds = 0.1, sluggish stream = 0.3, swift stream = 0.8, and rapid stream with falls = 1.7 d^{-1}.
K = reaction rate constant at any temperature T °C
K_2 = reaeration constant at temperature T °C
L_o = ultimate BOD of the mixture at the outfall, mg/L
P = DO supplied by photosynthesis, d^{-1}
Q_r = river discharge, m^3/s
q_w = wastewater discharge, m^3/s
R = DO uptake by algal respiration, d^{-1}
S = DO uptake by benthic deposits, d^{-1}
t = flow time in the river downstream from the outfall point, s
t_c = critical time when the critical dissolved oxygen deficit D_c occurs. This is the time to flow from the outfall to distance X_c, d.
T = temperature, °C

θ = temperature coefficient. For reaction rate constant K, the typical value of $\theta = 1.024$. For reaeration constant K_2, the typical value of $\theta = 1.135$.
v = river velocity, m/s
X = distance downstream from the outfall corresponding to time t or dissolved oxygen deficit of D_t, m
X_c = distance from the outfall where critical D_c occurs, m

Design Factors for Outfall. Depending on the characteristics of the receiving waters, many factors are considered for proper mixing and dispersion of effluent. These factors include (1) flow velocity, (2) stratification because of salinity or temperature, (3) depth (shallow or deep), (4) shape (wide or narrow), (5) reversal of current (tidal or wind-produced seiches), (6) wind circulation, and (7) temperature and salinity of effluent. For design and construction of outfalls, three principal factors are considered:

1. Material of the outfall pipe
2. Means of anchoring the outfall in the body of water
3. The design of diffusers

Each of these factors is briefly discussed below.

Material. The construction material for outfall may be steel, concrete, plastic, or any material that is compatible with the waste stream, receiving water, site characteristics, and economics of fabrication and installation.

Installation and Anchoring. Installation and anchoring of outfall pipes and diffusers is especially important if strong currents prevail, and additional problems such as errosion and scour and transport of bedload, particularly such objects as tree trunks, may be expected. Outfall pipes are secured by rock ballast, cement or sand bags, or chain and anchors. Often, pipes may be partly embedded in concrete apron.[35,43]

Diffusers. Diffusers must disperse the effluent efficiently and evenly to achieve good mixing. Knowledge of density and temperature gradients, as well as currents and wind conditions that influence the dispersion in receiving waters, is necessary. Procedure for determining initial dilution and dispersion for ocean outfalls is given in Ref. 35.

Equations (15-8)–(15-11) are used for the design and analysis of the diffusers.[14]

$$q_1 = C_{D_1} a_1 \sqrt{2gE_1} \tag{15-8}$$

$$E_1 = \frac{\Delta P}{\gamma} = ELV_1 - ELV_0 \tag{15-9}$$

$$E_2 = E_1 + h_{f_{1-2}} + \frac{\Delta S_g}{S_g} \times \Delta Z_{1-2} \tag{15-10}$$

$$h_{f_{1-2}} = f\frac{L_{1-2} \times V_{1-2}^2}{D_{1-2} \times 2g} \tag{15-11}$$

where

a_1 = area of port 1
C_{D_1} = coefficient of discharge for port 1. The value of C_{D_1} is a function of $(V_{1-2}^2/2\ g)/E_1$. This relationship is graphically shown in Figure 15-1.[35]
D_{1-2} = diameter of the diffuser pipe between ports 1 and 2
E_1 = assumed operating head for port 1
E_2 = total head in the diffuser pipe at port 2. Port 2 is upstream of port 1.
ELV_0 = free water surface elevation in the receiving water
ELV_1 = free water surface elevation in the diffuser pipe upstream of port 1
f = Darcy friction factor
$h_{f_{1-2}}$ = head loss caused by friction in the diffuser pipe between ports 1 and 2
L_{1-2} = length of pipe between ports 1 and 2
$\dfrac{\Delta P}{\gamma}$ = difference in pressure head between inside and outside of diffuser port 1
q_1 = discharge through port 1 (Figure 15-1). Port 1 is the outermost port.

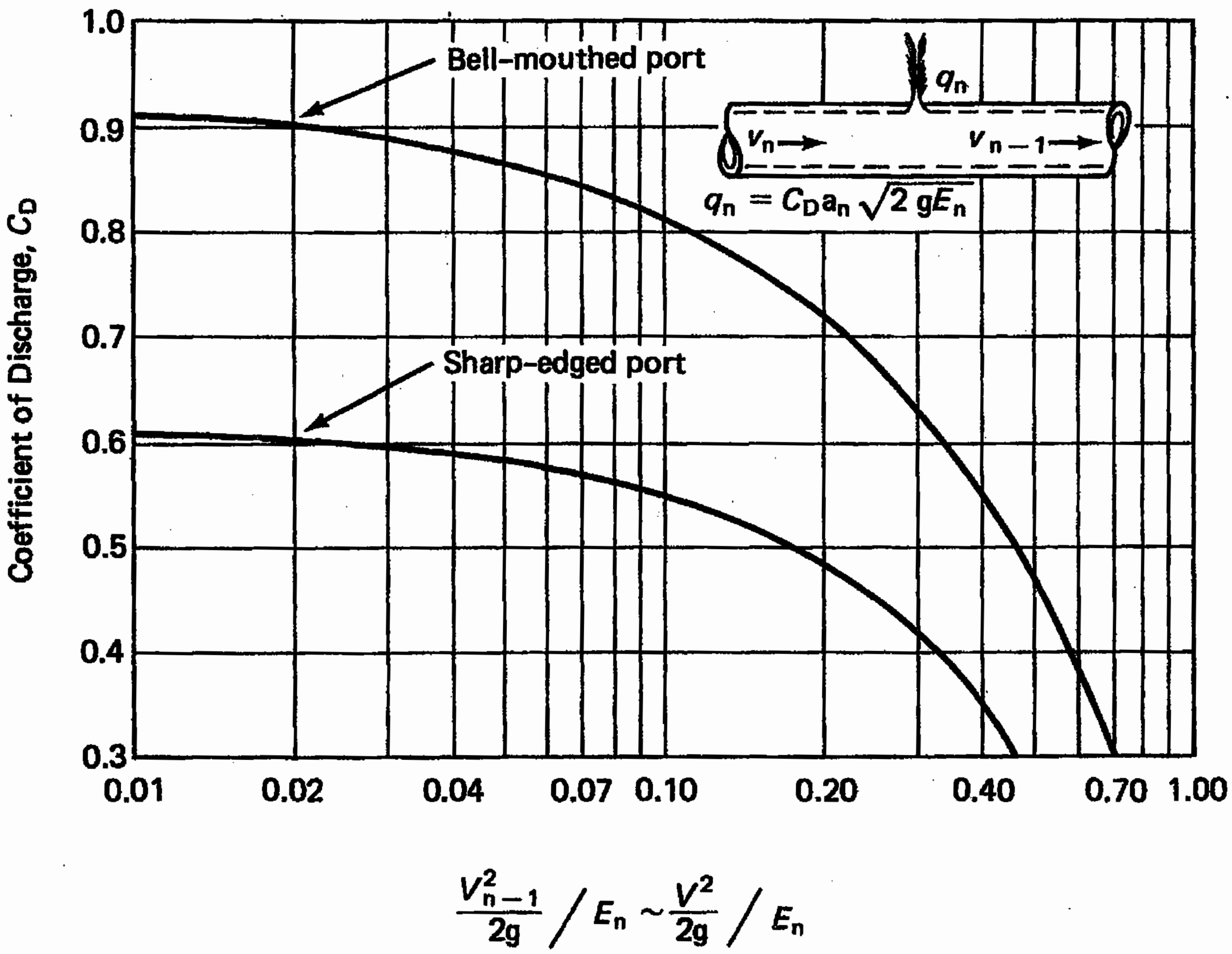

Figure 15-1 Coefficient of Discharge for Small Ports on the Side of a Pipe (from Ref. 28; used with permission of American Society of Civil Engineers).

S_g = specific gravity of the receiving water
ΔS_g = difference in specific gravity between receiving water and effluent
V_{1-2} = velocity in the diffuser pipe between ports 1 and 2 = $\dfrac{q_1}{\dfrac{\pi}{4} D_{1-2}^2}$

The diffusers may have a total area approximately half the cross-sectional area of the pipe. The diameter of the port is governed by considerations of clogging, hydraulic performance, and spacing. Typical diffusers have port diameters 10–30 cm, spaced over a desired length. Two ports are provided in the bulkhead at the end of the discharge pipe. One port is on the top and the other is at the bottom. These serve not only to discharge waste, but also to provide outlets for settleable and floatable materials. Other ports are normally alternated on the opposite side of the pipe at the center line.[43] The minimum center-to-center spacing of the port is 10 times the port diameter.[35]

Design Procedure. The hydraulic analysis of multiport diffusers is basically a problem in manifolds and is somewhat complex. A discussion of this subject may be found elsewhere.[44,45] A simplified procedure is given in Ref. 35. It is a process of hydraulic iteration using a trial-and-error solution. The procedure is illustrated in the Design Example.

The diffusers are generally designed for maximum flow. The performance of the diffuser at lower flows must also be determined. In case low velocities are encountered under initial flow conditions, it is customary to plug the inboard ports during initial flow periods and open them later as the flow increases.

15-3 INFORMATION CHECKLIST FOR DESIGN OF OUTFALL STRUCTURE

The following information is necessary to properly design an outfall system:

1. Type of receiving water (river, lake, estuary, or ocean) to be used for disposal of effluent
2. Uses of receiving water (water supply, propagation of aquatic life, recreation, etc.)
3. Treatment plant design criteria and effluent disposal requirements for the receiving waters prepared by the concerned regulatory agencies
4. Peak, average, and minimum flows for the design and initial years
5. Characteristics of receiving water (topography, salinity and temperature profiles, water currents, wind circulation, and water elevations at low flow and high flood conditions)
6. Available head, that is, the differential elevation between the water surface elevation in the outfall channel or pipe, and the high flood level in the receiving water
7. Length of outfall channel or pipe (distance from the last treatment unit to the receiving waters)
8. Postaeration requirements to meet the effluent discharge requirements
9. Existing topographic map from the treatment plant site to the receiving water

15-4 DESIGN EXAMPLE

15-4-1 Design Criteria Used

The following design criteria are used for the design of outfall structures:

1. Provide a shallow trapezoidal channel from the disinfection facility to the outfall structures. Postaeration is not required.
2. Provide a collection box to connect the outfall channel and the outfall pipe that contains the diffuser.
3. The outfall pipe shall be designed for peak design flow of 1.321 m^3/s.
4. The diffusers shall be designed for the peak design flow of 1.321 m^3/s.
5. The diffuser pipe shall be provided longitudinally along the centerline of the river confluence. The selected alignment is shown in Figure 15-2(a) and (b).

15-4-2 Design Calculations

Step A: Trapezoidal Outfall Channel. The outfall channel is a concrete, lined trapezoidal channel that transports the effluent from the UV disinfection basin to the outfall structure. The channel has a bottom width of 0.5 m and side slope of three horizontal to one vertical. A shallow depth of effluent is maintained in the channel to effect natural reaeration. Details of the outfall channel are shown in Figure 15-2(c). The slope of the trapezoidal channel and depth of flow calculations are given in Chapter 21.

Step B: Collection Box. A 2 m × 2 m collection box is provided to collect the flow from the trapezoidal channel and discharge it into the outfall pipe. A smooth transition shall be provided from the trapezoidal section to the square collection box. The depth of the collection box is determined from the maximum liquid level in the box.

The collection box has a sluice gate to be used in case high flood waters warrant protection of the treatment facility against flooding. Emergency pumps will be required to pump the effluent into the receiving water. Design procedures for a pump station are provided in Chapter 9. Details of the collection box are shown in Figure 15-2(a) and (b).

Step C: Outfall Pipe. The outfall pipe is a cast iron circular pipe that delivers the effluent into the diffuser pipe.

1. Compute the diameter and select length.
   Assume a velocity of 2 m/s through the outfall pipe at a peak design flow of 1.321 m^3/s.

$$\text{Area of the outfall pipe} = \frac{1.321\ \text{m}^3/\text{s}}{2\ \text{m/s}} = 0.661\ \text{m}^2$$

$$\text{Diameter} = \sqrt{0.661\ \text{m}^2 \times 4/\pi} = 0.92\ \text{m}$$

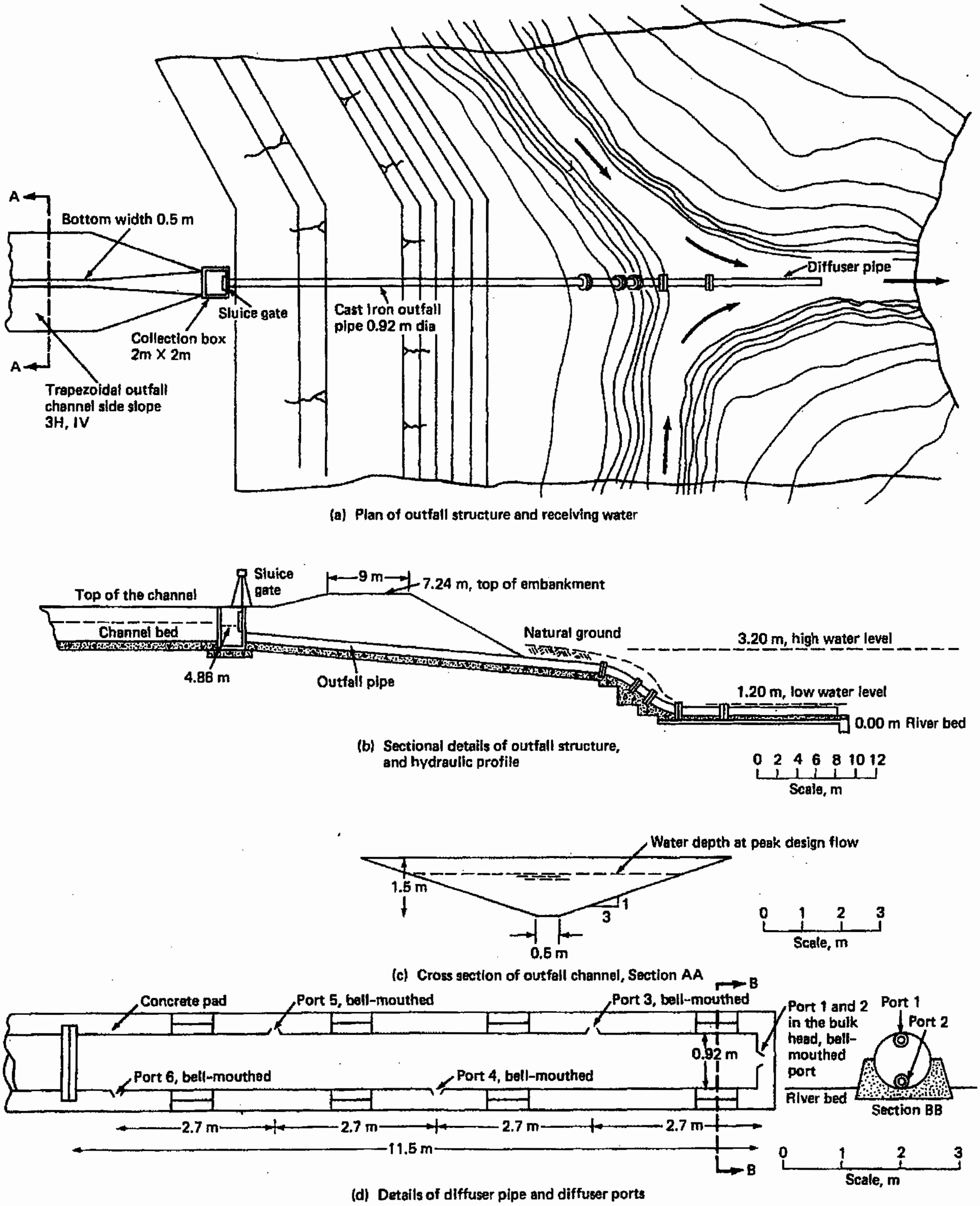

Figure 15-2 Details of Outfall Structure.

The outfall pipe is approximately 50 m long and has several bends, as shown in Figure 15-2. It will discharge under pressure at peak design flow.

2. Compute head losses.
The head losses in straight pipe, bends, and velocity head are calculated from Eqs. (9-3), (9-4), and (9-5). General discussion may be found in Chapter 9.

$$\text{Head loss caused by entrance } K = 0.5 \text{ (Appendix B)} = \frac{0.5 \times (2\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2}$$

$$= 0.1\text{ m}$$

Head loss caused by bends, 2 bends 22.5° each, $K = 0.11$ (Appendix B)

$$= 2 \times \frac{0.11 \times (2\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = 0.05\text{ m}$$

Head loss caused by friction ($f = 0.024$)

$$= \frac{0.024 \times 50\text{ m} \times (2\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2 \times 0.92\text{ m}}$$

$$= 0.27\text{ m}$$

$$\text{Total head loss} = 0.10\text{ m} + 0.05\text{ m} + 0.27\text{ m} = 0.42\text{ m}$$

Step D: Diffuser Pipe

1. Select a design procedure for diffuser pipe design.
The design of a diffuser pipe is basically a trial-and-error procedure. Once a tentative configuration is selected, it is designed by the procedure described below.[35,43] The procedure consists of selecting an operating head at the end of the diffuser and then calculating the flow through the outermost ports. Using the flow, the head loss in the pipe between the outermost port and the next upstream port is calculated. The head loss plus the velocity head, the density differential head, and the slope of the diffuser will give the operating head at the next inboard port. The flow at this port is then calculated and the procedure is continued over the entire length of the diffuser. The sum of discharges through all ports should equal the design flow through the outfall sewer. If they are not equal, the operating head is changed and the calculations are repeated. The process can be tedious if done manually.
2. Select the design equations.
Eqs. (15-8)–(15-11) are used for the design of diffuser pipe. The design procedure and calculation steps are provided later in this section.
3. Select a tentative diffuser configuration.
Provide a diffuser pipe equal in diameter to the outfall pipe (0.92 m):

$$\begin{array}{l}\text{Provide diffuser port area}\\ \text{approximately half the}\\ \text{area of the diffuser pipe}\end{array} = \frac{\pi}{4}(0.92\text{ m})^2 \times \frac{1}{2} = 0.33\text{ m}^2$$

Provide each port 27 cm in diameter:

$$\text{Area of each port} = \frac{\pi}{4}(0.27\text{ m})^2 = 0.057\text{ m}^2$$

$$\text{Number of ports} = \frac{0.33\text{ m}^2}{0.057\text{ m}^2} = 6$$

Provide two ports in the bulkhead at the end and four ports on the sides of the pipe. Total length of the diffuser pipe using center-to-center spacing 10 times the diameter of the port = 4(10 × 0.27 m) = 10.8 m (35.4 ft). Provide a 11.5-m-long diffuser pipe. The details and diffuser configuration are shown in Figure 15-2. All ports have a divergent mouthpiece.

4. Compute head losses.
The head loss calculations for the diffuser pipe involve an iterative process. The step-by-step calculations for the final trial at peak design flow are summarized in Table 15-2. In freshwater outfalls (rivers and lakes), the effect of density gradient or the factor $[(\Delta S_g/S_g)\Delta Z]$ in Eq. (15-10) is ignored. Also, the total discharge through all ports should equal the design flow. In this case total flow through six ports is 1.37 m^3/s against the peak design flow of 1.321 m^3/s. Perhaps another trial will be necessary to achieve a closer agreement. The total head loss in the diffuser pipe is 1.24 m. This head loss includes the operational head of 1.2 m at the end of the diffuser pipe.

Step E: Head Losses and Hydraulic Profile. The head loss calculations for various components of outfall structure have been given earlier. Following is the summary of head losses that are encountered at peak design flow when the differential operating head between the water surface at high flood level and inside of diffuser pipe = 1.2 m.

1. Total head loss in diffuser pipe = 1.24 m
2. Total head loss in outfall pipe = 0.42 m

Total 1.66 m

The hydraulic profile of the outfall structure under peak design flow condition is shown in Figure 15-2. This profile is prepared with respect to the channel bed.

Step F: Determine the Critical DO in the Stream

1. Establish the parameters for DO calculations.
The effluent is discharged into the Big River at the confluence of East and West Forks. The 30-d mean low flow and water quality data is summarized in Table 6-3. It is expected that, as the wastewater from the stablization ponds and trickling filter facility is diverted to the proposed wastewater treatment facility, there will be no effluent discharge into the West Fork and East Fork. Consequently, the water quality in the Big River at the confluence will improve significantly. An extensive water quality modeling effort was undertaken during the development phases of the water quality management (WQM) plan of the state. The following information was developed on the

TABLE 15-2 Design Calculations for Diffuser Pipe Used to Disperse Effluent into the River

Factor	Port Numbers					
	1[a]	2[a]	3	4	5	6
1. Distance from the end, m	0	0	2.700	5.400	8.100	10.800
2. Port diameter d, m	0.270	0.270	0.270	0.270	0.270	0.270
3. Port area a, m^2	0.057	0.057	0.057	0.057	0.057	0.057
4. Total head E, m	1.200[b]	1.200	1.202	1.206	1.214	1.225
5. Pipe diameter D, m	0.920	0.920	0.920	0.920	0.920	0.920
6. Area of pipe A, m^2	0.660	0.660	0.660	0.660	0.660	0.660
7. Port velocity v, m/s	4.850[c]	4.850	4.856	4.864	4.880	4.902
8. Discharge through pipe, m^3/s	—	0.486[d]	0.724[e]	0.949	1.163	1.370
9. Velocity in pipe V, m/s	—	0.736	1.100	1.438	1.762	2.076
10. Velocity head $V^2/2g$, m	—	0.028	0.062	0.105	0.158	0.220
11. $V^2/2g/E$	—	0.023	0.052	0.087	0.130	0.179
12. C_D	0.880	0.880[f]	0.860	0.810	0.770	0.740
13. Port discharge q, m^3/s	0.243	0.243	0.238	0.225	0.214	0.207
14. Friction factor	—	0.024	0.024	0.024	0.024	0.024
15. Pipe length to next port L, m	—	2.700	2.700	2.700	2.700	2.700
16. Head loss in pipe h_f, m	—	0.002[g]	0.004	0.008	0.011	0.016
17. Density head $[(\Delta S_g/S_g)\Delta Z]$, m	0	0	0	0	0	0
18. Total head E, m	—	1.202[h]	1.206	1.214	1.225	1.240

[a]Ports 1 and 2 are provided in the bulkhead at the end of the discharge pipe.
[b]Assumed operating head at the end of the diffuser.
[c]$v = \sqrt{2gE} = \sqrt{2 \times 9.81 \text{ m/s}^2 \times 1.2 \text{ m}} = 4.85$ m/s.
[d]Total flow through the ports downstream of the pipe section = flow from ports 1 and 2.
[e]Total flow in the pipe = (0.243 + 0.243 + 0.238) m^3/s = 0.724 m^3/s. The flow through port 3 (0.238 m^3/s) is obtained from Eq. (15-10) by using a trial-and-error solution.
[f]Value is obtained from Figure 15-1, $V^2/2g/E = 0.023$, and the curve for bell-mouthed port.
[g]$h_f = 0.02 \times 2.7 \text{ m} \times 0.028 \text{ m}/0.92 \text{ m} = 0.002$ m.
[h]Total head = 1.2 + 0.002 = 1.202 m.

Big River under a 30-d mean low flow condition and on the effluent from the proposed wastewater treatment plant.

The Big River, 30-d Mean Flow Condition

$$
\begin{aligned}
\text{Flow} &= 3.4 \text{ m}^3/\text{s} \\
\text{Average depth} &= 1.2 \text{ m} \\
\text{Average width} &= 5.5 \text{ m} \\
\text{Average velocity} &= 0.5 \text{ m/s (1.8 km/h)} \\
\text{Temperature} &= 21^\circ\text{C} \\
K_2 &= 0.6 \text{ d}^{-1} \\
\text{BOD}_5 &= 1.2 \text{ mg/L} \\
\text{DO saturation} &= 92\% \\
\theta \text{ for } K_2 &= 1.024
\end{aligned}
$$

- The effect of algal bloom in the river and the benthic oxygen demand on the DO sag curve are negligible.

Effluent Quality (Proposed Plant)

$$\begin{aligned}
\text{Temperature} &= 25°\text{C} \\
\text{Average effluent discharge} &= 0.44\ \text{m}^3/\text{s} \\
\text{DO after natural aeration in the channel} &= 4.0\ \text{mg/L} \\
\text{BOD}_5 &= 10\ \text{mg/L} \\
K &= 0.15\ \text{d}^{-1} \\
\theta \text{ for } K &= 1.12
\end{aligned}$$

2. Calculate temperature, DO, and BOD of the mixture.

Temperature of the mixture [Eq. (15-1)]
$$= \frac{3.4\ \text{m}^3/\text{s} \times 21°\text{C} + 0.44\ \text{m}^3/\text{s} \times 25°\text{C}}{(3.4 + 0.44)\ \text{m}^3/\text{s}}$$
$$= 21.5°\text{C}$$

BOD_5 of the mixture
$$= \frac{3.4\ \text{m}^3/\text{s} \times 1.2\ \text{mg/L} + 0.44\ \text{m}^3/\text{s} \times 10\ \text{mg/L}}{(3.4 + 0.44)\ \text{m}^3/\text{s}}$$
$$= 2.20\ \text{mg/L}$$

DO saturation at 21.3°C (Appendix A) $= 8.93$ mg/L

At 90% saturation, stream DO $= 8.93\ \text{mg/L} \times 0.92 = 8.22\ \text{mg/L}$

DO of the mixture
$$= \frac{3.4\ \text{m}^3/\text{s} \times 8.22\ \text{mg/L} + 0.44\ \text{m}^3/\text{s} \times 4.0\ \text{mg/L}}{(3.4 + 0.44)\ \text{m}^3/\text{s}}$$
$$= 7.74\ \text{mg/L}$$

3. Calculate initial DO deficit D_o.

$$D_o = 8.93\ \text{mg/L} - 7.74\ \text{mg/L} = 1.19\ \text{mg/L}$$

4. Calculate ultimate BOD (L_o) of the mixture.

$$L_o = \frac{\text{BOD}_5}{1 - e^{-K/\text{d} \times 5\text{d}}} = \frac{2.20\ \text{mg/L}}{1 - e^{-0.15/\text{d} \times 5\text{d}}}$$
$$= 4.17\ \text{mg/L}$$

5. Correct the rate constants to 21.5°C.

$$K = 0.15\ \text{d}^{-1}\ (1.12)^{21.5-20} = 0.30\ \text{d}^{-1}$$
$$K_2 = 0.6\ \text{d}^{-1}\ (1.024)^{21.5-20} = 0.62\ \text{d}^{-1}$$

6. Determine t_c and x_c and D_c.

$$t_c \text{ [Eq. (15-3)]} = \frac{1}{K_2 - K} \ln \left[\frac{K_2}{K} \left(1 - \frac{D_o(K_2 - K)}{KL_o} \right) \right]$$

$$= \frac{1}{0.62/\text{d} - 0.30/\text{d}} \ln \left[\frac{0.62/\text{d}}{0.30/\text{d}} \left(1 - \frac{1.19 \text{ mg/L}(0.62 - 0.30)/\text{d}}{0.30/\text{d} \times 4.17 \text{ mg/L}} \right) \right]$$

$$= 1.13 \text{ d}$$

Distance from outfall where critical DO will occur:

$$X_c \text{ [Eq. (15-5)]} = 1.13\text{d} \times 1.8 \text{ km/h} \times 24 \text{ h/d}$$

$$= 48.8 \text{ km}$$

Critical DO deficit D_c [Eq. (15-4)]

$$= \frac{K}{K_2} L_o e^{-Kt_c}$$

$$= \frac{0.30}{0.62} \times 4.17\ e^{-0.3 \times 1.13}$$

$$= 1.44 \text{ mg/L}$$

Critical DO

$$= 8.93 \text{ mg/L} - 1.44 \text{ mg/L}$$

$$= 7.49 \text{ mg/L}$$

The minimum DO concentration in the Big River under the low flow condition is high enough to support healthy aquatic life.

Step G: Design Details. The design details of the outfall structure are shown in Figure 15-2.

15-5 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT OUTFALL STRUCTURE

The operation and maintenance of the outfall structure include the following:

1. Check the trapezoidal effluent channel periodically for algae growth on the side slopes. Large masses of algae growth should be removed.
2. Check the collection box for obstructions or excessive slime growth on a regular basis. Any rise of water surface in the outfall channel is an indication of flow obstruction in the outfall structure.
3. The diffuser pipe is designed for the peak design flow. Under initial flow conditions, low velocities will be encountered. Therefore, a sufficient number of inboard ports shall be plugged during initial flow periods and opened later as the flow is increased with time.
4. Ports 1 and 2 are provided in the bulkhead at the top and bottom. These ports will facilitate self-cleaning of the diffuser pipe from the floating and settled solids.

15-6 SPECIFICATIONS

The effluent structure includes effluent channel (rectangular), outfall channel (trapezoidal), collection box with sluice gate, cast iron outfall pipe, and steel diffuser pipe. The contractor shall be responsible for furnishing, construction, installation, and testing of all concrete channels, cast and ductile iron pipe and fittings, manholes, and specials as shown on the plans and provided for in these specifications. The plans show sizes and general arrangements of channel, pipes, and appurtenances. Responsibility for furnishing exact lengths of all channels, pipes for proper makeup, and transitions rests with the contractor. All concrete, cast iron, and steel pipe fittings shall conform to the standards (see specifications in Chapters 7 and 12). The diffuser and outfall pipes shall be properly anchored deep in the channel bed and in the embankment to protect them from erosion, scour, and uplift. The concrete apron shall be secured in place by poured concrete of sufficient depth below the channel bed.

15-7 PROBLEMS AND DISCUSSION TOPICS

15-1 In this chapter, nine methods of effluent disposal and reuse are presented. Describe the advantages and disadvantages of each method for a wastewater treatment facility that is located in an urban setting.

15-2 Secondary effluent is discharged from a publicly owned treatment works (POTW) into a stream. The wastewater flow is 10,000 m^3/d and has a BOD_5 of 20 mg/L. The summer temperature and DO in the effluent are 25°C and 2 mg/L. The water quality of stream above the effluent discharge point are $Q = 0.3$ m^3/s, DO = 7.5 mg/L, BOD_5 = 3 mg/L, and average water temperature is 21°C. At 20°C, deoxygenation coefficient of effluent and reaeration coefficient of the stream after effluent mixing are 0.23/d and 0.45/d. The average velocity of water below the outfall is 0.2 m/s. Use the same temperature coefficients as given in the Design Example. Calculate critical DO and distance from outfall where this value occurs. Draw the DO profile from the outfall to a distance where 90 percent DO saturation is resumed.

15-3 An outfall pipe 15 cm in diameter is discharging into a lake. The pipe is at the bottom of a lake that has an elevation of 102.00 m. The water surface elevation is 105.50 m. The length of the pipe is 280 m, and it has four 45° bends ($K = 0.2$). Calculate the water surface elevation in the upstream unit. $Q = 27$ L/s, $f = 0.024$, $K_{entrance} = 0.5$, and $K_{exit} = 1.0$.

15-4 Develop at an average design flow of 0.44 m^3/s the hydraulic profile through the outfall system provided in the Design Example.

15-5 During the initial years when the flow is small, many inboard ports are tightly plugged. Calculate the head loss through the outfall structure in the Design Example at an initial peak flow of 966 L/s. Assume ports 5 and 6 are plugged.

15-6 Calculate the head loss through the outfall system given in the Design Example. Assume sharp-crested ports. Peak design flow is 1.321 m^3/s, and the number of ports are six, as given in Figure 15-2.

15-7 Describe different methods of effluent disposal. Prepare a checklist to indicate advantages and disadvantages of each method and conditions where each method may or may not be suitable for effluent disposal.

15-8 What are various factors that an engineer must take into consideration for designing an outfall structure?

15-9 Develop a design procedure for designing a marine outfall structure where the specific gravity term cannot be ignored. List your computational procedure.

15-10 Visit the wastewater treatment facility in your community. Describe the method of effluent disposal and its impact upon the environmental quality. How could the impacts be minimized?

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16

Sources of Sludge and Thickener Design

16-1 INTRODUCTION

The principal sources of sludge at municipal wastewater treatment plants are the primary sedimentation basin and the secondary clarifiers. Additional sludge may also come from chemical precipitation, nitrification-denitrification facilities, screening and grinder, and filtration devices if the plant has these processes. Many times, the sludge produced in these processes is recycled through the primary or secondary treatment systems so that the sludge is removed as either primary or secondary sludge. In some cases, secondary sludge is returned to the primary settling tank, ultimately giving a single sludge stream consisting of combined sludge.

Sludge contains large volumes of water. The small fraction of solids in the sludge is highly offensive. Thus, the problems involved with handling and disposal of sludge are complex. Common sludge management processes include thickening, stabilization, dewatering, and disposal. Chapters 16–19 are devoted exclusively to various sludge-processing systems. The main purposes of this chapter are to (1) describe the basic characteristics of sludge produced in the primary and secondary treatment processes, (2) present a general overview of sludge pumping and processing systems and environmental control measures, (3) describe various sludge-thickening methods, and (4) present the step-by-step design procedure of a gravity thickener in the Design Example.

16-2 CHARACTERISTICS OF MUNICIPAL SLUDGE

Sludge consists of organic and inorganic solids. The solids are largely composed of substances responsible for offensive odors. The primary sludge contains solids present in the raw wastewater, while secondary sludge contains chemical or biological solids produced during the treatment processes. The specific gravity of inorganic solids is about 2–2.5 and that of organic fraction is 1.2–1.3. The characteristics of wastewater residues and

TABLE 16-1 *Physical Characteristics of Sludge from Municipal Wastewater Treatment Plant*

			Characteristics		
Source of Sludge	**Description**	**Quality (g/m^3)**	**Solids (%)**	**Specific Gravity of Solids**	**Specific Gravity of Sludge**
Primary	Gray and slimy, extremely offensive odor. VSS is 60–70%. Readily digested in aerobic or anaerobic digesters	105–165	4–8	1.4	1.02
Activated sludge (waste sludge)	Biological solids, flocculating, brownish appearance; when fresh has earthy smell. Turns dark and septic rapidly and has disagreeable odor of putrification. VSS is 70–80%. Readily digested in aerobic and anaerobic digester	70–100	0.8–2.0 (from clarifier) 0.2–0.6 (from aeration basin)	1.25	1.005
Trickling filter (waste sludge)	Brownish and flocculating. Earthy smell when fresh; undergoes decomposition and turns offensive. VSS 60–75%. Readily digested	50–90	2–4	1.45	1.025
Chemical sludge (chemical addition in primary sedimentation basin for phosphorus removal),	Grayish brown, slimy, and gelatinous. Undergoes decomposition but at a slower rate				
Iron salts		200–250	0.5–3.0	1.6	1.04
Low lime		240–300	2.0–8.0	1.9	1.04
High lime		500–1000	4.0–15.0	2.2	1.05

$g/m^3 \times 8.3 \times 10^{-3} = lb/10^3$ gal.

TABLE 16-2 Typical Chemical Composition of Raw and Digested Sludges

Constituents	Primary Sludge	Waste Activated Sludge	Combined Digested Sludge
pH	5.0–6.5	6.5–7.5	6.5–7.5
Total dry solids, percent	3–8	0.5–1.0	5.0–10.0
Total volatile solids, percent dry wt.	60–90	60–80	30–60
Specific gravity of individual solids particles	1.3–1.5	1.2–1.4	1.3–1.6
Bulk specific gravity	1.02–1.03	1.0–1.005	1.03–1.04
BOD_5/TVS	0.5–1.1	—	—
COD/TVS	1.2–1.6	2.0–3.0	—
Alkalinity, mg/L as $CaCO_3$	500–1500	200–500	2500–3500
Cellulose, percent dry wt.	8–15	5–10	8–15
Hemicellulose, percent dry wt.	2–4	—	—
Lignin, percent dry wt.	3–7	—	—
Grease and fat, percent dry wt.	6–35	5–12	5–20
Protein, percent dry wt.	20–30	32–41	15–20
Nitrogen N, percent dry wt.	1.5–4.0	3.5–8.5	1.6–6.0
Phosphorus P, percent dry wt.	0.8–2.8	2.0–7.0	1.4–4.0
Potash, percent dry wt.	0.1–1.0	0.2–0.5	0.1–3.0
Heating value, kJ/kg	15,000–24,000	12,000–16,000	6000–14,000
Cadmium, mg/kg	—	—	3–3,410
Chromium, mg/kg	—	—	24–28,850
Copper, mg/kg	—	—	85–10,100
Lead, mg/kg	—	—	58–19,730
Nickel, mg/kg	—	—	2–3,520
Zinc, mg/kg	—	—	108–27,800
Xexachlorobenzene, mg/kg	0.4	0.8	—
Lindane, mg/kg	0.6	1.0	—
Chlordane, mg/kg	2.6	4.4	—

1 kJ/kg = 0.43 Btu/lb.
Source: Adapted in part from Refs. 1–6.

their production rates from various treatment processes are summarized in Table 4-3. The typical physical characteristics of sludge from different treatment processes are given in Table 16-1, and chemical compositions of raw and digested sludges are supplied in Table 16-2. The maximum allowable heavy.metals concentration in biosolids for land application and governing regulations are covered in Chapter 19 (Table 19-2).

16-3 SLUDGE-PROCESSING SYSTEMS AND ENVIRONMENTAL CONTROL

Various sludge-processing systems include (1) thickening, (2) stabilization or digestion, (3) dewatering, and (4) disposal. These systems were briefly discussed in Chapter 4 (see

Tables 4-6 and 4-8 and Figure 4-3). More detailed discussion on thickener design is given in this chapter. Other processes are covered in Chapters 17–19.

Large volumes of liquid waste are produced when sludge is processed. These liquid wastes contain high concentrations of suspended solids and BOD_5 and, therefore, are returned to the primary or secondary treatment facility. A material mass balance analysis for the liquid and sludge treatment processes is performed at average design flow. The material mass balance analysis is discussed in Chapter 13. The quantities of sludge thus calculated from the material mass balance analysis is generally increased to accommodate the temporary increases in the sludge production caused by sustained hydraulic or organic loadings. Many regulatory agencies require sludge-processing systems capable of handling an average of 30 consecutive days of sustained maximum flow or organic loading conditions (see Chapter 3). The quantities of sludge under such sustained loading is normally 5–20 percent of the material mass balance analysis performed at average design flow. The ratio of sludge solids for average maximum month and average maximum week to annual average day may be 1.15 and 1.25, respectively.[4,5] Therefore, proper allowances for sustained flow conditions should be made for sludge-handling and disposal facilities.

Odor control devices must also be considered for design of sludge management systems. These devices generally fall into three categories: containment, destruction, and masking. Containment of odors is accomplished by covering or enclosing processing equipment. Destruction of odors is achieved by chemical oxidation or scrubbing. Masking agents are used as a temporary measure or as a last resort to camouflage widespread odors.

16-4 DESIGN CONSIDERATIONS

The design components of a sludge management system are sludge storage, piping, pumping to a central location, mixing and blending, and processing. Many physical and chemical properties of sludge must be considered for proper system design.

16-4-1 Volume–Weight Relationship

Quantity Characteristics and Variation. The quantity and characteristics of solids vary, depending on their origin, storage and aging, and processing. Specific constituents include organics, nutrients, pathogens, metals, and toxic organic compounds. A variation in solids load may be significant. Peak solids load for two to five consecutive days' duration may be three to five times the average day load. The designer of the sludge-processing and disposal facility must consider average and maximum solids production under sustained loading.

Specific Gravity. The sludge contains a large volume of water. The specific gravity of sludge therefore depends on (1) volume of water and (2) specific gravity of solids. The specific gravity of solids depends upon the proportion of mineral matter and volatile matter. The general equation used to calculate the weight of dry solids in sludge is given by Eq. (16-1):

$$\frac{W_s}{S_s \rho_w} = \frac{W_f}{S_f \rho_w} + \frac{W_v}{S_v \rho_w} \tag{16-1}$$

where

ρ_w = density of water, kg/m^3
S_f = specific gravity of fixed solids in sludge
S_s = specific gravity of solids in sludge
S_v = specific gravity of volatile solids in sludge
W_f = weight of fixed solids, kg/m^3
W_s = weight of solids, kg/m^3
W_v = weight of volatile solids, kg/m^3

Specific gravity of solids and sludge obtained from different processes are provided in Table 16-1.

16-4-2 Sludge and Scum Pumping

Sludge and scum are pumped within the plant from one unit to another; sometimes, off-site pumping may be necessary. Different types of pumps may be needed for different pumping applications. Pumps commonly used for sludge application are plunger, torque-flow, centrifugal, progressive cavity, diaphragm, high-pressure piston, and rotary-lobe-type pumps. Some sludge application pumps are shown in Figure 9-1. A summary of many sludge pumps and their applications are provided in Table 16-3.

Head Loss. The head loss calculations are essential to design and select proper pumping equipment. The head loss depends upon the pipe material, pumping velocity, and flow properties of sludge. As a general rule, head loss increases with solids contents, volatile matter, and lower temperature. In the design of wastewater treatment plants, the sludge pumping is necessary for short distances within the plant property. Designers use the most familiar Hazen-Williams equation with a modified friction factor C applicable to different sludge. The modified values of C for different types of sludges are summarized in Table 16-4. These values are developed for a laminar range of flow, which may extend to a velocity of 1.1 m/s (3.5 ft/s).[1,5,7,8]

If the sludge is pumped long distances, more accurate head loss calculations are needed to design the pumping equipment. Friction losses should be calculated using thixotrophic behavior, rheological concept, and other flow properties of sludge. The procedure involves non-Newtonian fluid and turbulent conditions. Both the Reynolds number and Hedstrom number are needed to calculate the Darcy-Weishbach friction factor. The procedure may be found in Refs 1, 5, 7, 9 and 11.

TABLE 16-3 Summary Description of Sludge Pumps and Their Applications

		Application—Sludge							
Type of Pump	**Description**	**Scum**	**Primary**	**Return**	**Waste Activated**	**Trickling Filter**	**Chemical Precipitation**	**Thickened**	**Digested**
Plunger	Uses pistons or plungers that operate in a cylinder. They are self-priming, and heavy solids concentration may be pumped.	X	X				X	X	X
Torque flow	Consists of a recessed impeller on the side of the casing entirely out of the flow stream. A pumping vortex is set up by viscous drag.		X	X	X	X	X		X
Centrifugal	Consists of impeller enclosed in a casing with inlet and discharge connections. The head is developed principally by centrifugal force.		X	X	X	X	X		

Progressive cavity	The pump is composed of a single-threaded rotor that operates with a minimum of clearance in a double-threaded helix stator made of rubber.	X	X	X	X	X	X	X
Diaphragm	Uses flexible diaphragm or disc fastened over edges of a cylinder	X	X	X	X	X	X	X
High-pressure piston	Uses separate power pistons or membranes to separate the drive mechanisms from contacting the sludge						X	X
Rotary-lobe	Consists of a fixed casing containing gears, vanes, pistons, cams, screws, etc., operating with minimum clearance. The rotating element pushes the liquid around the closed casing into the discharge pipe.	X				X	X	X

TABLE 16-4 Hazen-Williams Roughness Coefficient for Head Loss Calculation for Different Types of Sludges

	Roughness Coefficient C	
Solids in Sludge (%)	**Raw Sludge**	**Digested Sludge**
0	100	100
1	83	100
2	71	91
3	60	83
4	53	78
5	47	73
6	42	69
7	37	65
8	33	60
9	29	55
10	25	48

Piping. Sludge has a tendency to settle and compact in the pipeline and connections. Also, grease will coat the inside of the piping. Therefore, special care is needed to design and construct the sludge piping and pumping equipment.

1. Pressure piping should not be less than 150 mm (6 in.) in diameter unless glass-lined.
2. Gravity piping shall not be less than 200 mm (8 in.) in diameter.
3. Many clean outs and plugged tees or crosses should be provided. Elbows and sharp turns should be avoided.
4. Provision for melting the grease should be made. Heating is achieved by steam, hot water, or digester supernatant.
5. If the sludge flow is small, large-capacity pumping with a timer should be used to flush the line during the pumping cycle. The pump should have sufficient head to move the settled solids.
6. Provision for a high-pressure water jet and pipe rodding and cleaning devices should be made.
7. Long sludge lines should have bypass lines around key sections for cleaning and maintenance.

16-5 PRELIMINARY OPERATIONS

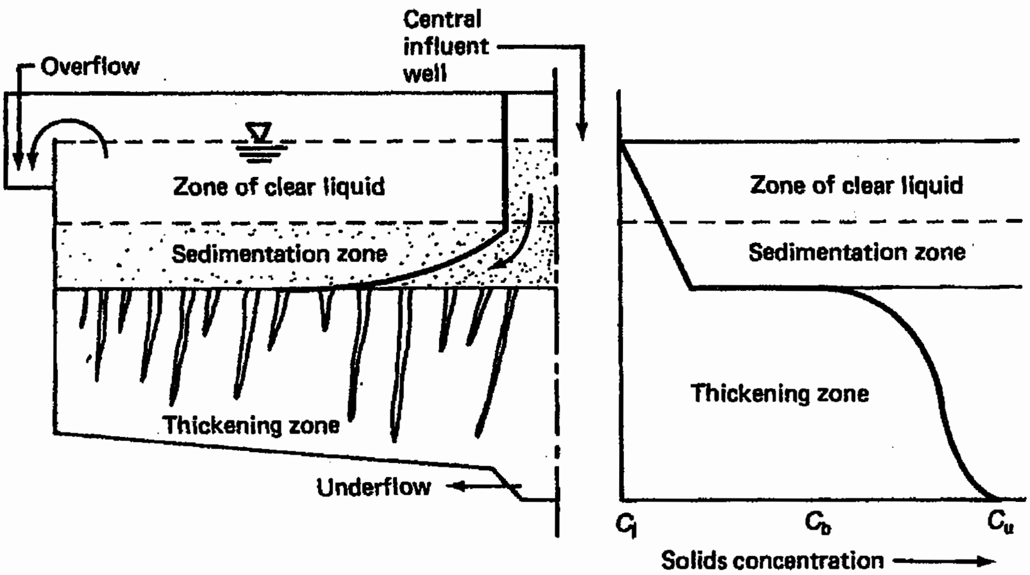
Sludge preparation may often be needed for processing. Common operations are degritting, grinding, and blending. Degritting may be needed if a separate grit removal facility is not provided (see Chapter 11). Grinding may be needed to cut the stringy material that may wrap around the rotating equipment. Grinding is recommended if a progressive cavity pump, solid bowl centrifuges, belt filter press, and heat treatment are used for sludge pumping and processing.[5] Blending is needed if sludge from different processes is combined for processing. The purpose is to homogenize the mixture for improving the performance of the subsequent processes. Design of a sludge-blending facility is covered in the Design Example.

16-6 SLUDGE THICKENING

Sludge contains a large volume of water. Thickening of sludge is used to concentrate solids and reduce the volume. Thickened sludge requires less tank capacity and chemical dosage for stabilization and smaller piping and pumping equipment for transport. Common methods of sludge thickening used at medium to large plants are (1) gravity, (2) flotation, (3) centrifuge, (4) gravity belt, and (5) rotary drum. Each of these methods of thickening are discussed below.

16-6-1 Gravity Thickening

Gravity thickening is accomplished in circular sedimentation basins similar to those used for primary and secondary clarification of liquid wastes. Solids coming to the thickener separate into three distinct zones, as shown in Figure 16-1. The top layer is a relatively clear liquid. The next layer is the sedimentation zone, which usually contains a stream of denser sludge moving from the influent end toward the thickening zone. In the thickening zone, the individual particles of the sludge agglomerate. A sludge blanket is maintained in this zone where the mass of sludge is compressed by material continuously added to the top. Water is squeezed out of interstitial spaces and flows upward to the



C_i - Influent solids concentration
C_b - Lowest concentration at which sludge thickening begins
C_u - Underflow concentration

Figure 16-1 Typical Concentration Profile of Municipal Wastewater Sludge in a Continuous-Flow Gravity Thickener (from Ref. 1).

channels. Deep trusses or vertical pickets are provided to gently stir the sludge blanket and move the gases and liquid toward the surface. The supernatant from the sludge thickener passes over an effluent weir and is returned to the plant. The thickened sludge is withdrawn from the bottom. In this section a discussion on the following topics related to gravity thickening are covered: (a) compression or Type IV settling, (b) process description, (c) design criteria, and (d) equipment.

Compression or Type IV Settling. The settling behavior of particles in the thickening zone is typical of compression or Type IV settling. This topic was briefly introduced in Sec. 11-3. Generally, the concentration of solids in thickener is such that a structure is formed, particles remain supported on top of each other, and further settling can occur by compression from the weight of particles above. The rate of consolidation at any time is proportional to the difference in the depth of the blanket and the maximum consolidated depth in a long period of time. The required compression zone can be determined by running settling tests similar to those for zone-settling tests (Type III). Experiments have shown that breaking the structure in the compression zone enhances thickening. That is why mechanical rakes are provided in the gravity thickeners. A theory of compression settling may be found in several references.[5,12-14]

Process Description. The gravity thickeners provide twofold benefits: (a) sludge concentration and equalization and (b) storage for sludge to enhance downstream operation. Gravity thickeners are commonly used to concentrate solids in sludges from the primary clarifier, trickling filter, and activated sludge. Combined primary and secondary sludges and chemical sludges are also thickened in the gravity thickeners. The primary and lime sludges thicken easily. The presence of biological sludge complicate gravity settling because they settle slowly, resist compaction, and tend to stratify. The degree of thickening may vary from two to five times the concentration of solids in the incoming sludge.

Maximum solids concentration achieved in gravity thickening is normally less than 10 percent. For good operation, a sufficient sludge blanket is maintained. The sludge volume ratio (SVR) is the volume of sludge blanket held in the thickener divided by the volume of the thickened sludge removed per day. The accepted value of SVR is 0.5–2 d. In general, a higher SVR provides thicker sludge. However, excessive retention times may lead to gasification and buoying of the solids.

Design Criteria. The design criteria for gravity thickeners include the following: (1) minimum surface area based on hydraulic and solids loading, (2) thickener depth, and (3) floor slope. Some of the design criteria are summarized in Table 16-5. Generally, the sludge thickeners are designed with a side water depth of 3–4 m (10–14 ft) and a detention period of 24 h. The hydraulic loading rates of 16–32 $m^3/m^2\cdot d$ (380–760 gpd/ft^2) for primary sludge, 4–8 $m^3/m^2\cdot d$ (100–200 gpd/ft^2) for waste activated sludge, and 6–12 $m^3/m^2\cdot d$ (150–300 gpd/ft^2) for combined sludge are used.[12] The thickener overflow rate is important because it relates to solids loading rate. A high hydraulic loading can cause excessive solid carryover. A low loading may create septic conditions. To achieve acceptable hydraulic loading, secondary effluent is blended with the sludge fed into the thickener. The sludge-blending tank may utilize mechanical mixing or air mixing. The

TABLE 16-5 Design Criteria for Gravity Thickeners

Type of Sludge	Influent Solids Concentration (percent)	Thickened Solids Concentration (percent)	Solids Loading ($kg/m^2 \cdot d$)	Solids Capture (percent)	Overflow, TSS (mg/L)
Primary	1.0–7.0	5.0–10.0	90–144	85–98	300–1000
Trickling filter	1.0–4.0	2.0–6.0	35–50	80–92	200–1000
Waste activated sludge	0.2–1.5	2.0–4.0	10–35	60–85	200–1000
Combined primary and waste activated sludge	0.5–2.0	4.0–6.0	25–80	85–92	300–800

$m^3/m^2 \cdot d \times 24.57 = gal/ft^2 \cdot d$.
$kg/m^2 \cdot d \times 0.2048 = lb/ft^2 \cdot d$.
Source: Adapted in part from Refs. 1–3.

gravity thickeners are often a significant odor source and are commonly covered and provided with odor control measures.

Equipment. Gravity thickeners are generally circular concrete tanks with bottoms sloping toward the center. The slope of the bottom floor is generally 2:12 to 3:12, which is steeper than that for standard settling tanks. The equipment includes (1) a rotating bottom scraper arm, (2) vertical pickets, (3) a rotating scum-collecting mechanism with scum baffle plates, and (4) an overflow weir. Other configurations of the tank include circular steel tanks, as well as rectangular concrete and steel tanks. Circular tanks are generally cheaper because of the simplicity of construction, equipment installation, and operation and maintenance.

16-6-2 Dissolved Air Flotation (DAF)

Process Description. Air flotation is primarily used to thicken the solids in chemical and waste activated sludge. Separation of solids is achieved by introducing fine air bubbles into the liquid. The bubbles attach to the particulate matter, which then rise to the surface. In a dissolved air flotation system, the air is dissolved in the incoming sludge under a pressure of several atmospheres. The pressurized flow is discharged into a flotation tank that operates at one atmosphere. Fine air bubbles rise that cause flotation of solids. The principal advantage of flotation over gravity thickening is the ability to remove more rapidly and completely those particles that settle slowly under gravity. The amount of thickening achieved is two to eight times the incoming solids. Maximum concentration of solids in the float may reach 4–5 percent.

Two variations of the dissolved air flotation process include (1) pressurizing all or only a small portion of the incoming sludge and (2) pressurizing the recycled flow from the flotation thickener. The latter method is preferred because it eliminates the need for high-pressure sludge pumps, which are generally associated with maintenance problems.

Chemicals such as alum and iron salts and organic polymers are often added to aid the flotation process.

Design Parameters. The important design and operation parameters for a dissolved air flotation system include (1) air/solids ratio, (2) solids loading rate, (3) hydraulic loading rate, and (4) polymer dosage. These parameters are established by laboratory tests using dissolved air flotation apparatus.[1,12] The air/solids ratio is expressed by Eq. (16-2):

$$\frac{A}{S} = \frac{1.3\, s_a\,(fP - 1)q}{S_a Q} \tag{16-2}$$

where

$\frac{A}{S}$ = air/solids ratio

s_a = solubility of air at the required temperature, mL/L[a]
S_a = solids in incoming sludges, mg/L
f = fraction of air dissolved at pressure P, usually 0.5–0.8

$$P = \text{pressure in atmospheres} = \frac{p + 101.35}{101.35} \text{ (SI units)}$$

$$= \frac{p + 14.7}{14.7} \text{ (U.S. customary units)}$$

p = gauge pressure, kPa (lb/in.^{2} gauge)
q = recycled flow, or a portion of incoming flow pressurized, m^3/d (mgd)
Q = sludge flow to the thickener, m^3/d (mgd)

[a]The values of s_a at temperatures 0, 10, 20, and 30°C are 29.2, 22.8, 18.7, and 15.7 mL/L, respectively.

TABLE 16-6 Typical Ranges of Design Parameters for Dissolved Air Flotation

Type of Sludge	Air/Solids Ratio	Solids Loading Rate (kg/m^2·d)	Hydraulic Loading Rate m^3/m^2·d	Polymer Added (mg/kg)	Solids Captured (Percent)	TSS in Side Stream (mg/L)
Primary	0.04–0.07	90–200	90–250	1000–4000	80–95	100–600
Waste activated sludge	0.03–0.05	50–90	60–180	1000–3000	80–95	100–600
Trickling filter	0.02–0.05	50–120	90–250	1000–3000	90–98	100–600
Primary + waste activated sludge	0.02–0.05	60–150	90–250	1000–4000	90–95	100–600

m^3/m^2·d × 24.57 = gal/ft^2·d.
kg/m^2·d × 0.2048 = lb/ft^2·d.
Source: Adapted in part from Refs. 1–3 and 12.

When the entire flow is pressurized (and recycled flow is zero), $q = Q$. The factor 1.3 is the weight of air in mg/mL. The use of Eq. (16-1) is illustrated in many textbooks.[5,12-14] Typical ranges for various design parameters for a dissolved air flotation thickener are summarized in Table 16-6.

Equipment. The common equipment for a dissolved air flotation thickener includes the following: (1) sludge feed pump with air compressors, (2) flotation tank with skimmer, (3) chemical storage and feed system, and (4) thickened sludge pump. A schematic flow diagram of a dissolved air flotation system pressurizing a portion of the return flow is shown in Figure 16-2.

16-6-3 Centrifugation

Process Description. Centrifugation is a process by which solids are thickened or dewatered from the sludge under the influence of a centrifugal field many times the force of gravity. There are three basic types of centrifuges available for sludge thickening: (1) basket, (2) disc nozzle, and (3) solid bowl (or scroll-type decanter). The basket centrifuge operates on a batch basis. The disc-nozzle centrifuges are the continuous type but require extensive and careful prescreening and grit removal from the sludge. The solid-bowl centrifuges offer continuous operation and have received widespread application in sludge thickening.

Centrifugal thickening of sludge requires high power and high maintenance costs. Use should be limited to plants where space is limited, skilled operation is available, and sludge is difficult to thicken by other means.

Design Parameters. The important design parameters for solid bowl, decanter-type centrifuges are summarized in Table 16-7.

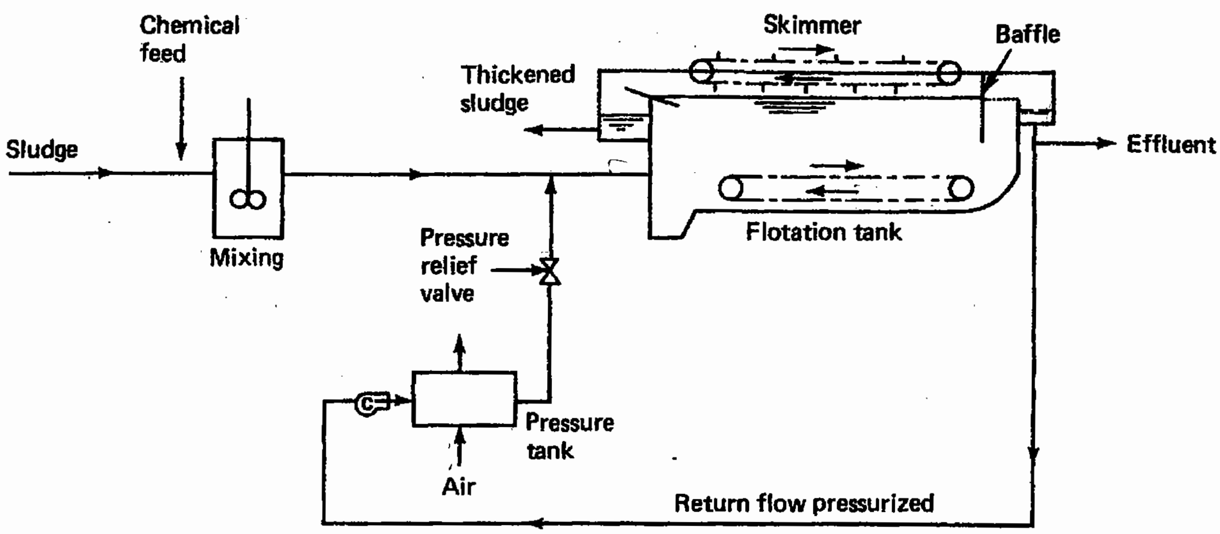


Figure 16-2 Schematic Flow Diagram of Dissolved Air Flotation System Pressurizing a Small Portion of the Effluent.

TABLE 16-7 Design Parameters for Solid Bowl, Decanter-Type Centrifuges

Parameter	Range of Values
Bowl diameter	36–152 cm (14–60 in.)
Capacity	38–600 L/min (10–160 gal/min)
Gravitational force	1400–2300 times gravity
Feed solids, waste activated sludge	0.3–2.0 percent
Thickened solids	5–8 percent
Solids recovery	85–95 percent
Polymer usage	0–3 g/kg dry solids (0–6 lb/ton)

Source: Adapted in part from Refs. 1, 2, 5, and 12.

Equipment. The typical equipment for centrifuge thickening is a centrifuge, sludge feed pump, centrate pump, and thickened sludge pump.

16-6-4 Gravity Belt Thickener

Process Description. The gravity belt thickeners are a recent development in sludge thickening and are used for secondary sludge. It resembles the gravity drainage zone of most belt filters presses used for dewatering sludge. The sludge becomes concentrated as the free water drains by gravity through a porous horizontal belt. Chemically conditioned sludge enters through an inlet gate and is dispersed evenly over the moving belt. As a mat or solid layer builds up over the belt, rows of plows or vans expose the belt area for more water to drain. Sludge pooling and backward rolling action may also be used to release more free water from the sludge. The sludge travels over the ramp into a sludge hopper. The solids are scraped and fabric is washed with a water jet. The process diagram of a gravity belt thickener is shown in Figure 16-3.

Design Parameters. The polymer addition is 1.5–4.5 kg/metric ton (3–10 lb/ton).[12] Thickened sludge may reach 4–8 percent solid, and solid capture efficiency may reach 90–98 percent. The hydraulic loading on a gravity belt thickener is 6–16 L/s per meter belt width for influent sludge having solid a concentration of 0.5–1 percent.

Equipment. The typical equipment for a gravity belt thickener is a polymer feed and mixing system, flocculation basin, belt thickener assembly with plows or vanes, thickened sludge hopper, belt scraper, and wash station.

16-6-5 Rotary Drum Thickener

Process Description. The rotary drum thickener uses media-covered drums that rotate slowly. The conditioned secondary sludge enters the drum, and free water drains by

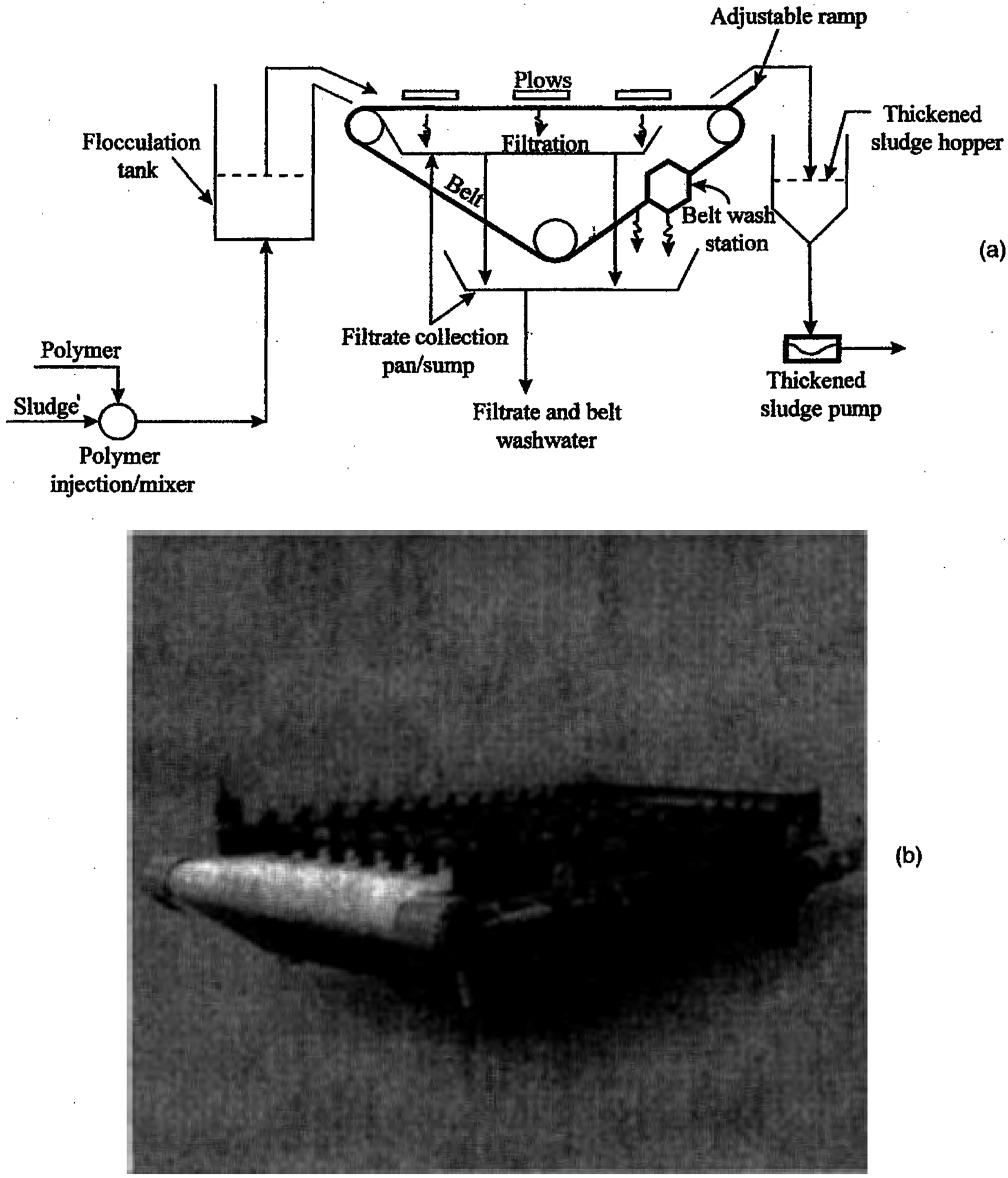


Figure 16-3 Mechanical Gravity Thickener: (a) schematic of gravity belt thickener and (b) photograph of gravity table thickener (courtesy Andritz).

gravity, similar to a belt thickener. The sludge is conveyed along the drum length by a continuous internal screw or diverted angle flights and exits through a discharge chute. Washwater periodically flushes the inside and outside of the drum to clean the screen openings. Rotary drum thickeners are often used as a prethickening step with belt filter press dewatering. A rotary drum screen thickener is shown in Figure 16-4.

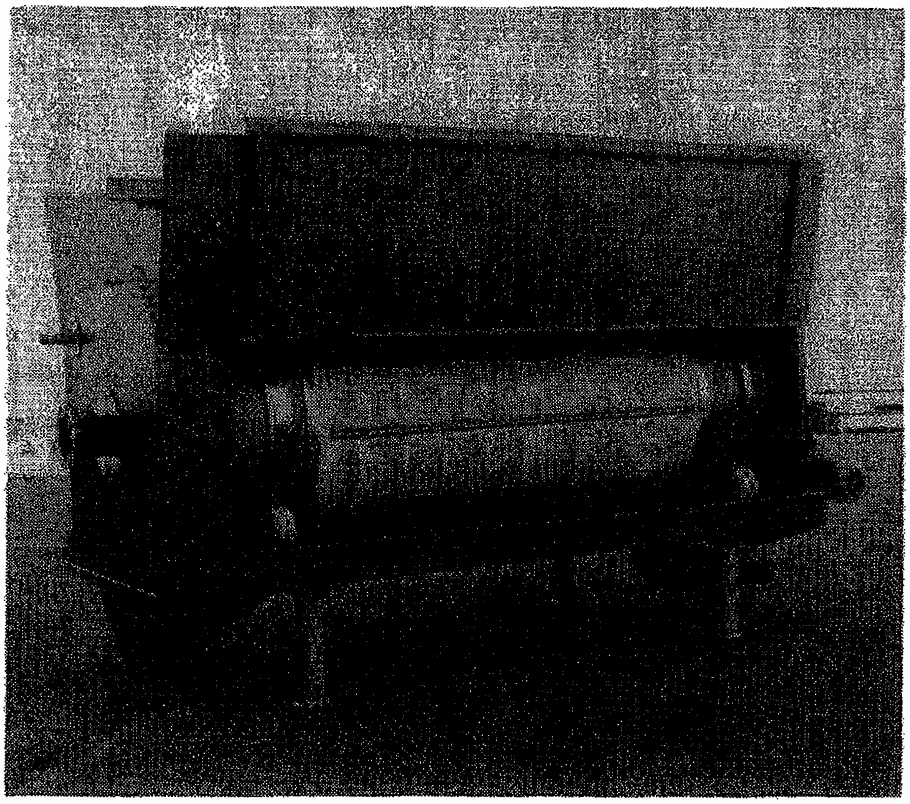


Figure 16-4 Photograph of Rotary Drum Screen Thickener (courtesy Andritz).

Design Parameters. The rotating speed of the drum is 5–20 rpm. Solid concentration in the thickened sludge may reach 3–4 percent. The amount of conditioning chemicals may be high. The solid capture is the same as in the gravity belt thickener. The basic design considerations include control of adjustable sludge and polymer feed rates, proper equipment drainage, and washwater supply for screen cleaning.

Equipment. The equipment for a rotary drum thickener includes a rotating drum with wedge wires, internal screw or diverted angle flights, perforations, stainless steel fabric, polyester fabric, or a combination. Like in gravity belt thickeners, chemical mixing and flocculating equipment, variable-speed drive to rotate the drum, filtrate drums, thickened sludge conveyor, and filter wash system are also necessary equipment.

16-6-6 Thickening Process Evaluation and Comparison

Each thickening process discussed above has many advantages and disadvantages. Their selection for a particular application warrants special consideration. Based on many design performance and economic criteria, various sludge thickening processes have been evaluated, and their comparative performance is listed in Table 16-8.

16-7 EQUIPMENT MANUFACTURERS

The equipment manufacturers of gravity thickeners, air flotation systems, and centrifuges are listed in Appendix D. Each equipment component must be evaluated for com-

TABLE 16-8 Comparative Evaluation of Different Sludge Thickening Processes

Evaluation Criteria	Gravity	Dissolved Air Flotation	Centrifugation	Gravity Belt	Rotating Drum
Space requirement	High	Medium	Low	Medium	Medium
Operation and maintenance	Simple	Medium	High	Medium	Medium
Applicable to	Primary and combined	WAS	WAS	WAS	WAS
Conditioning chemicals	None	High	High	Medium	Medium
Power requirement	Low	High	High	Medium	Medium
Capital cost	Low	High	High	Medium	Medium
Operation cost	Low	High	High	Medium	Medium
Thickened sludge solids concentration	Medium	Low	High	Medium to high	Medium to high
Building corrosion problem if enclosed	High	Medium	None	Medium	Medium
Odor problem	Serious	Moderate	Low	Moderate	Moderate

Source: Ref. 12.

patibility, operational flexibility, maintenance requirements, and design criteria. Equipment selection considerations and responsibilities of the design engineers are covered in Sec. 2-10.

16-8 INFORMATION CHECKLIST FOR THICKENER DESIGN

Before designing a thickener, the design engineer must develop design data and make many important decisions. Adherence to a carefully developed data for thickener design will minimize project delays and loss of engineer's time in redesigning the units and equipment. A checklist of such design activities is presented below:

1. Conduct material mass balance analysis at average daily design flow, and establish characteristics of primary, secondary, or any other type of sludges reaching the thickener.
2. If primary, secondary, or other treatment facilities have been designed, use the actual quantities and solids concentrations of the sludges reaching the thickeners.
3. Increase the quantities of sludges produced under average daily flow conditions by a certain factor to accommodate the sustained peak flow conditions reaching the plant. The increase in sludge production is normally assumed to be 5–20 percent.
4. Select the thickening processes for different types of sludge. Dissolved air flotation for thickening is generally suited for waste activated sludge, chemical sludges, or sol-

ids that settle slowly. On the other hand, gravity thickening is simple, economical to operate, and works well with primary and combined sludges. Centrifugal thickening is effective but requires high power and maintenance costs. Gravity belt and rotating drum filters are suited for waste activated and chemical sludges. Table 16-8 provides process evaluation and selection criteria. At medium-sized secondary treatment facilities, the gravity thickening of combined sludges is normally most cost-effective.

5. Develop design parameters such as solids and hydraulic loading rates, air/solids ratio, tank geometry, etc. Laboratory studies may be needed to develop the design parameters.
6. Obtain the design criteria from the concerned regulatory agency.
7. Select equipment manufacturers and equipment selection guides.

16-9 DESIGN EXAMPLE

16-9-1 Design Criteria Used

The following design assumptions and criteria are used for the design of thickeners:

1. Provide two gravity thickeners for thickening of combined primary and waste activated sludge.
2. Design the combined sludge thickener for the following loadings:
   a. Solids loading not to exceed 47 $kg/m^2{\cdot}d$ (9.6 $lb/ft^2{\cdot}d$)
   b. Hydraulic loading not less than 9.0 $m^3/m^2{\cdot}d$ (221 $gal/ft^2{\cdot}d$)
3. The design flow and combined sludge solids are summarized in Table 16-9. It may be noted that these quantities are 4–6 percent higher than the values obtained from the material mass balance analysis (Table 13-13) performed at average design daily flow. These increases are necessary to allow for the maximum sustained loading conditions that may occur at the plant. Procedure and justification for these allowances are also indicated in Table 16-9.
4. The influent structure shall consist of an influent well that will receive the primary and secondary sludges and plant effluent for blending to maintain proper solids consistency and hydraulic loading. The blended sludge shall be pumped into the central well of each thickener.
5. The effluent structure for thickener overflow shall consist of V-notches, effluent launder, and a common sump. The design of effluent structure shall be similar to that of the final clarifier described in Chapter 13. The thickener overflow shall be returned to the aeration basin.
6. The thickened sludge from both thickeners shall be pumped into the anaerobic digesters.

16-9-2 Design Calculations

Step A: Thickener Area and Diameter

1. Compute the surface area of the thickener based on solids loading.

TABLE 16-9 Characteristics of Primary, Secondary, and Combined Sludges for Design of Gravity Thickener

Types of Sludge	Dry Solids (kg/d)	Solids (percent)	Flow (m^3/d)
A. Average design condition developed from material mass balance analysis are given in Chapters 12 and 13.			
Primary sludge (Table 12-7)	6227	4.5	134
Waste activated sludge, including precipitated phosphorus and $Al(OH)_3$ (Table 13-13)	3385	0.45	752
Combined sludge[a]	9611	1.06	886
B. Peak design condition with allowance for peak sustained loadings			
Primary sludge	6643[b]	4.5	143[c]
Waste activated sludge, including precipitated phosphorus and $Al(OH)_3$	3558[d]	0.46	759[d]
Combined sludge[a]	10201[e]	1.09	902[f]

[a]Specific gravity of combined sludge = 1.02.
[b]Dry solids have been increased by 7 percent of the quantity of primary sludge produced at average daily design flow.

[c]Volume of primary sludge = $\dfrac{6643 \text{ kg/d}}{0.045 \text{ g/g} \times 1.03 \times 1000 \text{ kg/m}^3} = 143 \text{ m}^3\text{/d}$.

[d]The quantity of waste activated sludge was calculated in Sec. 13-11-5, Step E, 5. This quantity was obtained after increasing the influent flow and BOD_5 to the aeration basin. Therefore, no additional allowance for waste activated sludge under sustained maximum loading conditions was necessary.
[e]The quantity of solids in combined sludge is 6.0 percent higher than that obtained by material mass balance analysis.
[f]The volume of combined sludge is 1.8 percent higher than that at average design flow.

The surface area is computed using the solids loading and hydraulic loading:

$$\text{Total surface area for thickening at solids loading of 47 kg/m}^2\text{·d} = \frac{10{,}201 \text{ kg/d}}{47 \text{ kg/m}^2\text{·d}} = 217.0 \text{ m}^2$$

2. Compute the hydraulic loading and volume of dilution water.

$$\text{Hydraulic loading} = \frac{\text{sludge volume/d}}{\text{surface area}}$$

$$= \frac{902 \text{ m}^3\text{/d}}{217.0 \text{ m}^2} = 4.16 \text{ m}^3\text{/m}^2\text{·d}$$

This is significantly lower than the normal range of the recommended hydraulic rate for gravity thickeners for combined sludges. A decrease in surface area will exceed the solids loading outside the acceptable limit. Therefore, dilution water must be blended with the incoming sludge. Quantity of dilution water added to achieve a hydraulic loading greater than 9.0 $m^3/m^2{\cdot}d$ is calculated as follows. Start with an assumption that hydraulic loading = 9.8 $m^3/m^2{\cdot}d$.

$$\begin{aligned}
\text{Total flow to the thickener} &= 9.8\ m^3/m^2{\cdot}d \times 217.0\ m^2 = 2127\ m^3/d \\
\text{Combined sludge flow} &= 902\ m^3/d \\
\text{Dilution water needed} &= 2127\ m^3/d - 902\ m^3/d = 1225\ m^3/d
\end{aligned}$$

$$\begin{aligned}
\begin{array}{l}\text{Total solids concentration}\\ \text{in the blended sludge}\end{array} &= \frac{10{,}201\ kg/d}{1.01 \times 1000\ kg/m^3 \times 2127\ m^3/d} \\
&= 0.0048 \\
&= 0.0048 \times 100 = 0.48\ \text{percent}
\end{aligned}$$

3. Compute TSS and volume of the blended sludge.
The mass loads of TSS in the blended sludge will also include TSS in the dilution water. The effluent is used as dilution water. Therefore, the concentration of TSS in dilution water will be used for this purpose.

$$\begin{aligned}
\text{TSS in dilution water} &= 10\ g/m^3 \times 1225\ m^3/d \times 1000^{-1}\ kg/g \\
&= 12.3\ kg/d \\
\text{TSS in blended sludge} &= 10{,}201\ kg/d + 12\ kg/d = 10{,}213\ kg/d \\
\text{Total volume of blended sludge} &= 902\ m^3/d + 1225\ m^3/d = 2127\ m^3/d
\end{aligned}$$

$$\begin{aligned}
\begin{array}{l}\text{Concentration of TSS in blended}\\ \text{sludge}^{b}\end{array} &= \frac{10{,}213\ kg/d}{2127\ m^3/d \times 1010\ kg/m^3} \\
&= 0.48\ \text{percent}
\end{aligned}$$

4. Select the geometry of the gravity thickener.
Provide two circular thickeners.

$$\text{Area of each thickener} = \frac{217.0\ m^2}{2} = 108.5\ m^2$$

$$\text{Diameter of each thickener} = \sqrt{\frac{4}{\pi} \times 108.5\ m^2} = 11.75\ m\ (38.6\ ft)$$

Provide two thickeners each of 12.2 m (40 ft) diameter.[c]

$$\text{Surface area of each thickener} = \frac{\pi}{4}(12.2)^2 = 116.9\ m^2$$

[b]Specific gravity of blended sludge is assumed to be 1.01.

[c]Manufacturers usually supply equipment in 2-ft increments of diameter.

5. Check solids and hydraulic loading at peak condition when both thickeners are operating.

$$\text{Solids loading when both units are operating} = \frac{10{,}213 \text{ kg/d}}{2 \times 116.9 \text{ m}^2} = 43.7 \text{ kg/m}^2\cdot\text{d}$$

$$\text{Hydraulic loading when both units are operating} = \frac{2127 \text{ m}^3\text{/d}}{2 \times 116.9 \text{ m}^2} = 9.1 \text{ m}^3\text{/m}^2\cdot\text{d}$$

6. Check solids and hydraulic loadings at average flow when one thickener is out of service.
   With only one unit in service and at average daily design flows, the solids and hydraulic loadings are calculated as follows:

$$\text{Solids loading}^{d} = \frac{9612 \text{ kg/d}}{116.9 \text{ m}^2} = 82.2 \text{ kg/m}^2\cdot\text{d}$$

$$\text{The proportionate blended flow to the thickener at average design flow} = \frac{2127 \text{ m}^3\text{/d}}{10201 \text{ kg/d}} \times 9611 \text{ kg/d}$$

$$= 2004 \text{ m}^3\text{/d}$$

$$\text{Hydraulic loading}^{e} = \frac{2004 \text{ m}^3\text{/d}}{116.9 \text{ m}^2} = 17.1 \text{ m}^3\text{/m}^2\cdot\text{d}$$

Step B: Thickener Depth. The total sidewater depth of the gravity thickener comprises three separate zones: clear liquid zone, the settling zone, and the thickening zone. Provide a freeboard of 0.6 m (2 ft). Generally, in gravity thickeners the clear liquid zone of 0.5–1.0 m (1.6–3 ft) and a settling zone of 1.5–2.0 m (4.6–6.7 ft) are considered sufficient.

The thickening zone is generally sized to allow less than 1 d detention time for the sludge. An estimate of average sludge concentration must be made.

1. Determine the solids concentration in the thickener at the upper part of the thickening zone.
   The solids concentration in the combined and blended sludge = 0.48 percent. Assume that the blended sludge reaches its original solids concentration of 1.09 percent at the upper part of the thickening zone (see Table 16-9).
2. Determine the solids concentration at the bottom of the thickened zone.
   The desired concentration of thickened sludge is at least 6 percent. The concentration of solids at the bottom of thickened zone is therefore 6 percent.

[d] The solids at average daily design flow is obtained from Table 16-9.

[e] The volume of dilution water is reduced from 1225 m^3/d to 1118 m^3/d at average daily design flow (1118 m^3/d = (2004 − 886) m^3/d, where 2004 m^3/d is total flow to the thickener and 886 m^3/d is the average volume of combined sludge from Table 16-9.

3. Compute the average sludge concentration in the thickening zone.

$$\text{Percent average solids concentration} = \frac{1.09 + 6.00}{2} = 3.55 \text{ percent}$$

4. Compute the depth of the thickening zone for an average solids retention time of 0.8 d.

Assume the depth of thickening zone	$= h$ (m)
Volume of sludge blanket per thickener	$= \frac{\pi}{4}(12.2 \text{ m})^2 h$ (m)
	$= 116.9\ h$ (m^3)
Amount of solids in the thickening zone at 3.58 percent solids	$= (116.9\ h)$ m$^3 \times 0.0355$ g/g $\times 1.03 \times 1000$ kg/m^3
	$= 4275\ h$ (kg)
Quantity of solids held in the thickening zone per thickener	$= \frac{10{,}213}{2}$ kg/d $= 5107$ kg/d

At 0.8 day of solids retention period,

$$\frac{(4275\ h)\text{ kg}}{5107\text{ kg/d}} = 0.8\text{ d}$$

$$h = \frac{5107\text{ kg/d} \times 0.8\text{ d}}{4275\text{ kg}} = 1.0\text{ m (3.3 ft)}$$

Provide a 1.0-m depth of thickening zone. Additional storage will be available in the bottom cone of the thickener. In addition, under sustained loadings or equipment downtime periods, excess storage capacity will be available in the settling zone of the thickener that has a liberal allowance for storage.

Total side water depth of the thickener is as follows:

Clear liquid zone	1.0 m (3.3 ft)
Settling zone	1.9 m (6.2 ft)
Thickening zone	1.0 m (3.3 ft)
Total liquid depth	3.9 m (12.8 ft)
Provide a freeboard of 0.6 m (2.0 ft)	

5. Compute the depth of thickener at the central well.
The bottom slope for most thickeners with central well sludge withdrawal and with sludge scrapers is 17 cm/m (2:12).

$$\text{Therefore total drop to the central well} = \frac{17\ \text{cm/m}}{100\ \text{cm/m}} \times \frac{12.2\ \text{m}}{2}\ (\text{radius}) = 1.0\ \text{m}$$

The depth of the thickener at the central well = 4.5 m + 1 m = 5.5 m (18.1 ft). The design details of gravity thickener are given in Figure 16-5.

6. Compute the volume of the central well (cone).

$$\text{Volume} = \frac{1}{3}\ \text{depth} \times \text{top area of the cone}$$

$$= \frac{1}{3} \times 1.0\ \text{m} \times \frac{\pi}{4}\ (12.2\ \text{m})^2 = 39.0\ \text{m}^3$$

Step C: Blending Tank. The primary and waste activated sludge must be blended thoroughly to achieve a consistent feed to the thickener. The blending unit also provides a convenient place to meter dilution water and add pH adjusters, thickening aids, flocculants, and other chemicals. The size of the blending tank depends on the pumping schedule of primary and waste activated sludge and dilution water.

1. Compute the dimensions of the blending tank.
   Provide sludge storage and a blending period of 1.75 h under peak design sludge loading.

$$\text{Quantity of primary and waste activated sludge and dilution water} = 2127\ \text{m}^3/\text{d}$$

$$\text{Volume of blending tank} = 2127\ \text{m}^3/\text{d} \times \frac{1.75\ \text{h}}{24\ \text{h/d}}$$

$$= 155.1\ \text{m}^3$$

$$\text{Provide liquid depth} = 3\ \text{m}$$

$$\text{Provide freeboard} = 0.6\ \text{m}$$

$$\text{Area of the blending tank} = \frac{155.1\ \text{m}^3}{3\ \text{m}} = 51.7\ \text{m}^2$$

$$\text{Diameter of the blending tank} = \sqrt{51.7\ \text{m}^2 \times \frac{4}{\pi}} = 8.1\ \text{m}$$

   Select an 8.2-m (27-ft) diameter and 3-m (9.8-ft) deep blending tank. Design details of the blending tank are shown in Figure 16-6.

2. Select sludge-mixing and -blending system.
   Mixing keeps solids in suspension and blends primary and waste activated sludge. Mixing may be accomplished by (1) coarse air diffusers, (2) liquid recirculation, or (3) mechanical paddles. Diffuse air mixing has the advantage of freshening the sludge that will minimize odors in the thickener dome and may improve thickening. However, because of odor problems, the blending tank should be covered, and the captured air should be scrubbed.

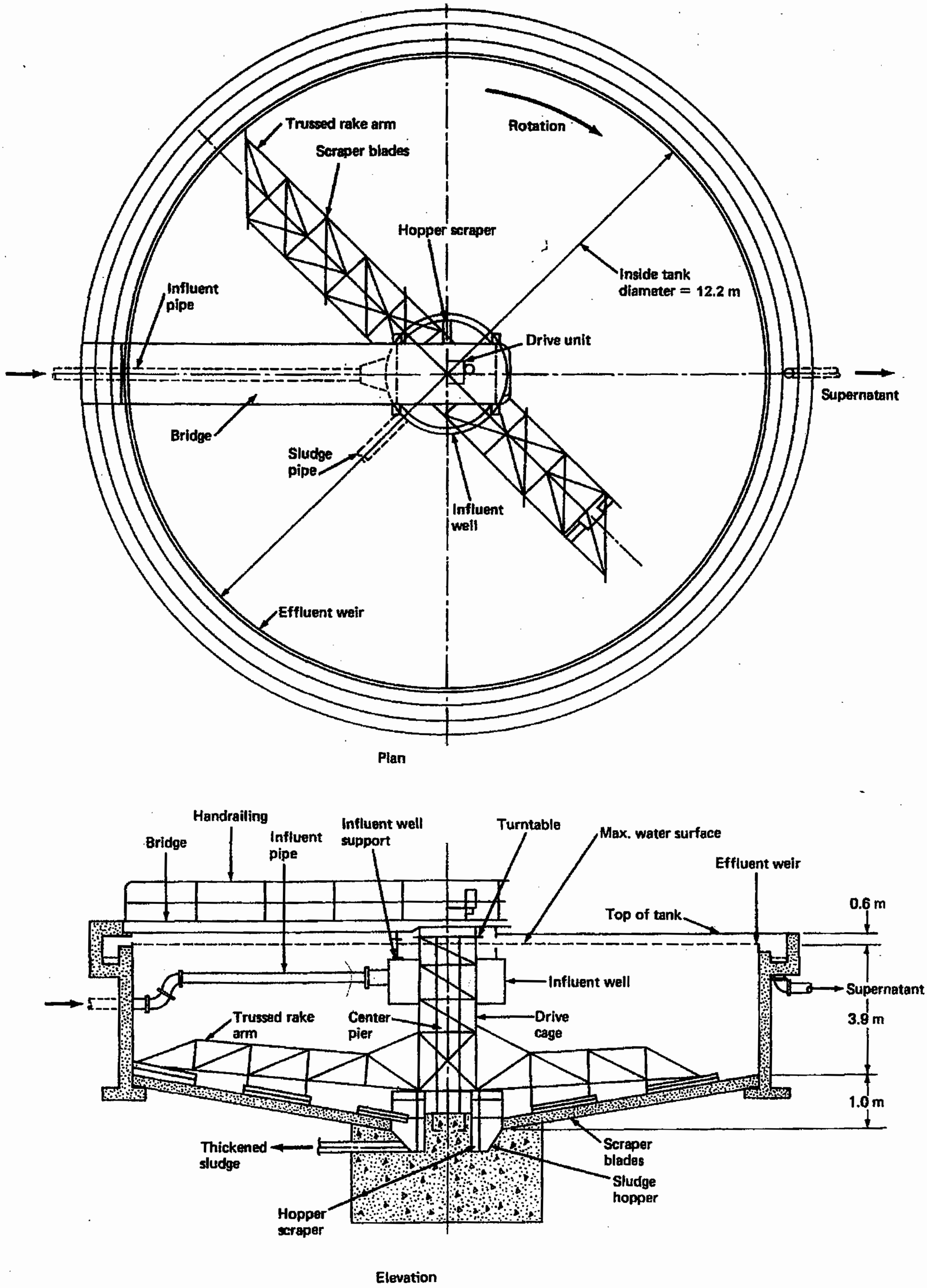


Figure 16-5 Design Details of Gravity Thickener.

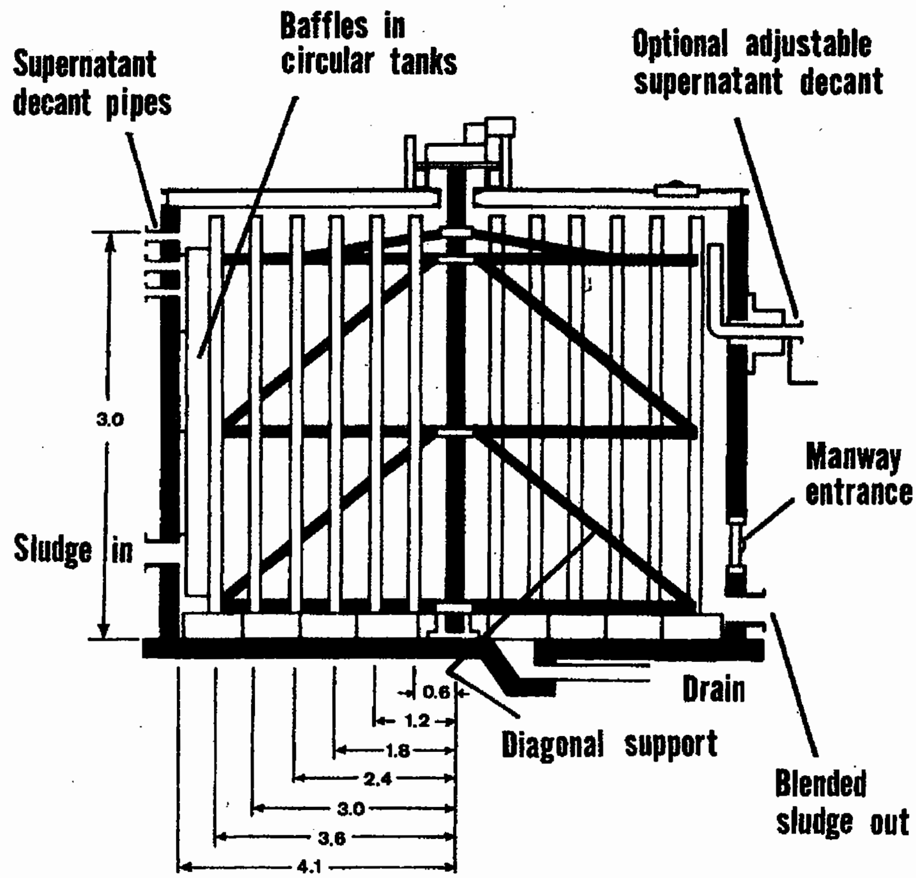


Figure 16-6 Details of Sludge-Blending Tank and Blender Mixer (courtesy EIMCO Process Equipment Company). All Dimensions Are in Meters.

The design of mixing equipment by gas and sludge recirculation is given in Chapter 17. The same design procedure can also be used for the blending tank. In this design, however, a mechanical paddle mixer is provided to illustrate the procedure of a different design.

3. Select the design equations.
   Equations (12-6) and (12-10) are commonly used equations for design of flocculation paddles. Readers are referred to Chapter 12 for theory and design of flocculation.
4. Compute the power requirement.

$$
\begin{aligned}
\text{Volume of blending tank } V &= 158 \text{ m}^3 \\
\text{Assume } G &= 60/\text{s} \\
\text{Assume } \mu &= 2 \text{ times the viscosity of water at } 20^\circ\text{C} \\
&= 2 \times 1.002 \times 10^{-3} \text{ N s/m}^2 \\
&= 2.004 \times 10^{-3} \text{ N s/m}^2 \\
P &= G^2 \mu V \\
P &= (60/\text{s})^2 \times 2.004 \times 10^{-3} \text{ N s/m}^2 \times 158 \text{ m}^3 \\
&= 1140 \text{ N m/s} = 1140 \text{ W} \\
&= 1.14 \text{ kW } (1.53 \text{ hp}) \\
\text{Motor power at 75 percent efficiency} &= 1.14 \text{ kW}/0.75 = 1.5 \text{ kW}
\end{aligned}
$$

5. Compute the area and dimensions of the paddles.
Provide 12 vertical flat paddles (six on each side of the central shaft, as shown in Figure 16-6). The distances to the middle of the paddles from the center of the column are 3.6, 3.0, 2.4, 1.8, 1.2, and 0.6 m.

Assume rotational speed of the paddle shaft, $n = 0.06$ revolutions/s. Average paddle speed $v_p = 2\pi \times$ distance from center $\times n$.

$$P = \frac{1}{2} C_D \ \rho A v^3$$

If a = area of each vertical paddle and v is its relative velocity, $P = \frac{1}{2} C_D \rho \Sigma a v^3$ using $C_D = 1.8$.

$$\rho = 1.01^{f} \times 1 \text{ g/cm}^3 \times 10^6 \text{ cm}^3/\text{m}^3 \ (1000 \text{ g/kg})^{-1} = 1010 \text{ kg/m}^3$$

$$v = 0.75 \ v_p$$

$$\begin{aligned} P = \tfrac{1}{2} C_D \ \rho \ [&2a(0.75\pi \times n \times 3.6 \times 2)^3 + 2a \ (0.75\pi \times n \times 3.0 \times 2)^3 \\ &+ 2a(0.75\pi \times n \times 2.4 \times 2)^3 + 2a(0.75\pi \times n \ \times 1.8 \times 2)^3 \\ &+ 2a(0.75\pi \times n \times 1.2 \times 2)^3 + 2a(0.75\pi \times n \times 0.6 \times 2)^3] \end{aligned}$$

$$\begin{aligned} P &= \tfrac{1}{2} \times 1.8\rho \times 2a(0.75\pi \times 0.06 \times 2)^3 \ (3.6^3 + 3.0^3 + 2.4^3 + 1.8^3 + 1.2^3 + 0.6^3) \\ &= \tfrac{1}{2} \times 1.8 \times 1010 \times 2a(0.023)(95.256) \end{aligned}$$

$$1140 = 3983a$$

$$a = 0.29 \text{ m}^2$$

The height of the paddle = 2.5 m
Width of the paddle = $0.29 \text{ m}^2/2.5 \text{ m} = 0.12$ m (4.7 in.)

Provide 12 redwood vertical paddles, each 2.5 m $\times$ 12 cm flat. The area of the diagonal support is ignored in this design.

6. Select the pumping arrangement.
The primary and waste activated sludges shall be pumped from their respective areas to the sludge-blending tank. Design and operational details of the primary sludge and waste activated sludge pumps are given in Chapters 12 and 13, respectively.

The dilution water shall be obtained from the chlorine contact basin.[g] Two constant-speed centrifugal pumps shall be provided. Each pump shall be synchronized with the operation of the primary sludge pump. Thus, the volume of the dilution water reaching the blending tank shall be proportional to the flow of primary sludge.

The blended sludge shall be pumped from the blending tank to the thickeners. Two identical constant-speed centrifugal pumps (one for each thickener) shall be provided to transfer blended sludge to the thickener. An additional pump shall be provided as a standby unit. The control system shall include high-level start and low-level stop and an extra high-level alarm (see Chapter 9 for more details). Metering for all pumps shall be provided in order to control the blending process and feed rate into the thickeners. Figure 16-7 is a schematic layout of the blending and thickening area.

[f]Specific gravity of the blended sludge.

[g]Many designers have successfully utilized raw wastewater, primary or secondary effluents, or side streams from the sludge-processing areas as dilution water.

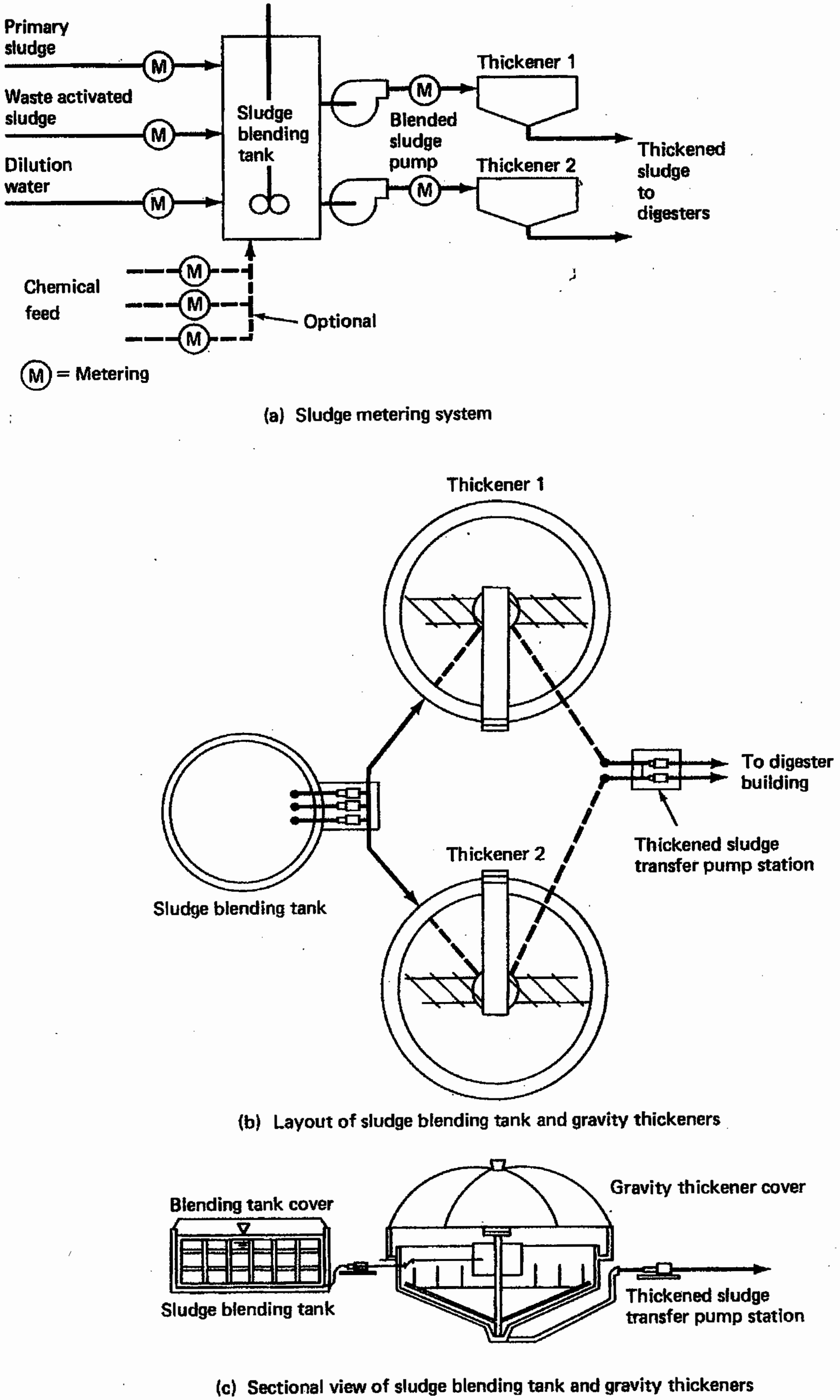


Figure 16-7 Schematic of Metering and Layout of Sludge-Blending and -Thickening Facilities.

Step D: Influent Structure. The influent structure to the thickener consists of a central well similar to that of the final clarifier. Design details are given in Figure 16-5.

Step E: Effluent Structure. The effluent structure consists of V-notch weirs around the periphery, effluent launder, and outlet pipe. The entire arrangement is similar to that of the final clarifiers. The design procedure for V-notch, effluent launder, outlet pipe, and hydraulic profile may be found in Chapters 12 and 13.

The effluent will be discharged under gravity into a sump. In this sump, the supernatants from digesters and dewatering facilities will also be collected. The combined returned flow will be pumped to the aeration basin. The dimensions of the sump are arbitrary. Provide a 6 m × 6 m × 4 m (deep) sump. The bottom of the sump shall be sloped to provide a hopper at the pump suction.

Step F: Characteristics of Blended Sludge. The blended sludge includes (a) primary sludge, (b) WAS after phosphorus stripping, and (c) TSS added by the dilution water. The flow and TSS content in the blended sludge is calculated in Sec. 13-11-4, Step A, 6. More detailed characteristics of blended sludge in terms of TSS, BOD_5, various forms of nitrogen, and total phosphorus are calculated in the material mass balance analysis (Sec. 13-11-4, Step A, 6, c), and final results are summarized in Table 13-13. These values, along with those for other streams, are provided in Table 16-10 and are compared with those developed in Sec. 16-9-2, Step A, 3.

TABLE 16-10 Characteristics of Combined Blended Sludge, Thickened Sludge, and Thickener Overflow Developed in Material Mass Balance Analysis (Table 13-13)

Parameter	Combined Blended Sludge	Thickened Sludge	Thickener Overflow
Flow, m^3/d	2011 (2127)	132	1878 (1987)
TSS, kg/d	9623 (10,213)	8180 (8681)	1444 (1532)
BOD_5, kg/d	5003	4253	750 (799)
Org.-N, kg/d	468	398	70
NH_4^+-N, kg/d	4.4	0.3	4.1
TN, kg/d	482	398	74
TP, kg/d	214	201	13
TVSS/TSS	0.71	0.71	
Biodegradable solids/TSS	0.54	0.54	
Org.-N/TVSS	0.068	0.68	
TP/TVSS	0.031	0.031	

Note: (1) The values in parentheses are calculated in Chapter 16 and are higher because of sustained loading effects. (2) The concentration mg/L or kg/m^3 can be calculated from mass/d and flow m^3/d. For example, TSS concentration in thickener overflow $= 1444\ \text{kg/d} \times \dfrac{1}{1878\ \text{m}^3/\text{d}} \times 1000\ \text{g/kg} = 769\ \text{mg/L}$ (see Table 13-13, Stream 11).

Step G: Characteristics of Thickened Sludge

1. Compute the quantity of thickened sludge.

Quantity of sludge reaching both thickeners = 10,213 kg/d
Solids capture efficiency = 85 percent
Quantity of solids withdrawn from both thickeners = 10,213 kg/d × 0.85

= 8681 kg/d

Quantity of solids withdrawn from each thickener $= \frac{8681}{2}$ kg/d = 4341 kg/d

2. Compute the mass of other constituents in thickened sludge.
   The mass loads of BOD_5 and various components of nitrogen and phosphorus and several useful ratios have been calculated in the mass balance analysis (Sec. 13-11-4, Step A, 7). The final results are summarized in Table 13-13. These values, along with those for other streams, are compared in Table 16-10. There will be a slight change in these values because flow and TSS loads were increased for the design of BNR system and gravity thickener.
3. Compute the thickened sludge-pumping rate.

Volume of thickened sludge-pumping rate from both thickeners at 6 percent solids
$$= \frac{8681 \text{ kg/d}}{0.06 \text{ g/g} \times 1.03 \times 1000 \text{ kg/m}^3} = 140.5 \text{ m}^3\text{/d}$$

Pumping rate from each thickener = 70.3 m^3/d

4. Select thickened sludge pumps.
   Provide one plunger-type pump for each thickener to transfer thickened sludge to the digester. Each pump shall have a variable time clock control to pump for 5 min at 96-min cycle time. The constant pumping rate shall be approximately 0.85 m^3/min (224 gpm). Each sludge pump shall have cross-connections such that if one pump fails, the other pump shall serve both thickeners. Each pump will have a low-level override to stop the pumping operation if the water surface in the thickener drops below a certain predetermined level.
5. Check the sludge volume ratio (SVR).
   The sludge volume ratio (SVR) is the volume of the sludge blanket held in the thickener divided by the volume of the thickened sludge removed per day.

Volume of sludge blanket held in each thickener (thickening zone + central cone)
$$= \frac{\pi}{4} (12.2 \text{ m})^2 \times 1.0 \text{ m} + 39.0 \text{ m}^3$$
$$= 155.9 \text{ m}^3$$

Volume of thickened sludge withdrawn per thickener = 72.3 m^3/d

SVR
$$= \frac{155.9 \text{ m}^3}{72.3 \text{ m}^3\text{/d}} = 2.2 \text{ d}$$

This is slightly higher than the value generally used for normal operation of the thickener.

Step H: Quality of the Supernatant from the Thickener Overflow

1. Compute the volume of thickener overflow.

$$\text{Average volume of thickener overflow}^{h} = 2127\ \text{m}^3/\text{d} - 2 \times 70.2\ \text{m}^3/\text{d}$$

$$= 1987\ \text{m}^3/\text{d}$$

2. Compute the concentration of solids in the thickener overflow.

$$\text{Amount of TSS lost in the thickener overflow}^{h} = 10{,}213\ \text{kg/d} \times 0.15$$

$$= 1532\ \text{kg/d}$$

$$\text{TSS concentration}^{h} = \frac{1532\ \text{kg/d} \times 1000\ \text{g/kg} \times 1000\ \text{mg/g}}{1987\ \text{m}^3/\text{d} \times 1000\ \text{L/m}^3}$$

$$= 771\ \text{mg/L}$$

3. Compute the mass of BOD_5 in the thickener overflow.

$$BOD_5 = 1532\ \text{kg/d} \times 0.54\ \text{g biodegradable solids/g TSS (Table 16-10)}$$

$$\times 1.42\ \text{g}\ \frac{BOD_L \times 0.68\ BOD/BOD_L}{\text{g biodegradable solids}}$$

$$= 799\ \text{kg/d}$$

$$\begin{array}{c}BOD_5\\ \text{concentration}\end{array} = \frac{799\ \text{kg/d} \times 1000\ \text{g/kg} \times 1000\ \text{mg/g}}{1987\ \text{m}^3/\text{d} \times 1000\ \text{L/m}^3} = 402\ \text{mg/L}$$

4. Compute the concentration of other constituents in the thickener overflow.
   The concentrations of various components of nitrogen and phosphorus in the thickener overflow are calculated in mass balance analysis (Sec. 13-11-4, Step A, 8). The final results are summarized in Table 13-13. These values and those for other thickener streams are provided in Table 16-9. There will be a slight change in these values because flow and TSS load were increased for the design of the BNR system and gravity thickeners.[15,16]

Step I: Design Details. The design details of gravity thickener are given in Figure 16-5.

[h]The values of thickener overflow calculated here are slightly higher than those developed in the material mass balance analysis (Table 13-13). Higher values are caused by an increase in flow and mass loads used for the design of the BNR system and gravity thickener.

16-10 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT SLUDGE-THICKENING FACILITY

Gravity sludge thickening can create serious odors if the units are not carefully designed, operated, and maintained. Poorly thickened sludge may hydraulically overload the sludge digester, and high solids in the thickener overflow will increase the loading to the plant. Therefore, proper operation and maintenance of the facility is essential. Important operation and maintenance considerations of a sludge-thickening facility are briefly described below.[15,16]

16-10-1 Common Operational Problems and Troubleshooting Guide

1. Septic odors or rising sludge in thickener are generally caused by a low or infrequent thickened sludge-pumping rate, low thickener overflow rate, or too high a depth of sludge blanket. The problem can be overcome by pumping thickened sludge more frequently, increasing the dilution for overflow rate, chlorinating influent, or adding air to the blending tank.
2. Too thin thickened sludge may be caused by a high overflow rate, high underflow rate, or short-circuiting through the tank. This situation is overcome by reduction in the influent sludge-pumping rate, reduction in dilution water, reduction in pumping of thickened sludge, and maintenance of high sludge blanket. Short-circuiting in gravity thickeners can be detected if uneven discharge of solids occurs over the effluent weir. Weir leveling and change in baffle arrangements may be necessary.
3. Torque overload of sludge-collecting equipment may be caused by accumulation of dense sludge or a heavy foreign object jamming the scraper. The problem may be solved by agitation of the sludge blanket in front of the collector arms with rod or water jets. Foreign objects must be removed by a grappling device or by draining the basin.
4. Plugging of sludge lines and pump may be caused by too thick sludge. The lines should be flushed, and all valves should be fully opened.
5. Hard-to-remove sludge may be caused by too much grit. The problem can be reduced by removing grit efficiently.
6. Excessive growth on weirs may be caused by accumulation of solids and the resultant growth. The weirs and all surfaces should be frequently and thoroughly cleaned by a water jet.

16-10-2 Routine Operation and Maintenance

1. Clean all vertical walls and channels by squeegee daily, and hose down and clean sludge spills without delay.
2. Check the sludge level daily. The sludge level should be kept well below the top of the thickener. Sludge wasting should be controlled to maintain proper sludge blanket.
3. Check daily electrical motor for overall operation, bearing temperature, overload detector, and unusual noises.

4. Check oil level in gear reducers weekly and add as needed. Change oil quarterly and lubricate worn gears weekly.
5. Drain the thickener annually and inspect the underwater portion of the concrete structure and mechanisms. Inspect the mechanical equipment for wear and corrosion, adjust the mechanism, and set proper clearance for flights at tank walls. Patch defective concrete. Metal surfaces should be inspected for corrosion, cleaning, and painting.

16-11 SPECIFICATIONS

The specifications of gravity sludge thickeners are briefly covered here to describe the general features of the equipment. Detailed specifications of similar equipment may be found in Chapters 12 and 13.

16-11-1 General

The gravity sludge thickeners shall consist of complete assembly, including blending tank, feed pumps, collector mechanism with flights, above water center drive mechanisms, influent central well, drive cage, access bridge, bridge support, center pier support, and overload alarm system. Following is a summary of thickener components:

Blending tank	
Number of units	1
Dimensions	8.2-m diameter, 3-m side water depth, 0.6-m freeboard
Mixing arrangement	Mechanical paddle mixer
Gravity thickener	
Number of units	2
Dimensions	12.2-m diameter, 3.9-m side water depth, 0.6-m freeboard, bottom floor slope 17 cm/m
Center column	0.38-m diameter
Feed well	2-m diameter, 1.5 m deep

16-11-2 Materials and Fabrication

All structural steel, iron castings, and concrete shall conform to the current ASTM standards.

16-11-3 Sludge Blending

The mechanical paddle assembly shall consist of a fabricated structural steel frame and redwood paddle blades attached to the frame, as shown on the drawings. The frame shall be designed to resist static and dynamic stresses under all operating conditions. The drive unit shall consist of a primary variable-speed motor reducer, a secondary worn gear reduction unit, and a motor reducer transfer device.

16-11-4 Gravity Thickener

All equipment specified herein is intended for use with combined primary and waste activated sludge. The gravity thickener mechanisms shall be designed to handle thickened sludge up to a maximum of 10 percent solids concentration and be capable of continuously plowing the thickened sludge and moving the settled sludge to the center channel for removal. Each thickener shall be of the center feed and peripheral overflow type, with a central driving mechanism, which shall support and rotate two attached rake arms. Rake collector blades attached to the arms shall be arranged to move the settled sludge on the tank bottom to a concentric sludge channel surrounding the center column. The scrapers attached to the arms shall provide 100 percent coverage of the tank floor.

All gravity-thickening equipment shall be designed so that there will be no chains, sprockets, bearings, or operating mechanisms below the liquid surface. The drive assembly shall comprise an electric motor connected to a primary gear reducer, drive and driver sprockets with drive chain, an intermediate-worn gear reducer, pinion gear, turntable base and main spur gear, and complete automatic overload-actuating system.

16-12 PROBLEMS AND DISCUSSION TOPICS

16-1 Calculate air to solids ratio and dimensions of the flotation tank. Use the following data:

Combined sludge	= 760 m^3/d
Solids concentration	= 1.2 percent
Operating temperature	= 30°C
The solids loading rate not to exceed	= 75 $kg/m^2 \cdot d$
Hydraulic loading rate not to exceed	= 80 $m^3/m^2 \cdot d$
Length to width ratio	= 5:1
Depth	= 2.5 m
Operating pressure	= 19 atm

16-2 Calculate the dimensions of a gravity thickener to thicken combined sludge. The sludge volume is 500 m^3/d and has a solids concentration of 1.0 percent and sp. gr. of 1.008. The solids loading must not exceed 50 $kg/m^2 \cdot d$, and hydraulic loading must not be less than 6 $m^3/m^2 \cdot d$. The underflow concentration of thickened sludge is 6 percent (sp.gr. = 1.03). Calculate the volume of thickened sludge, volume of dilution water, volume of supernatant, and TSS concentration in the supernatant. Assume a solids capture efficiency of 85 percent.

16-3 Design the effluent structure as described in Sec. 16-9-2, Step E. Use the design procedure as given for the secondary clarifier in Chapter 13. Effluent launder is 0.3 m wide, and 90° V-notches are arranged on one side of the launder around the periphery.

16-4 Determine the power requirement, and the paddle area required to achieve $G = 60/s$ in a tank that has volume = 4000 m^3, water temperature = 20°C, and $C_D = 1.8$. The paddle-tip velocity is 0.6 m/s.

16-5 Determine the motor power required for a sludge-blending tank using redwood paddles. There are 12 paddles, six on each side of the central shaft. Each paddle is 0.10 m wide × 2.75 m high. The distances to the middle of each paddle from the center of the shaft are 0.5, 1.0, 1.5, 2.0, 2.5, and 3.0 m. Assume rotational speed of the shaft = 0.06 rps, $C_D = 1.8$, motor efficiency = 75 percent, and specific gravity of blended sludge = 1.01.

16-6 Calculate the mass and ratios of various constituents in the blended sludge (stream 9). Compare these values with those given in Table 13-13. Explain the reasons for the slight differences in the calculated values.

16-7 Calculate the mass and ratios of various constituents in the thickened sludge (stream 10). Compare these values with those given in Table 13-13. Explain the reasons for the slight difference in the calculated values.

16-8 Calculate the concentration of various constituents in the thickener overflow. Compare these values with those given in Table 13-13. Explain the reasons for the slight difference in the calculated values.

16-9 Discuss advantages and disadvantages of gravity thickening over dissolved air flotation.

16-10 Under what conditions may centrifugation be selected for sludge thickening. Describe various types of centrifuges that may be used for sludge thickening.

16-11 Calculate the volume of dilution water needed to achieve a hydraulic loading of 9.5 m^3/m^2d in a gravity thickener. The quantity of dry solids in waste sludge is 8000 kg/d and the sludge volume is 900 m^3/d. Assume solids loading in the thickener is 40 kg/m^2d.

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17

Sludge Stabilization

17-1 INTRODUCTION

The principal purposes of sludge stabilization are to reduce pathogens, eliminate offensive odors, and control the potential for putrefaction of organic matter. Sludge stabilization can be accomplished by biological, chemical, or physical means. Selection of any method depends largely on the ultimate sludge disposal method. As an example, if the sludge is dewatered and incinerated, frequently no stabilization procedure is employed. On the other hand, if the sludge is applied on land, stabilization is necessary to control odors and pathogens.

Various methods of sludge stabilization are (1) anaerobic digestion, (2) aerobic digestion, (3) chemical stabilization (chemical), and (4) thermal conditioning (physical). In recent years, because of its inherent energy efficiency and normally low chemical requirements, anaerobic digestion process is the most widely selected stabilization process at medium- and large-sized municipal plants.

In this chapter each of the above methods of sludge stabilization is briefly discussed. Because of the increasing use of anaerobic and aerobic sludge digestion, these processes are discussed in greater detail. Also, step-by-step design procedure, design details, and operation and maintenance for an anaerobic digester is presented in the Design Example.

17-2 PROCESS SELECTION

The key factor for selection of a sludge stabilization process is the ultimate sludge disposal. The federal, state, and local regulatory agencies have regulations that directly impact the process selection, design criteria, redundancy requirements, and ultimate disposal restrictions. Considerations such as proximity to developed areas and potential for odors must be considered. Two criteria typically used to measure sludge stability following stabilization include volatile solid content and reduction of pathogen indicator organism. Sometimes, these criteria are also used to compare various sludge stabilization processes.[1,2]

17-3 ANAEROBIC DIGESTION

17-3-1 Process Description

Anaerobic digestion utilizes airtight tanks in which anaerobic microorganisms stabilize the organic matter, producing methane and carbon dioxide. The digested sludge is stable, inoffensive, low in pathogen count, and suitable for soil conditioning. The major difficulties with anaerobic digestion are high capital cost, vulnerability to operational upsets, and tendency to produce poor supernatant quality.

Anaerobic digestion involves a complex biochemical process in which several groups of facultative and anaerobic organisms simultaneously assimilate and break down organic matter. The process may be divided into three phases: hydrolysis, acid, and methane. In the hydrolysis phase, the complex molecules of proteins, cellulose, lipids, and other complex organics are solubilized into glucose, amino acids, and fatty acids.

In the acid phase, facultative acid-forming organisms convert the solubilized organic matter to organic acids (acetic, propionic, butyric, and other acids). In this phase little change occurs in the total amount of organic material in the system, although some lowering of pH results. The methane phase involves conversion of volatile organic acids to methane and carbon dioxide. A simplified representation of the anaerobic digestion process is shown in Figure 17-1.

The anaerobic process is essentially controlled by the methane-forming bacteria. Methane formers are very sensitive to pH, substrate composition, and temperature. If the pH drops below 6.0, methane formation essentially ceases, and more acid accumulates, thus bringing the digestion process to a standstill.[2,3] Thus, pH and acid measurements constitute important operational parameters. The methane bacteria are highly active in the mesophilic (27–43°C) and thermophilic (45–65°C) ranges. Anaerobic digesters are most commonly operated in the mesophilic range (35–40°C). Recent thinking, however, is to operate the digester in the thermophilic range. The main advantages of thermophilic digestion are increased efficiency and improved dewatering.[1-4] Overall volatile solids destruction is in the range of 40–60 percent.

17-3-2 Types of Anaerobic Digesters

The anaerobic sludge digestion can be achieved in three types of digesters: standard-rate, high-rate, and two-stage. These types of digesters and the design criteria are presented below.

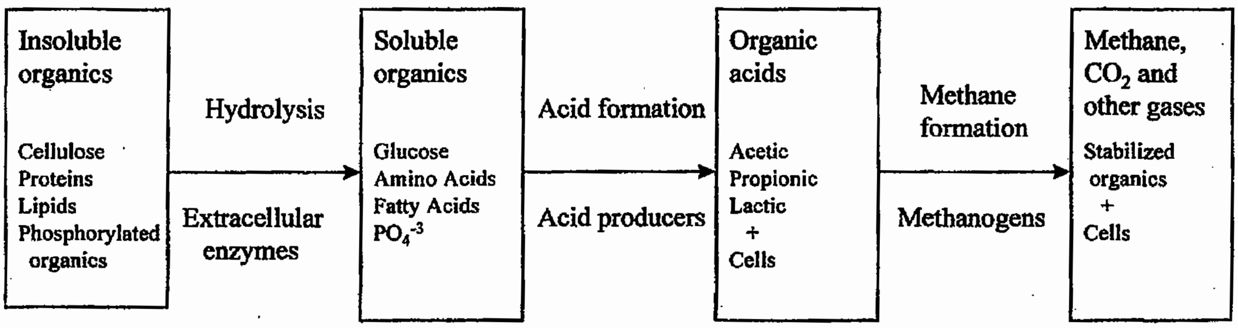


Figure 17-1 Simplified Process Summary for Anaerobic Digestion (Ref. 1).

Standard-Rate Digestion. In the standard-rate digestion process, the digester contents are usually unheated and unmixed. The sludge is fed intermittently. Stratification occurs, forming four zones: (1) a scum layer, (b) a liquid layer or supernatant, (c) a layer of actively digesting solids, and (d) a layer of digested and inert solids. The supernatant and digested sludge are withdrawn periodically. The standard-rate digestion process is generally unstable and inefficient. The digestion period may vary from 30 to 60 days. A cross section of a standard-rate digester is shown in Figure 17-2(a), and design criteria are provided in Table 17-1.

High-Rate Digestion. In a high-rate digestion process, the digester contents are heated and completely mixed. The required digestion period is 10–20 d, and solids loading is

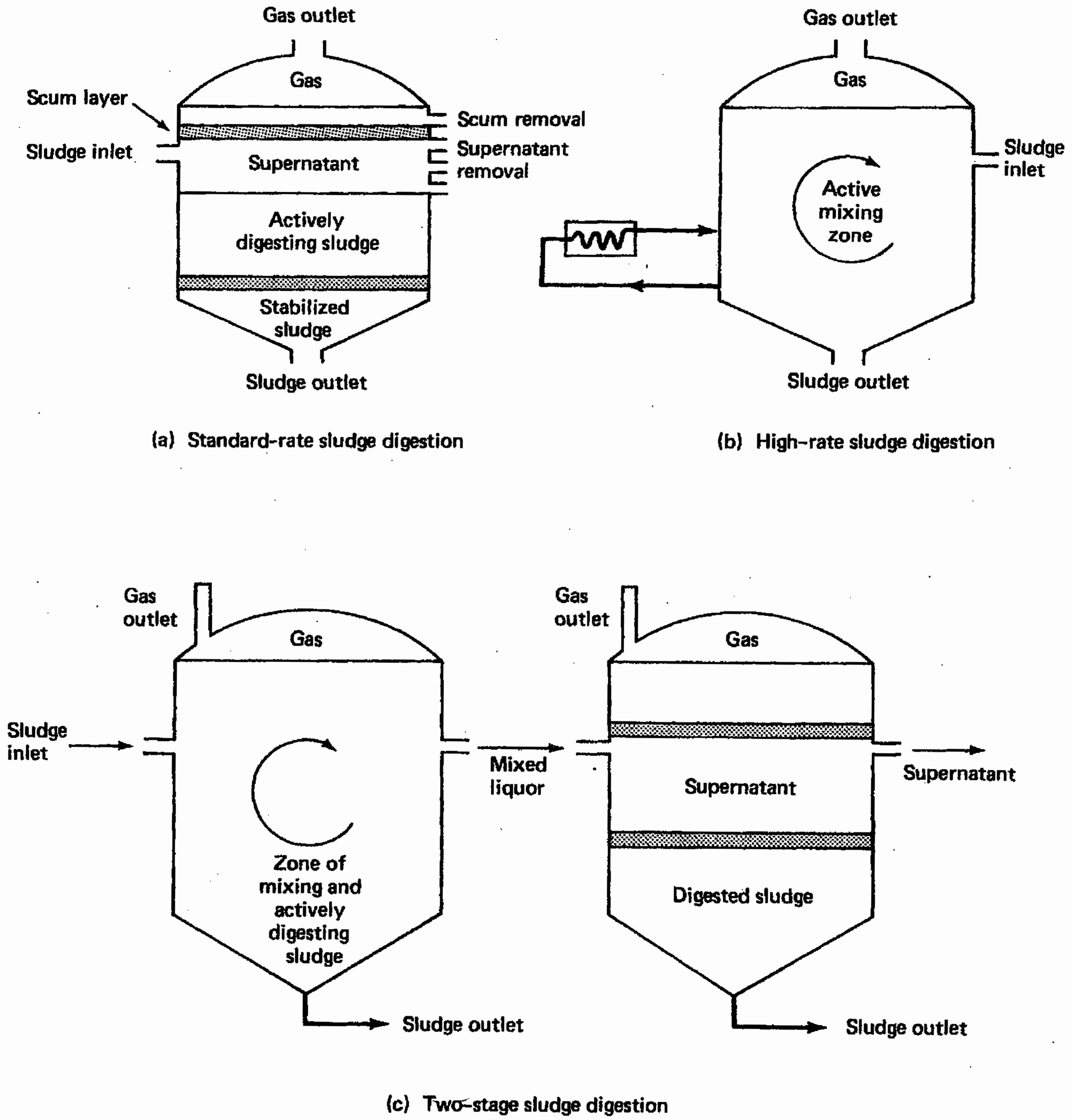


Figure 17-2 Typical Anaerobic Sludge Digesters.

TABLE 17-1 Typical Design Criteria for Standard-Rate and High-Rate Digesters

Parameter	Standard-Rate	High-Rate
Solids retention time (SRT), days	30–60	10–20
Sludge loading,		
kg VS/m³·d	0.64–1.60	2.40–6.41
lb VS/ft³·d	0.04–0.10	0.15–0.40
Volume criteria		
Primary sludge,		
m³/capita	0.03–0.04	0.02–0.03
ft³/capita	2–3	1–2
Primary sludge + waste activated sludge,		
m³/capita	0.06–0.08	0.02–0.04
ft³/capita	4–6	1–3
Primary sludge + trickling filter,		
m³/capita	0.06–0.14	0.02–0.04
ft³/capita	4–5	1–3
Sludge feed solids concentration, percent dry wt.		
Primary + waste activated sludge	2–4	4–6
Digested solids underflow concentration, percent dry wt.	4–6	4–6

Source: Refs. 1 and 2.

much higher. There is generally no supernatant separation, and total volatile solids reduction is 50–60 percent of feed TVSS. The schematic flow diagram of a high-rate digester is shown in Figure 17-2(b), and design criteria are provided in Table 17-1. In high-rate digestion, the sludge feeding is generally on a regular schedule either before or after withdrawing an equal volume of digested sludge.[2,3]

Two-Stage Digestion. Often, a combination of standard- and high-rate digestion is achieved in two-stage digestion. The second stage digester mainly separates the digested solids from the supernatant liquor although additional digestion and gas recovery may also be achieved. The schematic flow diagram of a two-stage digestion process is shown in Figure 17-2(c). Frequently, the tanks are made identical, in which case either one may be the primary. In other cases, the second tank may be an open tank or even a sludge lagoon.[1,4]

17-3-3 Process Design and Control

The most important factors controlling the design and operation of anaerobic digestion are tank design, digester capacity, digester heating and temperature control, mixing, gas production and utilization, digester cover, supernatant quality, and sludge characteristics. Each of these factors is discussed below.

Tank Design. The anaerobic digesters are cylindrical, rectangular, or egg-shaped. Cylindrical-shaped digesters are most common and have a diameter ranging from 6 to 40 m (20 to 130 ft). The floor is conical with slope 1 vertical to 4 horizontal toward the center. A waffle-shaped bottom is also used to minimize grit accumulation.[1,4] The cylindrical digesters have a depth ranging from a 7- to 14-m vertical wall. Egg-shaped digesters have been extensively used in Europe for many years, yet only recently have they been introduced to the United States. The reported advantage of egg-shaped digesters when compared with cylindrical digesters are (1) enhanced mixing and no dead zones, (2) little or no accumulation of grit, (3) better control of scum, (4) elimination of the need for cleaning, and (5) smaller land area. The mixing may be achieved by mechanical mixers or jet pump mixing. The schematic flow diagram of an egg-shaped anaerobic digester is shown in Figure 17-3.

Digester Capacity. The digester capacity is generally based on (1) digestion period, mean cell residence time, or solids retention time; (2) volumetric loading; (3) population basis; and (4) observed volume reduction. Each of these bases are discussed below.

Digestion Period, Mean Cell Residence Time, or Solid Retention Time. Most of the standard-rate digesters are designed for a digestion period of 30–60 d. A high-rate anaerobic digester is heated and is a completely mixed biological reactor (complete-mixed reactors are defined in Chapters 4 and 13). These digesters are generally designed for a digestion period of 10–20 d since solids are adequately stabilized in this period. Many kinetic equations have been proposed that describe the biochemical reactions.[4-6] These equations and design calculations are shown in the Design Example.

Volumetric Loading. The digester capacity is also estimated using the volumetric loading. The volumetric loading is generally expressed as kilogram total volatile solids added per d per m^3 of the digester capacity. Typical organic loadings for design of standard- and high-rate digesters are summarized in Table 17-1.

Population Basis. The digester capacity is estimated on a population basis using 120 g of solids per capita per day. The design values on population basis are given in Table 17-1.

Observed Volume Reduction. During digestion, the volume of solids is generally reduced, and a certain amount of supernatant may be returned to the plant. Thus, the volume of sludge remaining in the digester decreases exponentially. The required volume of the digester is calculated from Eq. (17-1):[1,3]

$$V = [Q_{\text{in}} - 2/3\ (Q_{\text{in}} - Q_{\text{out}})]\ D_T \tag{17-1}$$

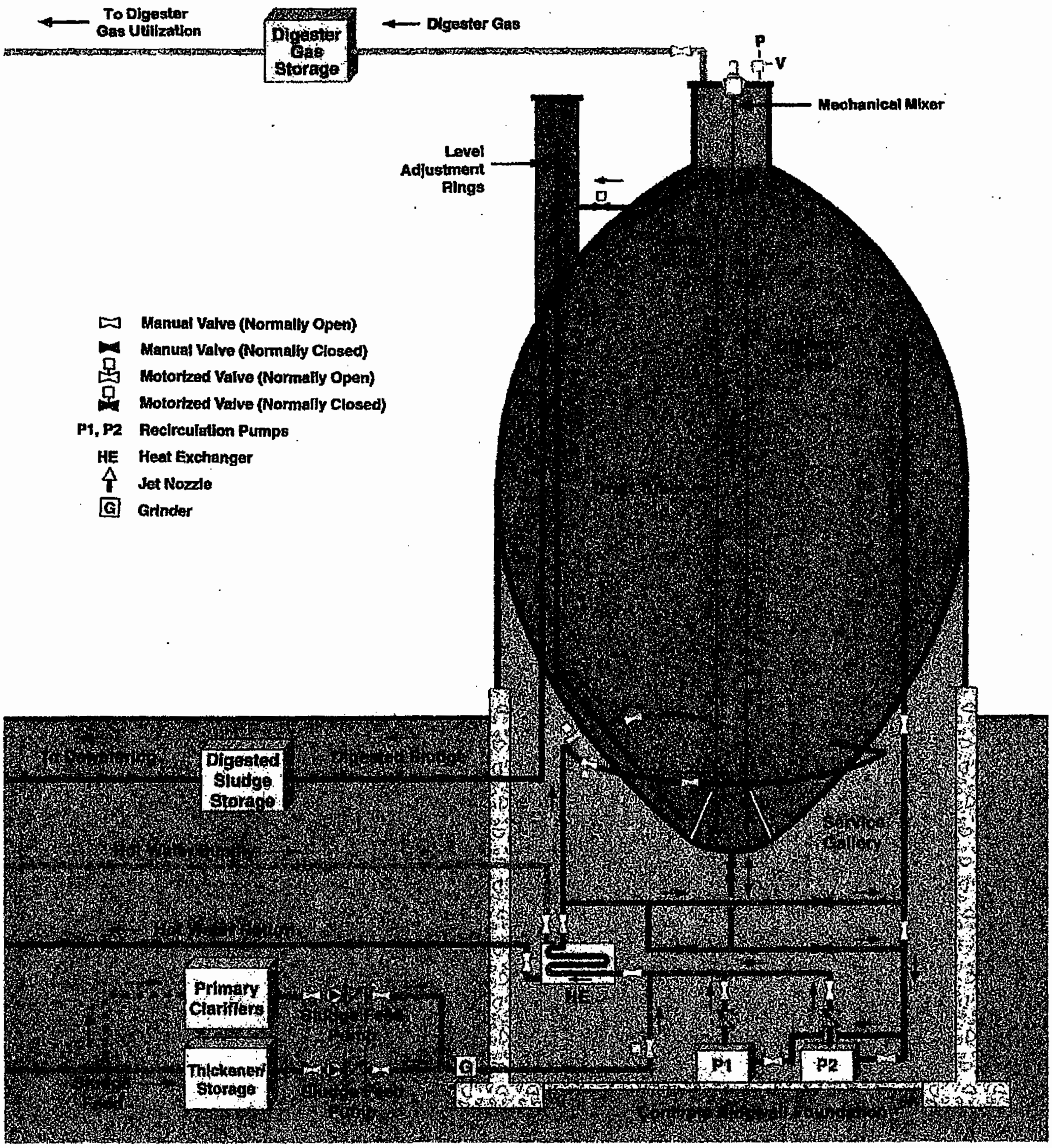


Figure 17-3 Schematic Flow Diagram of Egg-Shaped Anaerobic Digester (courtesy CBI Walker, a division of Chicago Bridge & Iron).

where

$$
\begin{aligned}
V &= \text{volume of digester, m}^3\ (\text{ft}^3) \\
Q_{\text{in}} &= \text{sludge feed rate, m}^3/\text{d}\ (\text{ft}^3/\text{d}) \\
Q_{\text{out}} &= \text{sludge withdrawal rate, m}^3/\text{d}\ (\text{ft}^3/\text{d}) \\
D_T &= \text{digestion period, d}
\end{aligned}
$$

In many designs, the digester capacity should be checked to make sure that the minimum digestion period during periods of high flow does not fall below the critical value of 10 d. At less than 10 d of digestion period, the production of methane bacteria cannot keep up with the production of acids; thus, the digestion process begins to slow down. The digester capacity, therefore, must be checked when the following conditions occur simultaneously:

1. Peak hydraulic loading: Peak condition should be estimated by combining poor thickener operation with the maximum expected 7-d sustained hydraulic or organic loading.
2. Maximum grit and scum accumulation: Considerable amounts of grit and scum may accumulate before a digester is cleaned. This reduces the active volume of the tank.
3. Liquid level: Approximately 0.5–1 m variability in liquid level must be provided in the design. This is necessary to allow for differences in the rate of feeding, withdrawal, and scum accumulation and to provide reasonable operational flexibility.

Digester Heating and Temperature Control. The rates of biological growth and solids stabilization increase and decrease, respectively, with temperature within certain limits. For mesophilic and thermophilic digestion, the optimum temperatures are around 35°C and 54°C, respectively. It is therefore important to maintain proper temperature by heating the incoming sludge and heating the digester content. The total amount of input heat should balance the heat losses from the digester. The heat loss sources from the digester are digester walls, floor, roof, piping, etc. Proper heat loss calculations must be made to design the heating system. Common digester heating methods are (1) internal heat exchanger coils, (2) steam injection, and (3) external heat exchangers. Each of these methods of heating is discussed below.

Internal Heat Exchanger. Digester heating in the early days was done by heat exchanger coils placed inside the digester. Serious problems developed when the coils became encrusted, reducing the heat transfer. To minimize caking of sludge on the coils, water recirculating through the coils is kept between 45°C and 55°C.

Steam Injection Heating. Steam is pumped into the digester for heating. The benefit of this system is that no heat exchanger is needed. The problems, however, are dilution of the sludge and 100 percent boiler makeup water.

External Heat Exchangers. Three types of external heat exchangers are commonly used for sludge heating: water bath, jacket pipe, and spiral exchanger. In the water bath exchanger the boiler tubes and sludge piping are located in a common, water-filled container. In a jacketed pipe exchanger, hot water is pumped countercurrent to pipe surrounding the sludge pipe. The spiral exchangers also utilize countercurrent flow design; however, the sludge and water passageways are cast in a spiral. The heat transfer coefficients for design of external heat exchangers range from 3000–5640 kJ/h·m^2·°C (150–275 Btu/h·ft^2·°F).

The hot water or steam used to heat digesters is most commonly generated in a boiler fueled by sludge gas. Up to 80 percent heat value of the gas can be recovered in a boiler. Provision for burning an alternate fuel source (natural gas) must be made. Of-

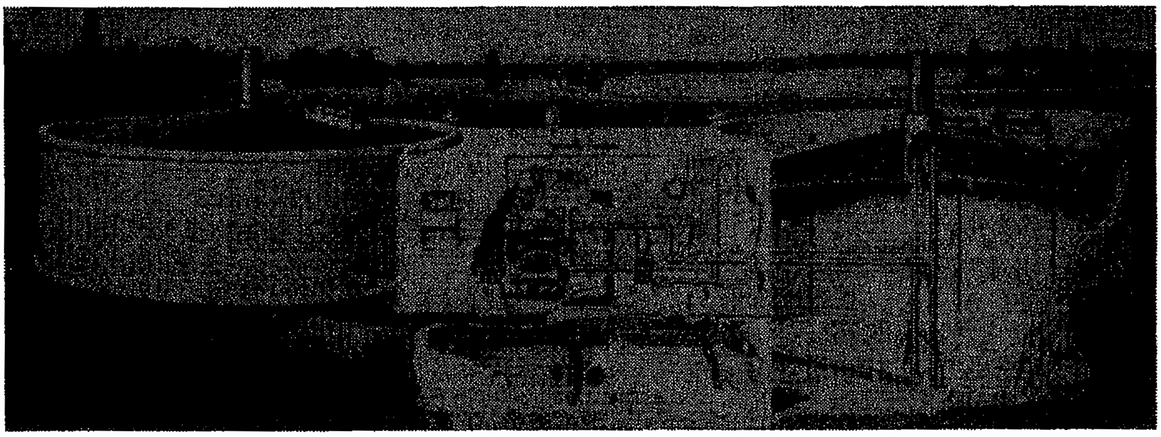


Figure 17-4 Control Room of an External Sludge Heating Unit Using a Two-Stage Digester System (courtesy Envirex, U.S. Filter).

ten, sludge gas is used in engines to generate electricity, and the waste heat from these engines is sufficient to meet digester heating requirements. Figure 17-4 shows the external heating and mixing arrangement in a digester building.

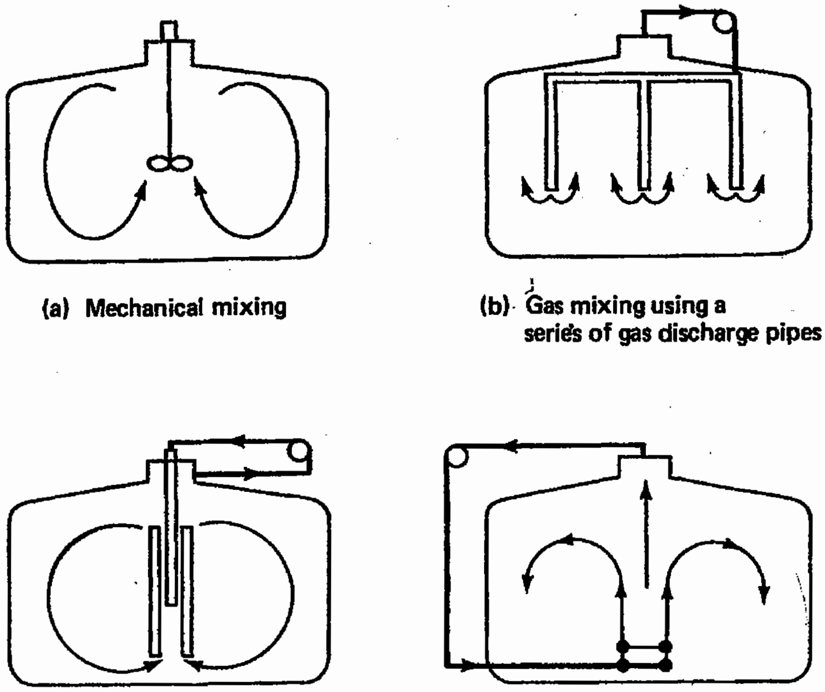
Digester Mixing. Anaerobic digesters must be mixed properly to provide optimum performance. Mixing has the following beneficial effects:

1. Maintains intimate contact between feed sludge and active biomass
2. Creates physical, chemical, and biological uniformity throughout the digester
3. Rapidly disperses metabolic end products produced and any toxic chemicals entering the digester
4. Prevents formation of surface scum

A certain amount of natural mixing occurs in an anaerobic digester caused by both the rise of sludge gas bubbles and the thermal convection currents created by the addition of heated sludge. However, natural mixing is not sufficient, and therefore, additional mixing is needed. Methods used for mixing include external pumped circulation, internal mechanical mixing, and internal gas mixing. Each of these methods is discussed below.

External Pumped Circulation. A large volume of sludge is pumped out and then returned to the digester. Besides circulation, external heating of digester contents is also possible. The sludge is generally withdrawn from the mid-depth of the digester and then pumped back through two nozzles located at the base of the digester on opposite sides or pumped at the top to break the scum. This method of mixing has a high energy demand.

Internal Mechanical Mixing. Mechanical mixers are generally installed into a shaft tube to promote vertical mixing. Mechanical mixing has not been very successful as large amounts of raggy materials in sludge result in fouling of the propellers and subsequent failure of the mechanisms.



(a) Mechanical mixing

(b) Gas mixing using a series of gas discharge pipes

(c) Gas mixing using a central draft tube

(d) Gas mixing using diffusers mounted at the tank bottom

Figure 17-5 Different Types of Mixing Arrangements Used in an Anaerobic Digester: (a) mechanical mixing, (b) gas mixing from suspended pipes, (c) gas mixing with central draft tube, and (d) gas mixing from rings of spargers mounted on floor.

Internal Gas Mixing. This is an effective method of digester mixing. Many types of gas-mixing arrangements have been successfully designed.[1,3,7] These include

- injection of sludge gas bubbles at the bottom of a central-draft-tube to create piston-pumping action and provide surface agitation
- injection of sludge gas sequentially through a series of lances suspended from the digester cover to as great a depth as possible
- release of gas from a ring of spargers mounted on the floor of the digester

Some of the digester mixing arrangements are shown in Figure 17-5. The design procedure for a gas-mixing system is covered in the Design Example.

Gas Production and Utilization. There is a great interest in the utilization of sludge gas as an energy source. The digester gas contains approximately 60–70 percent methane, 25–30 percent carbon dioxide, and small amounts of hydrogen, nitrogen, hydrogen sulfide, and other gases. The gas has a heating value of 21,000–25,000 kJ/m^3 (500–650 Btu/ft^3)[a] and density of 86 percent of air.[b] The digester gas has been successfully used to heat the digesters, buildings, and drive engines.

[a]For comparison, the heating values of methane and natural gases are 35,800 and 37,300 kJ/m^3, respectively.

[b]Air weighs approximately 1.162 kg/m^3 (0.07251 lb/ft^3).

The methane generation rate may be estimated from the kinetic equations developed for the anaerobic digester. Equations (17-2) and (17-3) express these kinetic relationships:[1-3]

$$P_x = \frac{YES_o}{1 + k_d \theta_c} \tag{17-2}$$

$$V = 0.35\ \text{m}^3/\text{kg}\ \{[ES_o] - 1.42(P_x)\} \tag{17-3}$$

where

P_x = net mass of cell produced, kg/d
Y = yield coefficient, g/g. For municipal sludge it ranges from 0.04–0.1 mgVSS/mgBOD utilized.
E = efficiency of waste utilization (0.6–0.9)
S_o = ultimate BOD_L of the influent sludge, kg/d
k_d = endogenous coefficient, d^{-1}. For municipal sludge it is 0.02–0.04 d^{-1}.
θ_c = mean cell residence time, d. This is also equal to the digestion period.
V = volume of methane produced, m^3/d
0.35 = theoretical conversion factor for the amount of methane produced from the conversion of 1 kg of BOD_L
1.42 = conversion factor for cellular material to BOD_L

Other rules of thumb for estimating digester gas volume are

1. 0.5–0.75 m^3/kg (8–12 ft^3/lb) of volatile solids loading
2. 0.75–1.12 m^3/kg (12–18 ft^3/lb) of volatile solids reduced
3. 0.03–0.04 m^3 per person per day (1.1–1.4 ft^3 per person per day)

The gas collection system includes fixed or floating covers in the digesters, gas pipings and pressure relief valves, adequate flame traps, gas compressors, gas meters, and gas storage tank. It should be noted that the digester gas makes an explosive mixture with air. Therefore, necessary safety precautions must be utilized to prevent explosion.

Digester Cover. Anaerobic digester covers are provided to keep out oxygen, contain odors, maintain operating temperature, and collect digester gas. The digester covers can be either fixed or floating.

Fixed cover digesters are less expensive and are designed to maintain a constant surface level in the tank. Often, rapid withdrawals of digested sludge can draw air into the tank, producing an explosive mixture of sludge gas and oxygen.[3] The explosive range of sludge gas in air is 5–20 percent by volume. In addition, if the liquid level in the digester is increased, it can damage the cover structurally. Fixed cover designs are shown in Figure 17-6.

Floating covers are more expensive but allow the independent addition and withdrawal of the sludge, reduce gas hazards, and can be designed to control formation of a

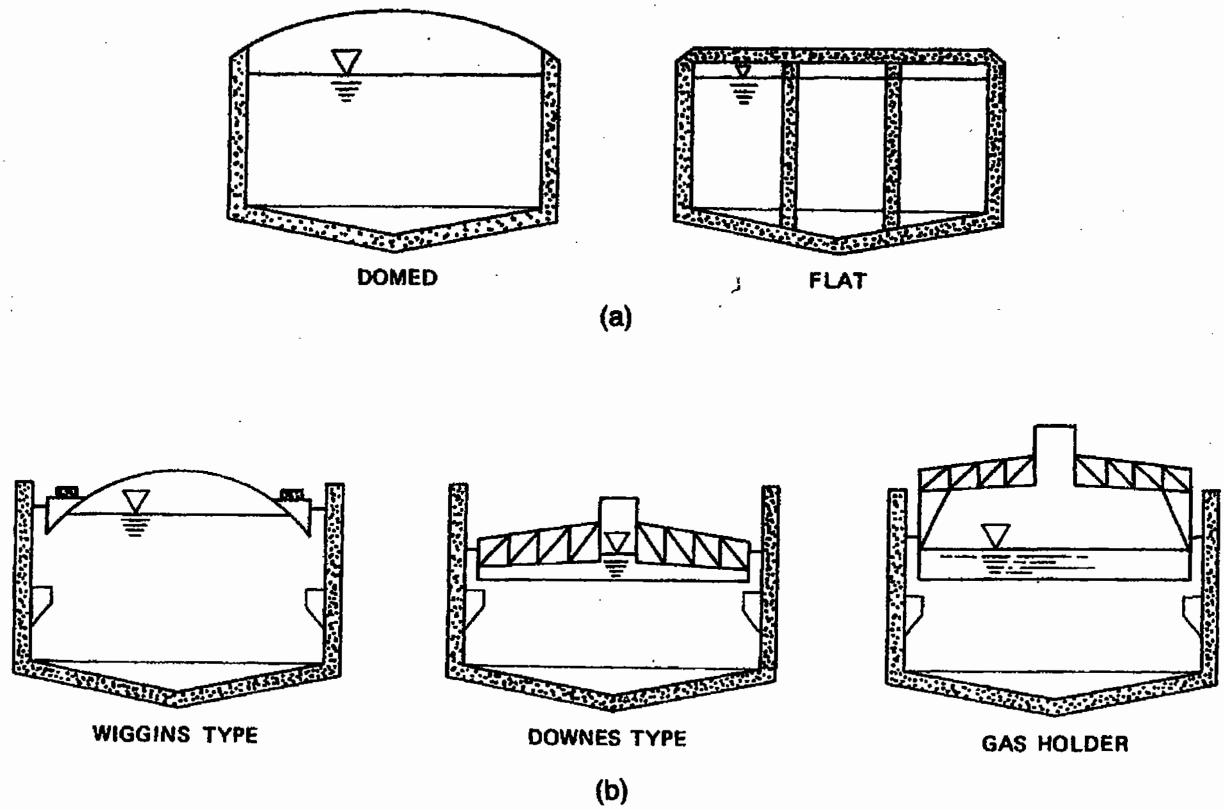


Figure 17-6 Types of Digester Covers: (a) fixed covers and (b) floating covers (from Ref. 2).

scum mat. There are two common types of floating cover designs: (1) Pontoon or Wiggins type and (2) the Downes type. Both types of floating covers float directly on liquid and commonly have a maximum vertical travel of 2–3 m (6–8 ft). These covers differ primarily in the method used to maintain buoyancy, which, in turn, determines the degree of submergence. These types of floating covers are shown in Figure 17-6. General discussion on each type may be found in Refs. 2 and 3.

A variation of the floating cover is the floating gas holder. Basically, the gas holder cover is a floating cover with an extended 3-m (10-ft) skirt to allow storage of gas during periods when gas production exceeds the demand. Gas-holding covers are less stable because they are supported on a gas cushion, and they also expose a large side area to lateral wind loads. A gas holder cover is shown in Figure 17-6.

Typical appurtenances for a digester cover include sampling ports; manholes for access, ventilation, and debris removal during cleaning; a liquid overflow system; and a vacuum pressure relief system equipped with a flame trap. The gas pressure under a digester cover is typically 0–3.7 kN/m^2 (0–15 in. of water).

Supernatant Quality. The digester supernatant quality is dependent on whether the digester has one or two stages, whether it is mixed, and how well the solids separate from the liquor. The supernatant is generally returned to the plant for treatment. It may often add significant loading to the plant. Characteristics of supernatant from the anaerobic digester are given in Table 17-2.

TABLE 17-2 Characteristics of Supernatant from Anaerobic Digester Treating Thickened Primary and Waste Activated Sludge

Parameter	Concentration (mg/L)
TS	3000–15,000
BOD_5	1000–10,000
COD	3000–30,000
Ammonia nitrogen as N	400–1000
Total phosphorus as P	300–1000

Source: Refs. 2 and 3.

Sludge Characteristics and Other Factors. pH, volatile acids, nutrients, and toxic materials are important for process control. Some of these factors are discussed in digester operation (Sec. 17-8).

17-4 AEROBIC DIGESTION

Aerobic sludge digestion is commonly used at small plants to stabilize the organic matter in the sludge. The process involves aeration of sludge for an extended period in open tanks. The process is similar to an activated sludge and involves the direct oxidation of biodegradable matter and oxidation of microbial cellular material (endogenous respiration). Stabilization is not complete until there has been an extended period of primarily endogenous respiration (10–20 days). The process has the following advantages: (1) it is simple to operate, (2) it involves low capital cost; (3) the digested sludge is odorless, biologically stable, and has excellent dewatering properties; and (4) the supernatant is low in BOD_5. The digested sludge is normally dewatered on sand drying beds. The disadvantage of aerobic digestion is a high operating cost.

The important process parameters are (1) air or oxygen requirement, (2) aeration period, (3) sludge age, (4) temperature, (5) biodegradable volatile solids, (6) processing requirements of the digested sludge, and (7) supernatant quality. Current practice is to provide approximately 15 days of detention time to achieve 40–50 percent reduction in volatile solids. Oxygen requirements, exclusive of nitrification, can vary from 3 to 30 mg per hour per gram of volatile solids under aeration.[2] When nitrification occurs, both pH and alkalinity are reduced. A schematic flow diagram of an aerobic digestion system is given in Figure 17-7. Basic design parameters of aerobic digesters are summarized in Table 17-3. Characteristics of an aerobic digester supernatant are given in Table 17-4.

An innovative aerobic sludge digestion system utilizes deep shaft technology. The reactor typically has a 3-m diameter shaft, 100 m in depth. Because of the depth, a high temperature reaching the thermophilic range is reached. The sludge after grinding is pumped into the shaft. Air is pumped into two zones. The overall process diagram is di-

TABLE 17-3 Aerobic Digestion Design Parameters

Parameter	Value	Remark
Solids retention time (SRT), d	10–15[a] 15–20[b]	Depending on temperature, type of sludge, mixing, etc., the sludge age may range from 10 to 40 days.
Volume allowance		Depending on temperature, type of sludge, mixing, etc.
m^3/capita	0.085–0.113	
ft^3/capita	3–4	
VS loading		Depending on temperature, type of sludge, mixing, etc.
$kg/m^3 \cdot d$	0.384–1.600	
$lb/ft^3 \cdot d$	0.024–0.10	
Air requirements		Enough to keep the solids in suspension and to maintain a DO of 1–2 mg/L
Diffused system[a]		
$m^3/m^3 \cdot min$	0.020–0.035	
$ft^3/1000\ ft^3 \cdot min$	20–35[a]	
Diffused system[b]		
$m^3/m^3 \cdot min$	>0.06	
$ft^3/1000\ ft^3 \cdot min$	>60	
Mechanical system		
kW/m^3	0.0263–0.0329	
$HP/1000\ ft^3$	1.0–1.25	
Minimum DO, mg/L	1.0–2.0	
Temperature, °C	>15	If sludge temperatures are lower than 15°C, additional detention time should be provided because the digestion will occur at the lower biological reaction rates.
VSS reduction, percent	35–50	
Tank design		Aerobic digestion tanks are open and generally require no special heat transfer equipment or insulation. For small treatment systems 0.044 m^3/s (1.0 mgd) or less, the tank design should be flexible enough so that the digester tank can also act as a sludge-thickening unit. If thickening is to be utilized in the digestion tank, sock-type diffusers should be used to minimize clogging.
Power requirements		
kW per 10,000 population equivalent	6–7.5	
brake horsepower (Bhp) per 10,000 population equivalent	8–10	

[a]Waste activated sludge only.
[b]Primary plus waste activated sludge or primary sludge only.
Source: Adapted in part from Refs. 2 and 3.

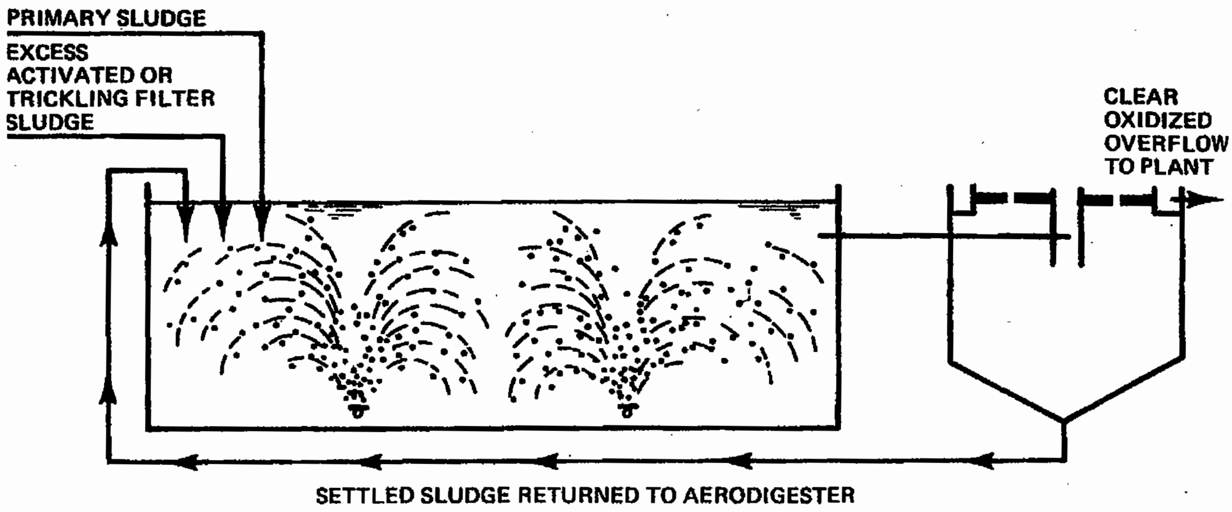


Figure 17-7 Schematic of Aerobic Sludge Digestion (from Ref. 2).

vided into seven steps: (1) sludge inlet, (2) mixing zone where incoming sludge is mixed with recirculating mixed liquor, (3) high oxygen transfer zone, (4) headtank zone where spent gases are released, (5) lower aeration and high temperature and pressure zone, (6) digested sludge withdrawal zone, and (7) rapid depressurization and separation of digested sludge. These steps are schematically shown in Figure 17-8.

17-5 OTHER SLUDGE STABILIZATION PROCESSES

Other sludge stabilization processes utilize chemical and physical means. Commonly used chemical and physical stabilization processes are chemical stabilization and fixation, lime stabilization, and thermal conditioning. Each of these processes is briefly discussed below.

TABLE 17-4 Characteristics of Supernatant from an Aerobic Digester[a]

Parameter	Range	Typical
pH	5.9–7.7	7.0
BOD_5	9–1700	500
Soluble BOD_5	4–183	51
COD	288–8140	2600
TSS	46–11,500	3400
Kjeldahl N	10–400	170
Total P	19–241	98
Soluble P	2.5–64.0	26

[a]All units in mg/L except pH.

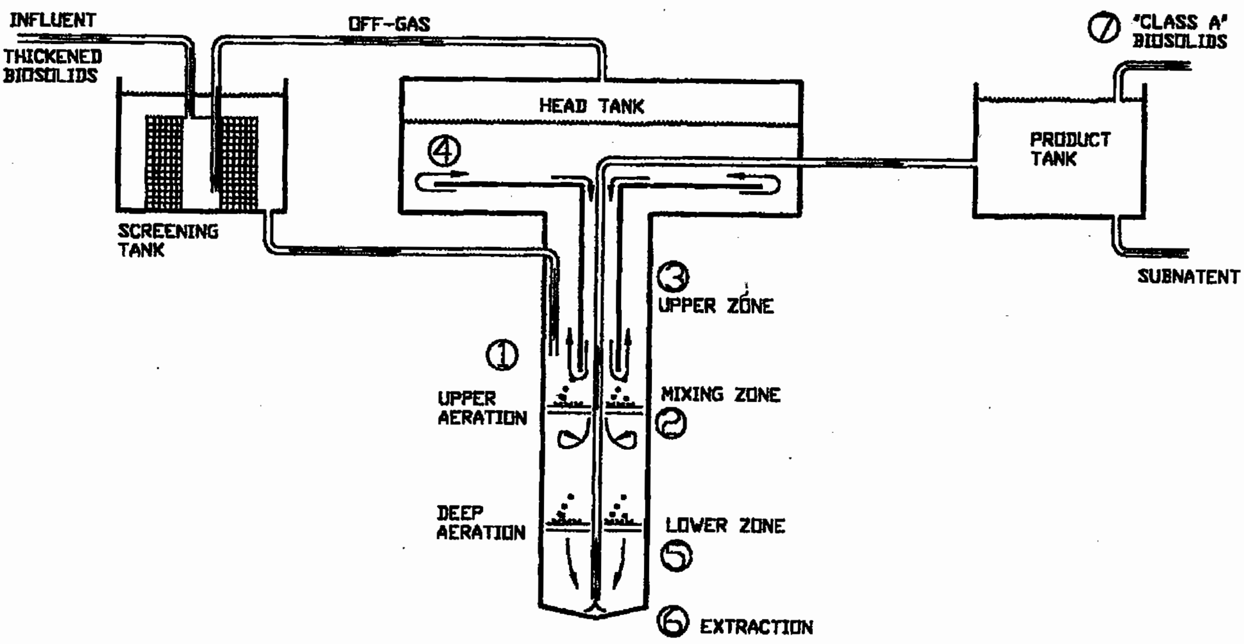


Figure 17-8 Schematic Process Diagram of Deep-Shaft Aerobic Sludge Digestion Process (VERTAD™ Process) (courtesy Deep Shaft Technology, Inc.).

17-5-1 Chemical Stabilization and Fixation

Chemical stabilization involves treatment of sludge with high dosages of oxidant. The most commonly used oxidant is chlorine. The sludge is deodorized and microbiological activity slows down. The sludge is then dewatered. The chlorine oxidation units usually have a prefabricated modular design and are generally applicable to small plants only.

Chemical oxidation using chlorine is not cost-effective because of the large amount of chlorine required and because of safety concerns. Other chemicals such as hydrogen peroxide and ozone have also been used.[1,3] Chemical fixation of hazardous wastes has been used for many years. Recently, there has been some interest to utilize this technology in chemical fixation of wastewater treatment plant residues such as grit, scum, incinerated sludge ash, and sludge. Chemical fixation involves the addition of material with pozzolanic (cementitious) properties to immobilize the solid. Portland cement, fly ash, polymers, and thermoplastic materials have been used for hazardous wastes. The application of this technology in wastewater sludge is relatively new and is in the development phase.[8-10]

17-5-2 Lime Stabilization

Lime is added to raw sludge to raise the pH to 12 or higher.[1,3] At high pH, the sludge does not putrefy, create odors, or pose a health hazard. A high lime dosage and about a 3-h contact time is required to get a significant level of pathogen kill. The process also inactivates bacteria, viruses, and other microorganisms and halts or substantially retards the microbial reactions that are generally associated with nuisance.

Lime also reacts with many organic and inorganic constituents in sludge, thus altering the chemical composition of sludge. The proper lime dosage varies with the type of

TABLE 17-5 Lime Dosage for Sludge Stabilization

Sludge Type	Solid Concentration		Lime Dosage, kg $Ca(OH)_2$/kg Dry Solids	
	Range	Typical	Range	Typical
Primary	3–6	4.3	0.06–0.17	0.12
Waste activated	1–1.5	1.3	0.21–0.43	0.30
Combined Primary and Waste activated	1.5–3.0	2.5	0.15–0.35	0.24
Digested	5–6	5.5	0.15–0.30	0.25

Adapted in part from Refs. 1, 4, and 10.

sludge and solids concentration. As the solids concentration increases, the lime dosage decreases. The typical lime dose for different types of sludge stabilized is given in Table 17-5. The quantity of lime dosage is sufficient to maintain a pH of 12 for 30 minutes. In lime stabilization the organic matter is not destroyed; rather, it is denaturized. Therefore, the sludge must be disposed of before the organic matter starts to putrefy again. Studies have shown that much process- and design-related information must be developed to understand and design the process components. Some of the information is summarized below:[11-13]

1. Lime stabilization may be used as an interim process when the primary stabilization process is temporarily out of service.
2. If quicklime ($CaCO_3$) is used, a considerable amount of heat is released, and the temperature of the sludge may increase significantly.
3. Pathogen reduction as high as 99 percent or more may be reached with proper lime stabilization. Regrowth of pathogens is minimal.
4. There is odor reduction by lime stabilization. Odor production remains minimal as long as pH remains high. Odor generation remains insignificant if lime stabilized sludge is incorporated into the soil. The sludge will benefit agricultural land that has acidic soil.
5. The organic matter is not destroyed. A reduction of 10–35 percent in TVSS is caused by lime dosage, an increase in TSS, and the dilution effect.
6. There is a loss of NH_4^+-N because of volatilization, increase in alkalinity because of lime, reduction in soluble phosphorus in sludge because of precipitation, and immobilization of heavy metals because of precipitation.
7. The lime feed facility will require design of lime handling, delivery, storage, transfer, dry feeder, slurry preparation, solution feeding, and mixing system. The basic design information on these components may be found in Refs. 1, 4, and 14.

17-5-3 Heat Treatment or Thermal Conditioning

Heat treatment is both a conditioning and stabilization process. It involves heating the sludge (140–250°C) for approximately 30 min under a pressure of 2760 kN/m^2 (400 psi). Heat coagulates the solids, breaks down the gel structure, and reduces the water affinity of the sludge solids. The resulting sludge is sterilized, practically deodorized, and dewatered readily on a vacuum filter or belt filter press without the need of chemicals.[1,2,4] This process is most applicable to biological sludges that are normally difficult to condition or stabilize by chemicals. Because of high capital costs, the process is generally limited to large plants.

Sludge stabilization is also achieved by wet oxidation. In this process the sludge solids are incinerated in the liquid phase. This process is discussed in Chapter 19.

17-6 EQUIPMENT MANUFACTURERS OF SLUDGE STABILIZATION SYSTEMS

The equipment manufacturers of various sludge stabilization systems are given in Appendix D. These systems include anaerobic and aerobic sludge digestion, chemical oxidation and lime stabilization, and thermal conditioning and incineration. Proper selection of the equipment is necessary to assemble the required components that will provide operational flexibility and maintenance requirements. Basic considerations for manufacturer selection are given in Sec. 2-10.

17-7 INFORMATION CHECKLIST FOR DESIGN OF SLUDGE STABILIZATION FACILITY

It is important that the design engineers make the necessary design decisions prior to starting the design calculations for sludge stabilization facility. The following checklist may be helpful in developing the necessary predesign data:

1. Select the ultimate sludge disposal method because the degree of sludge stabilization will depend on the requirements of the disposal practice.
2. Develop the characteristics of thickened sludge that will reach the sludge stabilization facility. This includes flow and solids contents under average design flow conditions and under maximum sustained loading conditions, including poor thickener operation. Material mass balance analysis similar to that given in Chapter 13 may be necessary.
3. Select the sludge stabilization method that is compatible with the influent sludge characteristics, dewatering, and ultimate disposal method.
4. Develop design parameters for the selected sludge stabilization facility. The design parameters may include organic loading, hydraulic loading, chemical dosage, reaction period, etc. Laboratory tests may be necessary to develop some of the design parameters.
5. Obtain the design criteria from the concerned regulatory agency.
6. Obtain necessary manufacturers' catalogs and equipment selection guides.

17-8 DESIGN EXAMPLE

17-8-1 Design Criteria Used

The following design standards and criteria are used for the sludge stabilization facility:

1. Select anaerobic sludge digestion for stabilization of organic solids.
2. Provide two complete-mixed, high-rate anaerobic heated digesters with digestion temperature of 35°C (95°F).
3. The design flow to the sludge digester shall be equal to thickened sludge under the daily design flow condition. The digester shall have the flexibility to adequately handle the thinnest sludge at the highest solids production rate and the thickest sludge at the lowest solids production rate. The average and extreme characteristics of sludge reaching the digester are summarized in Table 17-6.
4. Total volatile solids loading to the digester shall not exceed 3.6 kg/m^3·d under extreme high loading conditions.
5. The solids retention time at extreme high-flow conditions shall not be less than 10 days.
6. The digester mixing shall be achieved by internal gas mixing.
7. The solids content in digested sludge is 5 percent, and specific gravity is 1.03.
8. The TSS content in the supernatant is 4000 mg/L.

TABLE 17-6 Characteristics of Sludge under Average and Extreme Conditions Reaching the Anaerobic Digester

Factors	Average Flow Condition	Extreme Low Flow	Extreme High Flow
Thickened sludge production, kg/d	8180[a]	6952[b]	8681[c]
Concentration of solids dry wt. basis, percent	6	8	4
Specific gravity	1.03	1.04	1.02
Average daily flow rate, m^3/d	132[a]	84	213[d]
Pumping rate into each digester during the pumping cycle	0.85[e]	0.85	0.85
Influent temperature, °C	21	30	12
Volatile solids fraction before digestion	0.71[a]	0.71	0.71

[a]Value under average design conditions are given in Table 13-13 and Table 16-10.
[b]Extreme low solids to the digester is 85 percent of the average solids loading.
[c]Extreme high solids to the digester = quantity of thickened sludge withdrawn under sustained loading (Table 16-10 and Sec. 16-9-2, Step G, 1).

[d] $$\frac{8681\ \text{kg/d}}{0.04\ \text{g/g} \times 1.02 \times 1000\ \text{kg/m}^3} = 213\ \text{m}^3/\text{d}$$

[e]The pumping rate of 0.85 m^3/min (Sec. 16-9-2, Step G, 4) gives a velocity of 0.80 m/s in the 15-cm diameter pipe.

9. The ratio TVS/TS = 0.71, $Y = 0.05$ g VSS produced per gram BOD_5 utilized, $E = 0.8$, and $k_d = 0.03$ per day.
10. The digester heating shall be achieved by recirculation of sludge through the external heat exchanger. The sludge recirculation system shall also be designed to provide digester mixing.
11. Provide floating digester cover for gas collection. A separate gas storage sphere shall be provided to store excess gas.
12. The heat loss from the digester cover, side walls, and floor shall be calculated using the standard heat transfer coefficients for the digester construction material.
13. Provide a gas-fired hot water boiler for external heat exchanger. The fuel-burning equipment shall include the necessary accessories for burning sludge gas. At the time of insufficient gas supply, the burner shall change over automatically to the natural gas.
14. Explosion prevention devices shall be provided to minimize the possibility of an explosive mixture being developed inside the floating cover. Proper flame traps shall be provided to assure protection against the passage of flame into the digester, gas storage sphere, and supply lines.
15. The digester design shall include a supernatant withdrawal system, sight glass, sampler, manhole, etc.
16. Arrangements shall be provided to break the scum that may form on the sludge surface.

17-8-2 Design Calculations

Step A: Digester Capacity and Dimensions. The digester capacity may be calculated using (1) digestion period, (2) mean cell residence time equations, (3) volumetric loading, (4) population basis, and (5) observed volume reduction. The digester capacity using each of the above methods is computed below:[14]

1. Compute digester capacity at average flow condition using 15 days digestion period.

$$\begin{aligned} \text{Average flow to the digester} &= 132\ \text{m}^3/\text{d} \\ \text{Digester volume} &= \text{flow} \times \text{digestion period} \\ &= 132\ \text{m}^3/\text{d} \times 15\ \text{d} = \underline{1980\ \text{m}^3} \end{aligned}$$

2. Compute digester capacity using volatile solids-loading factor.

$$\begin{aligned} &\text{Assume VS loading at average flow condition} = 2.5\ \text{kg/m}^3{\cdot}\text{d} \\ &\text{Total volatile solids reaching the digester} = 8180\ \text{kg/d (Table 13-13)} \times 0.71\ \text{(see Design Criteria)} = 5808\ \text{kg/d} \\ &\text{Digester capacity} = \frac{5808\ \text{kg/d}}{2.5\ \text{kg/m}^3{\cdot}\text{d}} = 2323\ \text{m}^3 \end{aligned}$$

3. Compute digester capacity using volume per capita allowance.
   Assume 0.030-m^3 digester capacity per capita.

$$\text{Population served} = 80{,}000 \text{ (Chapter 6)}$$
$$\text{Digester capacity} = 80{,}000 \times 0.030 = 2400 \text{ m}^3$$

4. Compute the digester capacity using volume reduction method.

$$\text{Volume of digested sludge (Table 13-13)} = 97 \text{ m}^3/\text{d}$$
$$\text{Volume of raw sludge to the digester (Table 13-13)} = 132 \text{ m}^3/\text{d}$$

   Digester capacity is calculated from Eq. (17-1):

$$V = (132 \text{ m}^3/\text{d} - \tfrac{2}{3}(132 \text{ m}^3/\text{d} - 97 \text{ m}^3/\text{d}))\ 15 \text{ d} = 1630 \text{ m}^3$$

5. Select the digester capacity.
   The digester capacity calculated from the volume reduction method is significantly smaller than the capacity obtained from other methods. It should be mentioned that in completely mixed high-rate digesters, the supernatant withdrawal is generally achieved by stopping all mixing devices and letting the solids settle in the digester for 1–2 h. The supernatant and digested sludge are then withdrawn from the digester. Frequently, this cannot be done, and the digested sludge is withdrawn under completely mixed conditions. It is therefore desirable to provide excess capacity for stabilization and operational flexibility. Select an active digester capacity of 2500 m^3.

Step B: Digester Dimensions and Geometry

1. Correct for volume displaced by grit and scum accumulations and floating cover level.
   - Provide a 1-m depth for grit accumulation in the bottom cone.
   - Provide a 0.6-m depth for the scum blanket.
   - Provide a 0.6-m minimum clearance between the floating cover and maximum digester level.

$$\text{Total inactive cone depth} = 1 \text{ m}$$
$$\text{Total inactive upper depth of digester} = 0.6 \text{ m} + 0.6 \text{ m} = 1.2 \text{ m}$$

   Assume that the active side water depth is 8.0 m (26.3 ft).
   - Provide two digesters.

$$\text{Volume of each digester} = 1250 \text{ m}^3$$

$$\text{Area of each digester} = \frac{1250 \text{ m}^3}{8.0 \text{ m}} = 156.3 \text{ m}^2$$

$$\text{Diameter of each digester} = \sqrt{\frac{4}{\pi} \times 156.3 \text{ m}^2} = 14.1 \text{ m (46.3 ft)}$$

   Generally, floating covers come in 5-ft (1.5-m)-diameter increments. Select digesters with 13.7-m (45-ft) diameter.

$$\text{The vertical side depth} = \frac{1250\ \text{m}^3}{\frac{\pi}{4} \times (13.7\ \text{m})^2} = 8.5\ \text{m}\ (27.9\ \text{ft})$$

- Provide two digesters, each with a 13.7-m (45-ft) diameter, and an 8.5-m (28-ft) vertical side depth. The digester details are given in Figure 17-9.

2. Check the active volume of the digesters, including volume of cone.
The floor of the digester is sloped at 1 vertical to 3 horizontal. The bottom cone depth of 2.3 m adds additional volume. Therefore,

$$\begin{aligned}
\text{Active digester volume}^{c} &= (\text{volume of active cylindrical portion}) \\
&\quad + (\text{total volume of the cone}) \\
&\quad - (\text{allowance for grit accumulation}) \\
&= \frac{\pi}{4}(13.7\ \text{m})^2 \times 7.3\ \text{m} + \frac{1}{3}\left(\frac{\pi}{4}\right)(13.7\ \text{m})^2 \\
&\quad \times 2.3\ \text{m} - \frac{1}{3}\left(\frac{\pi}{4}\right)(6.0\ \text{m})^2 \times 1.0\ \text{m} \\
&= 1076.1\ \text{m}^3 + 113.0\ \text{m}^3 - 9.4\ \text{m}^3 = 1179.7\ \text{m}^3
\end{aligned}$$

$$\text{Active volume of two digesters} = 2 \times 1179.7\ \text{m}^3 = 2359.4\ \text{m}^3$$

$$\begin{aligned}
\text{Total inactive volume of two digesters} &= (\text{volume for scum accumulation and clearance}) + (\text{allowance for grit accumulation}) \\
&= 2\left[\left(\frac{\pi}{4}\right)(13.7\ \text{m})^2 \times 1.2\ \text{m} + 9.4\ \text{m}^3\right] \\
&= 372.6\ \text{m}^3
\end{aligned}$$

$$\begin{aligned}
\text{Total active + inactive volume of digesters} &= 2359.4\ \text{m}^3 + 372.6\ \text{m}^3 \\
&= 2732.0\ \text{m}^3
\end{aligned}$$

$$\text{Active volume ratio, including cone} = \frac{2359.4\ \text{m}^3}{2732.0\ \text{m}^3} = 0.86$$

An active volume ratio above 0.85 is considered efficient utilization of digester volume.

[c]Active depth of cylindrical portion = side water depth (8.5 m) − scum blanket (0.6 m) − space below floating cover (0.6 m) = 7.3 m. Grit accumulation allowance up to 1 m in the bottom cone will reduce the effective volume of the bottom cone. Grit accumulation will occur in the cone around the gas mixing diffusers and sludge withdrawl ports. Total allowance for grit accumulation still remains 9.4 m^3.

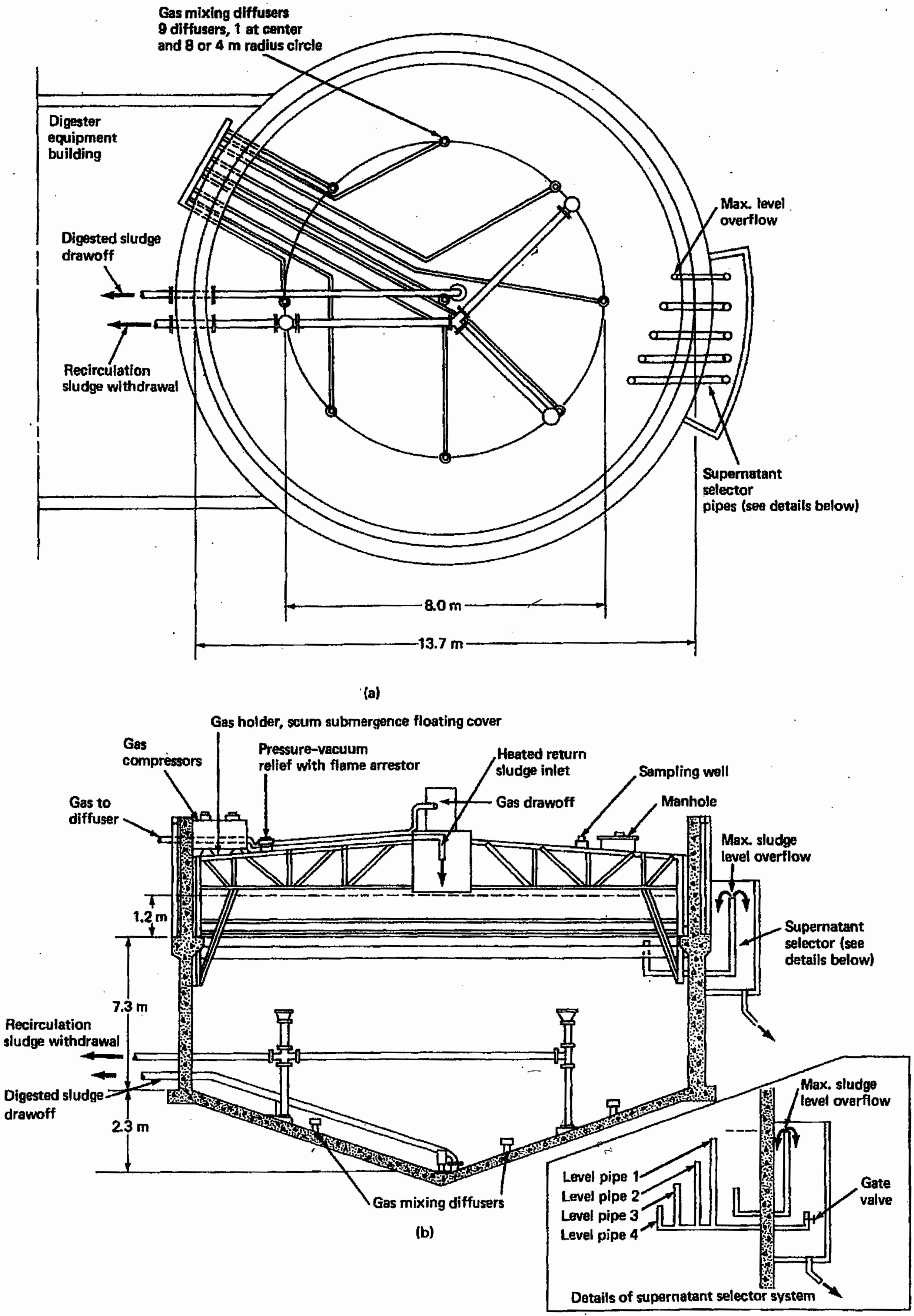


Figure 17-9 Design Details of Anaerobic Sludge Digester: (a) plan and (b) section.

Step C: Actual Solids Retention Time and Solids Loading

1. Compute actual digestion period at average, extremely low, and extremely high flows.

$$\text{Digestion period at average flow} = \frac{2359.4\ \text{m}^3}{132\ \text{m}^3/\text{d}} = 17.9\ \text{d}$$

$$\text{Digestion period at extreme high flow} = \frac{2359.4\ \text{m}^3}{213\ \text{m}^3/\text{d (Table 17-6)}} = 11.1\ \text{d}$$

$$\text{Digestion period at extreme low flow} = \frac{2359.4\ \text{m}^3}{84\ \text{m}^3/\text{d (Table 17-6)}} = 28.1\ \text{d}$$

2. Compute actual solids loadings based on volatile solids (VS) at average, extreme low, and extreme high conditions.

$$\text{Solids loading at average loading condition} = \frac{8180\ \text{kg/d} \times 0.71\ \text{VS}}{2359.4\ \text{m}^3}$$

$$= 2.5\ \text{kg VS/m}^3\cdot\text{d}$$

$$\text{Solids loading at extreme minimum loading condition} = \frac{6952\ \text{kg/d} \times 0.71\ \text{VS}}{2359.4\ \text{m}^3}$$

$$= 2.1\ \text{kg VS/m}^3\cdot\text{d}$$

$$\text{Solids loading at extreme high loading condition} = \frac{8681\ \text{kg/d} \times 0.71\ \text{VS}}{2359.4\ \text{m}^3}$$

$$= 2.6\ \text{kg VS/m}^3\cdot\text{d}$$

Step D: Gas Production. The gas production rate is estimated from several relationships. Calculations are given below.

1. Calculate gas production from Eqs. (17-2) and (17-3).
   BOD_5 in the thickened sludge (Table 13-13, stream 10) = 4253 kg/d

$$\text{BOD}_L \text{ in sludge} = 4253\ \text{kg/d} \times \frac{1}{0.68}\ \text{BOD}_5/\text{BOD}_L = 6254\ \text{kg/d}$$

$$P_x = \frac{YES_o}{1 + k_d\theta_c}$$

$$P_x = \frac{0.05 \times 0.8 \times 6254\ \text{kg/d}}{1 + 0.03/\text{d} \times 17.9\ \text{d (average condition)}}$$

$$= 163\ \text{kg/d}$$

The volume of methane gas is calculated from Eq. (17-3).

$$\begin{aligned} V &= 0.35 \text{ m}^3/\text{kg } \{(0.8 \times 6254 \text{ kg/d}) \\ &\quad - 1.42 \text{ g/g} \times 163 \text{ kg/d}\} \\ &= 1670 \text{ m}^3/\text{d} \end{aligned}$$

If methane is 66 percent in the digester gas,

$$\text{Digester gas production} = 1670 \text{ m}^3/\text{d} \times \frac{1}{0.66} = 2531 \text{ m}^3/\text{d}$$

2. Estimate gas production from other rules of thumb.
   a. Based on total volatile solids (TVS) loading using TVS = 0.71 of total solids and gas production rate of 0.50 m^3/kg TVS:

$$\begin{aligned} \text{Gas produced} &= 8180 \text{ kg/d} \times 0.71 \times 0.50 \text{ m}^3/\text{kg} \\ &= 2904 \text{ m}^3/\text{d} \end{aligned}$$

   b. Based on TVS reduction:
   Assume average TVS reduction of 52 percent and gas production of 0.9 m^3/kg TVS reduced.

$$\begin{aligned} \text{Total TVS reduced} &= 8180 \text{ kg/d} \times 0.71 \times 0.52 = 3020 \text{ kg/d} \\ \text{Gas produced} &= 3020 \text{ kg/d} \times 0.94 \text{ m}^3/\text{kg} = 2839 \text{ m}^3/\text{d} \end{aligned}$$

   c. Based on per capita:

$$\text{Total population served} = 80{,}000$$

   Use gas production rate of 0.032 m^3/capita

$$\begin{aligned} \text{Gas produced} &= 80{,}000 \text{ persons} \times 0.032 \text{ m}^3/\text{person} \cdot \text{d} \\ &= 2560 \text{ m}^3/\text{d} \end{aligned}$$

   d. Based on the above analysis, assume a conservative gas production rate of 2550 m^3/d at standard conditions (0°C and 1 atm).

Step E: Digested Sludge Production

1. Compute the quantity of solids in digested sludge.

$$\begin{aligned} \text{TVS} &= 8180 \text{ kg/d} \times 0.71 = 5807 \text{ kg/d} \\ \text{TVS destroyed} &= 5807 \text{ kg/d} \times 0.52 = 3020 \text{ kg/d} \\ \text{TS remaining after digestion} &= \text{Nonvolatile solids} + \text{VS remaining} \\ &= (8180 - 5807) \text{ kg/d} + 0.48 \\ &\quad \times 5807 \text{ kg/d} \\ &= 5159 \text{ kg/d} \end{aligned}$$

2. Compute the total mass reaching the digester.

$$\begin{aligned} \text{Total solids reaching the digester} &= 8180 \text{ kg/d} \\ \text{Total solids in thickened sludge} &= 6 \text{ percent by weight} \\ \text{Total mass reaching digester} &= (8180 \text{ kg/d})/(0.06 \text{ kg/kg}) \\ &= 136{,}317 \text{ kg/d} \end{aligned}$$

3. Compute the volume and TSS in digested sludge and the digester supernatant. Assume that no liquid volume change occurs in the digester.

$$\begin{array}{c} \text{Volume of influent} \\ \text{thickened sludge} \\ (V_{\text{influent}}) \end{array} = \begin{array}{c} \text{Volume of digested} \\ \text{sludge removed} \\ \text{from digester} \\ (V_{\text{sludge}}) \end{array} + \begin{array}{c} \text{Volume of digester} \\ \text{supernatant} \\ (V_{\text{supernatant}}) \end{array}$$

$$\begin{array}{c} \text{TSS remaining} \\ \text{in digested sludge} \\ (W_{\text{remaining}}) \end{array} = \begin{array}{c} \text{TSS in digested} \\ \text{sludge removed} \\ \text{from digester} \\ (W_{\text{sludge}}) \end{array} + \begin{array}{c} \text{TSS lost in digester} \\ \text{supernatant} \\ (W_{\text{supernatant}}) \end{array}$$

$$V_{\text{sludge}} = 132 \text{ m}^3/\text{d}$$

$$W_{\text{remaining}} = 5159 \text{ kg/d}$$

$$V_{\text{sludge}} = \frac{W_{\text{sludge}}}{0.05 \times 1030 \text{ (see the Design Criteria for solids content and sp. gr. of digested sludge}}$$

$$V_{\text{supernatant}} = \frac{W_{\text{supernatant}}}{0.004 \times 1000 \text{ (see Design Criteria for TSS in supernatant)}}$$

$$132 \text{ m}^3/\text{d} = \frac{W_{\text{sludge}}}{0.05 \times 1030} + \frac{W_{\text{supernatant}}}{0.004 \times 1000}$$

$$W_{\text{supernatant}} = \text{Total solids remaining after digestion in digested sludge} - W_{\text{sludge}}$$

$$= 5159 \text{ kg/d} - W_{\text{sludge}}$$

Substitute the value of $W_{\text{supernatant}}$ in the previous equation and solve for W_{sludge}. Calculate other values. All mass and volume values for digested sludge and supernatant are calculated and summarized below:

$$W_{\text{sludge}} = 5021 \text{ kg/d} \qquad W_{\text{supernatant}} = 138 \text{ kg/d}$$
$$V_{\text{sludge}} = 98 \text{ m}^3/\text{d} \qquad V_{\text{supernatant}} = 35 \text{ m}^3/\text{d}$$

It may be noted that these values are close to those obtained in mass balance analysis and are provided in Table 13-13 (streams 12 and 13).

4. Determine the mass and concentrations of the components in digested sludge and supernatant.
The values for BOD_5, Org.-N, NH_4-N, NO_3-N, TN, NPP (nonprecipitated phosphorus), and PP (precipitated phosphorus) have been calculated in mass balance analysis. The step-by-step calculations for the first iteration are provided in Sec. 13-11-4, Steps A, 9 and A, 10. The final values are summarized in Table 13-13 under streams 12 and 13. These values are provided in Table 17-7. There may be a slight difference (less

TABLE 17-7 Characteristics of Digested Sludge and Digester Supernatant (Table 13-13)

Parameter	Digested Sludge, kg/d (Stream 12)	Supernatant (Stream 13)	
		kg/d	mg/L
Flow, m^3/d	97 (98[a])	35 (35[a])	—
TSS	5008 (5021[a])	140 (138[a])	4000 (3942[a])
BOD_5	1596	105	3000
Org.-N	320	19	533
NH_4-N	44	16	453
NO_3-N	0	0	0
TN	364	35	986
NPP	67	7.4	211
PP	126	—	—
TP	193	7.4	211
TVSS/TSS, ratio	0.54		
Biodegradable solids/TSS	0.33		
Org.-N/TVSS	0.12		
NPP/TVSS	0.025		

[a]Values in parentheses are calculated in Step E, 3 and provided for comparison. These values are slightly higher because of sustained loading effect.

than 0.5 percent) in these values if calculated from the increased sludge quality used for thickener and digester design.

5. Select a supernatant selector system.
   The purpose of the supernatant selector is to withdraw liquid from the top. Ideally, it should serve the following purposes: (1) allow direct visual inspection of the sludge; (2) allow removal of clear liquid from the top; (3) permit operation by one person; (4) be extremely reliable, with a minimum of moving parts; (5) minimize the danger of allowing air to enter the digester; and (6) be easy to service in case of blockage by grease, scum, or sludge.

   Many types of supernatant selector and control systems are available from equipment manufacturers. One simple type is used in this design and is shown in Figure 17-9. The clogged supernatant selector pipe is cleaned by pumping hot water into the digester through the pipe.

Step F: Influent Sludge Line to the Digester. The sludge from the thickeners is pumped into the digester control building. The sludge pipe is 15 cm (6 in.) in diameter, and pump operation is intermittent with a constant pumping rate of 0.8 m^3/min from each thickener. The pumping operation is controlled by a time clock that can be manually varied to achieve the thickest possible sludge. The pumping cycle is adjusted such that both pumps do not operate simultaneously. The thickened sludge from each digester is di-

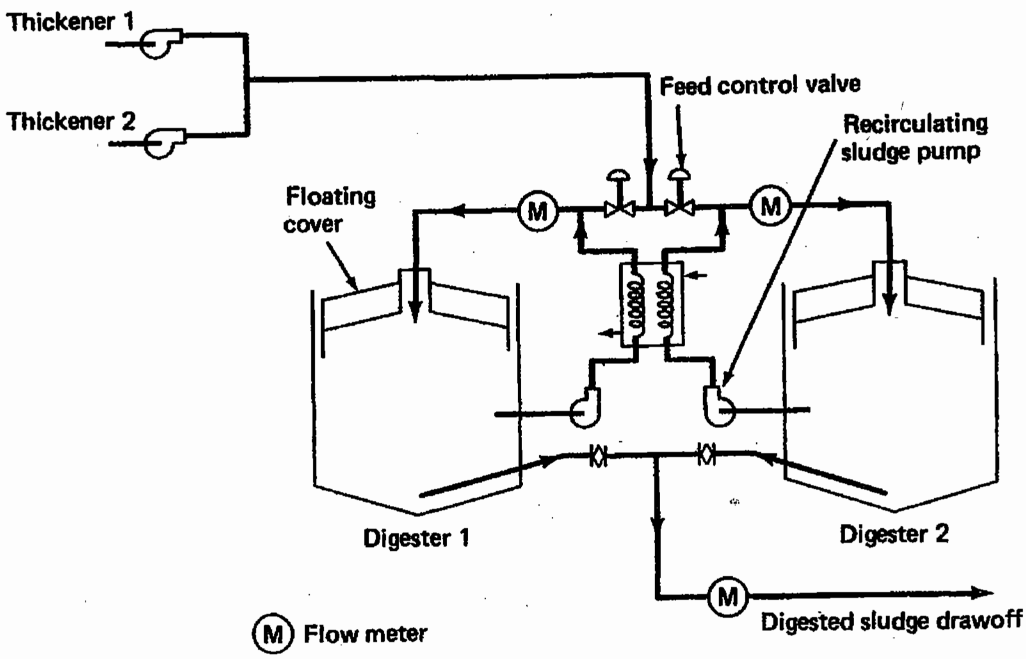


Figure 17-10 Schematic Flow Scheme of Sludge Heating and Recirculation System.

vided equally in the digesters. Details of the influent sludge pipe are shown in Figure 17-10.

Step G: Digester Heating Requirements

1. Compute the heating required for raw sludge.
   Heating required for sludge is calculated from Eq. (17-4):

$$H_R = \text{flow} \times C_p(T_2 - T_1) \tag{17-4}$$

where

H_R = heat required, J/d
C_p = specific heat of sludge (same as for water = 4200 J/kg·°C or 1.0 Btu/lb·°F)
T_2 = digestion temperature, °C
T_1 = temperature of the thickened sludge, °C

The critical heat requirement for raw sludge is reached when sludge flow is maximum and influent temperature is lowest:

$$\begin{array}{l}\text{Heat required} \\ \text{(see Table 17-6)}\end{array} = \frac{8681 \text{ kg/d} \times 4200 \text{ J/kg°C } (35 - 12)\text{°C}}{0.035 \text{ kg/kg}}$$

$$= 2.39 \times 10^{10} \text{ J/d}$$

2. Compute heat loss from the digester.
   Heat loss is calculated from Eq. (17-5):

$$H_L = UA(T_2 - T_1) \tag{17-5}$$

where

H_L = heat loss, J/h (Btu/h)
U^d = overall coefficient of heat transfer, $J/s{\cdot}m^2{\cdot}°C$ ($Btu/h{\cdot}ft^2{\cdot}°F$)
A = area through which heat loss occurs, m^2 (ft^2)
T_2 = digester operating temperature, °C (°F)
T_1 = outside air temperature, °C (°F)[e]

Heat losses from the digester occur from the roof, bottom, and side walls

a. Compute the area of the roof.

$$\text{Roof area} = \pi D\left(\frac{\text{slant length}}{2}\right)$$

$$\text{Slant height} = \sqrt{\left(\frac{D}{2}\right)^2 + (\text{vertical rise of cover})^2}$$

$$= \sqrt{\left(\frac{13.7\ \text{m}}{2}\right)^2 + (0.46\ \text{m})^2} = 6.87\ \text{m}$$

$$\text{Roof area} = (\pi \times 13.7\ \text{m} \times 6.87\ \text{m})/2 = 147.9\ \text{m}^2$$

b. Compute the area of side walls.

Area of side wall above ground level = $\pi D \times$ (exposed height)

Assume 50 percent side wall is exposed.

$$\text{Side wall area above ground} = \pi \times 13.7\ \text{m} \times \frac{8.5}{2}\ \text{m} = 182.9\ \text{m}^2$$

$$\text{Area of side wall below ground} = 182.9\ \text{m}^2$$

c. Compute the bottom area.
Digester bottom is generally sloped at 1 vertical to 3 horizontal. Therefore,

$$\text{Total drop of the bottom slope at the center} = \frac{D}{2 \times 3} = \frac{13.7\ \text{m}}{2 \times 3} = 2.3\ \text{m}$$

$$\text{Bottom area} = \pi \times 13.7\ \text{m} \times \tfrac{1}{2} \times \sqrt{\left(\frac{13.7\ \text{m}}{2}\right)^2 + (2.3\ \text{m})^2}$$

$$= 155.5\ \text{m}^2$$

d. Select the overall coefficients of heat transfer for different areas.
- digester floating covers and roofing consisting of 6.5-mm ($\frac{1}{4}$-in.) plate steel,

[d] $J/s{\cdot}m^2{\cdot}°C \times 0.1763 = Btu/h{\cdot}ft^2{\cdot}°F$.

[e] Critical average air and ground temperatures are 0 and 5°C, respectively.

76-mm (3-in.) rigid foam insulation,[f] inside air space, and buildup roofing 1236 kg/m^2 (70 lb/ft^2), $U = 0.90$ $J/s{\cdot}m^2{\cdot}°C$

- exposed digester side 300-mm (12-in.) concrete, 76-mm (3-in.) urethane foam insulation, 100-mm (4-in.) brick siding, $U = 0.68$ $J/s{\cdot}m^2{\cdot}°C$
- buried digester side 300-mm (12-in.) concrete surrounded by moist soil, $U = 0.80$ $J/s{\cdot}m^2{\cdot}°C$
- digester bottom surrounded by moist soil, $U = 0.62$ $J/s{\cdot}m^2{\cdot}°C$

e. Compute heat loss from the digester.[g]

Heat loss from the cover and roofing $= 147.9\ m^2 \times 0.9\ J/s \cdot m^2 \cdot °C\ (35 - 0)°C \times 86{,}400\ s/d$

$= 4.03 \times 10^8\ J/d$

Heat loss from exposed wall $= 182.9\ m^2 \times 0.68\ J/s{\cdot}m^2{\cdot}°C \times (35 - 0)°C \times 86{,}400\ s/d$

$= 3.76 \times 10^8\ J/d$

Heat loss from buried wall $= 182.9\ m^2 \times 0.8\ J/s{\cdot}m^2{\cdot}°C \times (35 - 0)°C \times 86{,}400\ s/d$

$= 4.43 \times 10^8\ J/d$

Heat loss from bottom $= 155.5\ m^2 \times 0.62\ J/s{\cdot}m^2{\cdot}°C \times (35 - 5)°C \times 86{,}400\ s/d$

$= 2.50 \times 10^8\ J/d$

Total heat loss from each digester $= 14.72 \times 10^8\ J/d$
Total heat loss from both digesters, including 20 percent minor losses, and 25 percent emergency condition $= 14.72 \times 10^8\ J/d \times 2 \times 1.45 = 42.69 \times 10^8\ J/d$

f. Compute the heating requirements for the digester.

Heating requirements for raw sludge under critical condition $= 2.39 \times 10^{10} J/d$
Heat loss from the digester $= 42.69 \times 10^8\ J/d$
Total heating requirement $= 2.82 \times 10^{10}\ J/d$
$= 1.175 \times 10^9\ J/h$
$= 1.175 \times 10^6\ kJ/h$

Step H: Selection of Heating Units and Energy Balance

1. Select heating units for external heat exchanger.
   Provide two heating units, each rated at 1.25×10^6 kJ/h (1.19 million Btu/h) with natural gas. The digester gas has approximately 65 percent of the heating value of the

[f]Common insulating materials are glass wool, insulation board, urethane foam, lightweight insulating concrete, dead air space, etc.

[g]Critical heat loss occurs under average winter condition.

natural gas.[h] Therefore, each unit will be derated at 0.813×10^6 kJ/h (0.77 million Btu/h). Total heat provided by two units $= 2 \times 0.813 \times 10^6 = 1.626 \times 10^6$ kJ/h.

$$\text{Percent extra capacity available} = \frac{(1.626 \times 10^6 - 1.175 \times 10^6)\ \text{kJ/h} \times 100}{1.175 \times 10^6\ \text{kJ/h}}$$

$$= 38 \text{ percent}$$

The heating requirements are determined at the critical conditions of sludge flow and external temperatures. Therefore, these values are used to size the equipment. The actual average heat requirements would be substantially less.

2. Compute the digester gas requirements.
At 75 percent efficiency of the heating units, the digester gas requirements are calculated as follows:

$$\text{Digester gas needed} = \frac{1.626 \times 10^6\ \text{kJ/h}}{0.75 \times 24{,}300\ \text{kJ/m}^3}$$

$$= 89.22\ \text{m}^3/\text{h}$$

$$= 89.22\ \text{m}^3/\text{h} \times 24\ \text{h/d} = 2141\ \text{m}^3/\text{d}$$

$$\text{Total quantity of digester gas produced} = 2550\ \text{m}^3/\text{d}$$

This gives approximately 20 percent excess gas under the most critical condition when the digester heating demand is greatest. Under average and minimum digester heating requirements, a considerably larger quantity of excess gas will be available. Excess gas will also be used to produce heated water for other plant uses.

3. Design makeup heat exchangers for external sludge heating.
The heating of the digesters and thickened sludge is achieved by recirculating digester contents through external heat exchangers.
   a. Compute the average temperature rise of the sludge through the external exchangers.
   Provide a 23-cm- (9-in.) diameter sludge recirculation pipe and a constant flow recirculation pump for each digester. A common external jacketed-type heat exchanger will be used to heat the recirculated sludge. If a velocity of 1 m/s is maintained in the pipe,

$$\text{The sludge recirculation pumping rate from each digester} = \frac{\pi}{4} \times (0.23\ \text{m})^2 \times 1\ \text{m/s} \times (86{,}400)\ \text{s/d}$$

$$= 3590\ \text{m}^3/\text{d}$$

$$= 3590\ \text{m}^3/\text{d} \times 1020\ \text{kg/m}^3$$

$$= 3.662 \times 10^6\ \text{kg/d}$$

   Average sludge temperature entering the external heat exchanger = 35°C.
   Assume the average sludge temperature increase after passing through the heat

[h]Heating value of natural gas = 37,300 kJ/m³. Heating value of digester gas = 23,000 kJ/m³.

exchanger = ΔT°C. Assume the specific heat of sludge is 4200 J/kg°C (same as for water).

$$\text{Total heat supplied to the sludge} = 4200 \frac{\text{J}}{\text{kg°C}} \times \Delta T\text{°C} \times 3.662 \times 10^6 \text{ kg/d}$$

$$= (1.538 \times 10^{10}) \times \Delta T \text{ J/d}$$

Total heat requirement for each digester $= 2.82 \times 10^{10}$ J/d (Step G, 2, f)/2 $= 1.41 \times 10^{10}$ J/d

If the efficiency of the heat exchangers is 80 percent,

$$(1.538 \times 10^{10}) \times \Delta T \text{ J/d} \times 0.8 = 1.41 \times 10^{10} \text{ J/d}$$

where

$$\Delta T = \frac{1.41 \times 10^{10} \text{ J/d}}{1.538 \times 10^{10} \text{ J/d} \times 0.8} = 1.15\text{°C}$$

Average temperature of the sludge entering heat exchanger = 35°C. Average temperature of the sludge leaving the heat exchanger = 36.15°C. Sludge recirculation of 3590 m^3/d (660 gpm) in each digester will also provide digester mixing.

b. Compute the hot water recirculation rate through the external heat exchanger. Provide one jacketed pipe heat exchanger for both digesters. The hot water is pumped countercurrent to the sludge flow through the jacket surrounding the sludge pipes. Assume that the water enters the jacket pipe at 95°C and leaves at 60°C.

Drop in heating water temperature = 95°C − 60°C = 35°C

Total heating required for each digester $= 1.41 \times 10^{10}$ J/d

If 25 percent additional heating is provided to account for heat losses,

Total heat required per digester $= 1.41 \times 10^{10} \times 1.25 = 1.76 \times 10^{10}$ J/d

Total heat required for both digesters $= 3.52 \times 10^{10}$ J/d (1.47×10^6 kJ/h)

Total heat available in digester gas = 23,000 kJ/m^3 × 1.162 kg/m^3 (see footnote h) × 2550 m^3/d × 1000 J/kJ

$= 6.82 \times 10^{10}$ J/d

Using specific heat of water = 4200 J/kg°C

Total heat supplied by water = 4200 J/kg°C × 35°C = 147,000 J/kg

Hot water recirculation rate through the common heat exchanger $= \dfrac{3.52 \times 10^{10}\ \text{J/d}}{147{,}000\ \text{J/kg}}$

$= 2.40 \times 10^5$ kg/d

Volume of water recirculated $= \dfrac{2.4 \times 10^5\ \text{kg/d}}{1000\ \text{kg/m}^3}$

$= 240$ m³/d (44 gpm)

c. Compute the length of sludge pipe in the heat exchanger jacket.

Average temperature of the sludge in the heat exchanger $= \dfrac{35°\text{C} + 36.15°\text{C}}{2} = 35.58°\text{C}$

Average temperature of the heating water in the heat exchanger $= \dfrac{95°\text{C} + 60°\text{C}}{2} = 77.5°\text{C}$

Assume heat transfer coefficient of external water jacketed heat exchanger $= 4000\ \text{kJ/h·m}^2\text{·°C}$ (196 Btu/h·ft²·°F)

Total heat radiated from the heating water $= (77.5 - 35.58)°\text{C} \times 4000\ \text{kJ/h·m}^2\text{·°C} \times 24\ \text{h/d} = 4.02 \times 10^6\ \text{kJ/d·m}^2$

Total surface area of the sludge pipe for each heat exchanger $= \dfrac{1.76 \times 10^{10}\ \text{J/d}}{4.02 \times 10^6\ \text{kJ/d·m}^2 \times 1000\ \text{J/kJ}}$

$= 4.38\ \text{m}^2$

Length of 23-cm- (9 in.) diameter jacketed pipe $= \dfrac{4.38\ \text{m}^2}{\pi(0.23\ \text{m})} = 6.0\ \text{m}$

Provide a 6-m-long, 23-cm-diameter heat exchanger sludge pipe per digester into a common hot water jacket. The flow scheme of a makeup heat exchanger is shown in Figure 17.10.

Step I: Gas Storage and Compressor Requirements

1. Compute the diameter of the gas storage sphere.
   Provide a total of 3-d gas storage to serve the digester heating requirements and other plant uses.

Total gas stored $= 3\ \text{d} \times 2550\ \text{m}^3/\text{d}$
$= 7650\ \text{m}^3$ (standard condition; 0°C and 1 atm)
Storage pressure $= 5.1$ atm (assume)
Storage temperature $= 50°\text{C}$ summer

Storage volume $V_2 = \dfrac{P_1 V_1 T_2}{P_2 T_1}$

where P_1, V_1, and T_1 are pressure, volume, and absolute temperature, respectively, of the digester gas produced; and P_2, V_2, and T_2 are the storage pressure, volume, and absolute temperature, respectively, of the digester gas stored.

$$V_2 = \frac{1 \text{ atm} \times 7650 \text{ m}^3 (273 + 50)°\text{K}}{5.1 \text{ atm} (273 + 0)°\text{K}}$$

$$= 1774.7 \text{ m}^3$$

Provide a high-volume gas storage sphere:

$$\text{Volume of sphere} = \frac{\pi}{6} (\text{diam})^3$$

$$\text{Diam of sphere} = \left[\frac{1774.7 \text{ m}^3 \times 6}{\pi}\right]^{1/3}$$

$$= 15.0 \text{ m (49 ft)}$$

Provide a 15.0-m- (49-ft) diameter sphere for gas storage.

2. Compute the size of high-pressure gas compressors.
Compressors are used to compress the digester gas into the gas storage sphere. Compressor power requirement is calculated from Eq. (13-67). Total weight of digester gas produced under standard conditions = 2550 kg/d (see Sec. 17-7-2, Step D, 2d).
    Assuming the weight of gas compressed is 200 percent of the production rate, then

$$w = 2 \times 2550 \text{ kg/d} \times \frac{1}{24 \text{ h/d} \times 3600 \text{ s/h}} = 0.0590 \text{ kg/s}$$

R = 8.314 kJ/kmole °K
e = compressor efficiency of 75%
T_o = inlet temperature = (273 + 35)°K
P_o = 1.03 atm (gas pressure inside the floating cover is normally less than 0.4 m of water)
P = 5.10 atm

$$P_w = \frac{0.0590 \text{ kg/d} \times 8.314 \text{ kJ/kmole °K} \times (273 + 35) °\text{K}}{8.41 \times 0.75 \text{ kg/kmole}} \times \left[\left(\frac{5.1}{1.03}\right)^{0.283} - 1\right]$$

$$= 13.7 \text{ kW (18.3 hp)}$$

Provide two constant-speed compressors, each driven by a 7.5-kW (10-hp) electric motor.

Step J: Digester Gas Mixing

1. Compute power requirements for gas mixing.

Power requirements for gas mixing of the digester is obtained from Eq. (12-6). The volume of each digester V = 1179.7 m^3.

$$\begin{aligned}\mu &= 2 \text{ times the viscosity of water at } 35°\text{C} \\ &= 2 \times 0.73 \times 10^{-3} \text{ N s/m}^2 \ (2 \times 1.51 \times 10^{-5} \text{ lb·s/ft}^2) \\ &= 1.46 \times 10^{-3} \text{ N s/m}^2 \ (3.02 \times 10^{-5} \text{ lb·s/ft}^2)\end{aligned}$$

A velocity gradient of 75/s or higher is needed for proper mixing of sludges above 5 percent solids is over 75/s. Use $G = 85/\text{s}$ for this problem.

$$\begin{aligned}P &= (85/\text{s})^2 \times 1.46 \times 10^{-3} \text{ N·s/m}^2 \times 1179.7 \text{ m}^3 \\ &= 12{,}444 \text{ N m/s (9178 ft lb/s)} \\ &= 12.4 \text{ kW (16.6 hp)}\end{aligned}$$

Total power required for two digesters $= 2 \times 12.4 \text{ kW} = 24.8 \text{ kW (33.2 hp)}$

Provide three compressors, each driven by 15-kW (20-hp) motor.

Total power provided for mixing = 45 kW (60 hp)

Two compressors will deliver the required power, while the third compressor will be a standby unit, serving both digesters.

2. Compute the gas flow.
The digester gas flow rate for mixing is calculated from Eq. (13-70).

$P_w = 30$ kW
$R = 8.314$ kJ/kmole °K
$T_o = (273 + 35) = 308$°K
$P = 2.4$ atm. This is sufficient to overcome the static sludge head in the digester and head losses in the piping.
$P_o = 1.03$ atm (gas pressure inside the floating digester cover)
e = compressor efficiency of 75%
w = mass of gas compressed per digester, kg/s

$$w = \frac{15.0 \text{ kW} \times 8.41 \text{ kg/kmole} \times 0.75}{8.314 \text{ kJ/kmole °K} \times 308 \text{ °K}\left[\left(\dfrac{2.4}{(1.03)}\right)^{0.283} - 1\right]}$$

$$= 0.14 \text{ kg/s}$$

Gas flow[i] per digester $= \dfrac{0.14 \text{ kg/s}}{1.162 \text{ kg/m}^3 \times 0.86} = 0.14 \text{ m}^3/\text{s}$

3. Select the digester-mixing arrangement.
The digester mixing will be achieved by flow recirculation, raw sludge, and internal gas mixing. The purpose of digester mixing is to transfer the required quantity of heat and aid in the mixing of digester contents so that the digestion efficiency is maintained.

[i]The density of digester gas is 86 percent of air. Weight of air is assumed to be 1.162 kg/m^3 (0.07251 lb/ft^3).

The sludge recirculation of 3590 m^3/d (658 gpm) in each digester was calculated in Step H, 3. The sludge will be withdrawn from the mid-depth and discharged above the scum blanket level to break the scum layer (see Figure 17-10).

A multipoint gas-mixing system is provided for the effective use of the gas. The gas is withdrawn from the top and recirculated by means of nine ports for gas injection. One gas injection diffuser is located in the center, and eight are equally spaced around a 4-m radius circle. Details are shown in Figure 17-9. Details of diffusers are covered in the specifications (Sec. 17-9-2).

17-9 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT ANAEROBIC SLUDGE DIGESTION FACILITY

The operation and control of anaerobic digesters is difficult because it depends not only on the results of the laboratory tests, but also on operators' judgment and skills, treatment plant loadings, industrial wastes, and weather conditions. The routine operations are further complicated by the need for repairs, shutdown, cleaning, and startup. EPA has published an operations manual for anaerobic sludge digestion, which provides very detailed information on process and troubleshooting.[16] Following is the summary information on digester startup, troubleshooting, and routine operation and maintenance.[15-18]

17-9-1 Digester Startup

If sufficient digested sludge seed is available from a nearby treatment plant, the startup operation is simplified. The following steps are necessary:

1. Haul approximately 250 m^3 seed and transfer it into one digester.[j]
2. Fill the digester with raw wastewater.
3. Start heating and mixing, and bring to operating temperature.
4. Begin feeding raw sludge at a uniform rate approximately 25 percent of daily feed per digester. Increase the loading gradually.
5. Maintain the following records:
   a. Quantity of TVS fed daily
   b. TVS, volatile acids and alkalinity ratio (VA/ALK), and pH
   c. Temperature, gas production, and CO_2 content in gas
6. At a low feeding rate, it is possible to begin normal operation without adding chemicals for pH control. If the VA/ALK ratio rises to 0.8 or more and pH drops below 6.5, addition of chemicals such as lime or soda ash may be considered.
7. Fairly stable conditions should be reached in 30–40 days if loading is kept below 1.0 kg VS/m^3·d.

[j]The amount of seed required is approximately four to five times the anticipated volatile solids in the raw daily sludge. Raw TVS per digester = 8180/2 × 0.71 = 2904 kg/d. Seed quantity = 4.5 × 2904 kg/d × 1/0.05 × 1/1030 = 254 m^3

17-9-2 Common Operational Problems and Troubleshooting Guide

1. A rise in the VA/ALK ratio (greater than 0.3), increase in CO_2 content, decrease in pH, and rancid or H_2S odors are indications of hydraulic or organic overloading, excessive withdrawal of digested sludge, or incoming toxic materials. The problem may be overcome by decreasing organic loading, adding seed from another digester, decreasing the sludge withdrawal rate, increasing the mixing rate and mixing time, exerting proper temperature control, and instituting an industrial pretreatment program.
2. Poor supernatant quality may be caused by excessive mixing, insufficient settling time before sludge withdrawal, too low a supernatant drawoff point, and insufficient sludge withdrawal rate. The problem may be solved by reducing mixing, allowing a longer time for settling, using higher supernatant withdrawal ports, and increasing the digested sludge withdrawal.
3. Foam in the supernatant may be caused by the scum blanket breaking up, excessive gas recirculation, and organic overload. Stop withdrawal supernatant, throttle compressor output, and reduce feeding rate.
4. Thin digested sludge may be caused by short-circuiting, excessive mixing, or too high a sludge-pumping rate. Stop mixing several hours before supernatant draw off and sludge withdrawal. Proper selector level should be used for supernatant removal. The sludge withdraw should be made by a short pumping cycle.
5. Falling sludge temperature may be caused by plugged sludge recirculation lines, inadequate mixing, hydraulic overload, lower water feed rate in heat exchangers, and the boiler burner not firing. The problem can be overcome by back-flushing the sludge recirculation lines by heated and digested sludge and checking and correcting the boiler and heat exchangers.
6. Too high a sludge temperature may be caused by a faulty controller, boiler and hot water temperature too high, and high hot water recirculation rate. Check and take proper action.
7. Insufficient mixing may be caused by gas mixer feed lines being plugged and gas flow being too small. Clean gas lines and valves. Increase the capacity of the compressor.[k]
8. Low gas pressure in the digester may be caused by gas leaking from the pressure relief valve (PRV), digester cover, gas lines, and hoses; too high a gas withdrawal rate; and high supernatant withdrawal. Check and repair leaks, and control gas and supernatant withdrawal rates.
9. High gas pressure in the digester may be caused by insufficient gas withdrawal, PRV being stuck, or not opening because of freezing or defect. Increase gas withdrawal and correct PRV.
10. High scum blanket generally is caused by a plugged supernatant overflow. Lower the liquid level in the digester using bottom drawoff pipes, and then rod the supernatant line.
11. Too thick scum is caused by a lack of mixing and high grease contents. Break the scum manually, increase the mixing, increase the sludge recirculation to discharge liquid above the scum, or use chemicals to soften the blanket.

[k]A gas recirculation rate of 5–10 m^3/·min·1000 m^3 of digester capacity is sufficient. In some cases a recirculation rate of 20 m^3/min·1000 m^3 may be necessary.

12. A tilting floating cover may be caused by uneven distribution of load, thick scum accumulation around the edges, broken rollers or guide, or rollers out of adjustment. The problem can be overcome by distributing the ballast or weight around, breaking the scum, and readjusting the rollers. It may be necessary to lower the level of liquid in the digester.
13. Binding cover (even when rollers and guides are free) may be caused by damaged internal guides or guy wires; lower the liquid level. If cover does not go all the way down, use a crane to secure the cover in one position, and then drain the digester and repair the guides.

17-9-3 Routine Operation and Maintenance

Routine digester operation must utilize the laboratory results to protect the digester from upset. The key operational goals are to (1) minimize excess water, (2) control organic loading, (3) control temperature, (4) control mixing, (5) reduce accumulation of scum, and (6) withdraw supernatant that is low in solids.

Monitoring Program for Process Control. The following important tests must be performed daily for control of the digestion process:

1. Volatile solids (VS) and total alkalinity (TA)
2. Gas production rate and composition (CH_4 and CO_2)
3. pH
4. Volatile solids reduction
5. Digester temperature
6. Feed sludge volume and VS
7. Supernatant volume and TSS and BOD
8. Digested sludge volume and VS
9. Visual gas test (a yellow flame with blue at the base is normal; too much blue and the inability to stay lit indicates too much CO_2; orange flame with smoke indicates H_2S)
10. Sniff test; simply smelling the gas, supernatant, and digested sludge may give an indication of septic, sour, putrid, well digested, or presence of chemicals such as oils, solvents, sulfides, etc.

Results of these tests should be fully utilized in operation and control and in troubleshooting.

Routine Operation and Maintenance Checklist. The following checklist should be used for routine operation and maintenance of high-rate anaerobic digesters.

Feed Sludge

1. Record the daily volume pumped for a 24-h period.
2. Perform a daily total solids test, and make sure that there is not too much water being fed.

3. Check daily pump operations for packing gland leaks, proper adjustment of cooling water, unusual noises, undue bearing heat, and suction and discharge pressures.
4. Monitor feed pump time clock operation for on-and-off and running time cycle. Also check the sludge consistency with these time cycles.

Recirculated Sludge

1. Record the daily temperature and flow of recirculated sludge.
2. Collect samples of recirculated sludge two to three times per week and determine pH, alkalinity, TS, TVS, etc.
3. Check daily boiler temperatures, burner flame, and exhaust fan for proper operation.
4. Check daily temperature and flow of recirculating hot water.
5. Check daily and record heat exchanger inlet and outlet temperatures.
6. Check weekly for leaks in sludge lines.
7. Check daily pump operations—packing gland leaks, proper adjustment of cooling water, unusual noises, undue bearing temperatures, and suction and discharge pressures.

Digesters

1. Daily check gas manometers for proper digester gas pressure.
2. Daily drain the condensate traps.
3. Daily drain sediment traps.
4. Daily check the gas burner for proper flame.
5. Daily record the floating cover position, check cover guides, and check for gas leaks.
6. Daily record digester and natural gas meter readings.
7. Daily check and record fuel oil.
8. Daily check gas-mixing equipment for flow of gas to all feed points.
9. Daily check pressure relief and vacuum breaker valves. Verify operation with manometer and check for leaking gas.
10. Daily check supernatant tubes for proper operation, collect sample, and hose down supernatant box.
11. Daily check the level and condition of the water seal on digester cover.
12. Daily check the flow meters for correct flow, leaks, and vibration.
13. Daily check the feed sludge density meter for correct density, leaks, and other items specified by the manufacturer.
14. Daily check the scum blanket through sight glass.
15. Daily check the gas storage tank for gas leaks and odors. Record readings on pressure gauges and drain condensate traps.

17-10 SPECIFICATIONS

Anaerobic digesters utilize complex equipment and controls. Brief specifications of various components are given below to describe many components that could not be fully covered in the Design Example. The design engineer should work closely with the equipment manufacturers to develop the detailed specifications for the selected components.

17-10-1 Digester Cover

General. Furnish two floating covers for 13.7-m (45-ft) diameter digesters. The cover shall be arranged to float directly on the liquid contents of the tank, thereby providing positive submergence of all materials that tend to float. The gas holder covers shall be provided with spiral guides to engage rollers set in the concrete wall to assure that the cover is level at all times. The cover shall be designed to receive a welded steel plate roof and shall be made up of fabricated assemblies that shall be piecemarked for erection. The top cover shall be designed to accommodate a housing containing gas-mixing compressors and all appurtenances.

Material and Fabrication

1. All structural steel shall comply with the ASTM specifications for structural steel.
2. The digester cover shall be designed for the following loadings: dead load = 98 kg/m^2 (35 lb/ft^2) and live, snow, and vacuum load = 140 kg/m^2 (50 lb/ft^2).
3. The materials to form the completed floating cover shall be shipped in subassemblies, and the gas dome and appurtenances shall be assembled at the site.
4. All shop welding shall be shielded arc welding and shall conform to the latest standards of the American Welding Society.
5. The installed weight of the floating cover shall be sufficient to provide a static gas pressure of at least 0.2 m (8 in.) of water.
6. After erection, the gas holder shall be tested for gas tightness and leaks as specified by the designer.

Appurtenances. In addition to the main assemblies, the following accessories shall be provided in each digester:

- a pressure equalization unit
- a 76-cm (30-in.) entrance hatch
- a 1.22-m (48-in.) manhole with bolted and gasketed cover
- six 20-cm (8-in.) sampling wells with a gas-tight, quick-opening cover
- a 60-cm (24-in.) gas pipe housing and combination pressure and vacuum relief assembly of the unit-weighted diaphragm-operated type. The weights provided shall allow adjustment of the relief pressure in increments of 0.6 cm.

17-10-2 Mixing

General. Furnish and install a complete gas-mixing system utilizing an external compressor furnishing gas at a controlled rate through feed lines to diffusers mounted on the digester floor.

Gas Diffusers. The gas-mixing equipment shall comprise nine 2.5-cm (1-in.) stainless steel gas lines through the digester wall, as shown in Figure 17-9; nine gas diffusers mounted on the floor with PVC connecting pipes and gas manifold; gas filter, flame trap shutoff valves; and gas metering for each line.

Compressor. Provide three parallel combination, identical gas compressors, each driven by a 15-kW motor. Each compressor shall deliver a minimum of 9 m^3/min (317 cfm) gas against a pressure of 2.4 atm with suction conditions of 1.03 atm. Each compressor shall have an explosion-proof motor with belt drive, guard, pressure gauge, pressure relief valve, and shutoff valve for intake and discharge side of the compressor. Furnish each compressor with (1) a combination flame trap and thermal shutoff valve with fusible elements, sediment trap assembly, and gas filter on inlet side and (2) a motorized valve on the discharge line to provide a positive shutoff when the compressor is without power supply. All gas compressors, valves, controls, internal piping, and wiring shall be contained in fabricated steel housings.

17-10-3 Heater and Heat Exchanger

General. Furnish and install boiler and heat exchanger units for heating the recirculated sludge. The boiler and heat exchangers must be an integral unit, designed for a working pressure of 207 kPa[1] (30 psi) and constructed in accordance with the ASME Low Pressure Heating Boiler Code.

Boiler. Provide two boilers, each having a rating of 0.813×10^6 kJ/h (0.77 million Btu/h) with a digester gas consumption of 45 m^3/h. The boiler shall have refractory lined access at the rear for easy servicing of fire tubes and the fire box. The boiler shall contain inspection ports, a combination pressure gauge, low water cutoff, safety devices, sensing elements, blowers and hot water recirculating pumps, and all other appurtenances and controls for complete installation and operation as required. Each unit shall be factory-assembled, factory-wired, and factory-tested.

The fuel-burning equipment shall include the necessary accessories for the burning of either digester gas or natural gas or a mixture of the two gases with automatic switch-over. Pressure regulators, pressure check valves, and an ignition transformer shall be provided.

Recirculating Sludge Pipe. The recirculating sludge pipe from each digester shall be of standard weight steel, 23 cm (9 in.) in diameter. The heating length of the pipe shall be 6 m. The constant-speed sludge pump shall have a rated capacity of 2.5 m^3/min (660 gpm) to recirculate the sludge through the heat exchanger.

Heat Exchanger. Provide a common jacketed pipe heat exchanger to heat the recirculating sludge from each digester. The heat exchangers shall be designed for a working pressure of 207 kPa (30 psi). The sludge-heating capacity of the heat exchanger shall be 3.52×10^7 kJ/d. Indicating thermometers shall be provided at the inlet and outlet of the sludge tubes and at the hot water line. Automatic controls shall be provided to vary the flow of hot water through the water jacket in order to maintain a uniform heating temperature of the recirculating sludge.

[1] 1 kPa = kN/m^2 = 0.145 psi.

17-11 PROBLEMS AND DISCUSSION TOPICS

17-1 An aerobic digester was designed to stabilize combined sludge from a secondary treatment facility serving a population of 10,000. The combined sludge is not thickened and reaches the digester at a solids concentration of 1.5 percent, sp. gr. of 1.03, and TVSS/TS of 0.8. TS reaching the digester is 1100 kg/d. The dimensions of the digester that has $L:W = 3:1$ and a liquid depth of 4 m. Calculate volume allowance m^3/capita, VS loading kg/m^3·d, diffused air requirement, and TSS in the effluent. The digested sludge has 5 percent solids and provides a digestion period of 17 d. Assume that 40 percent of the cell tissue is oxidized completely, and 2.3 kg·O_2 is required to destroy each kg of TVSS.

17-2 An anaerobic sludge digester has an average liquid depth of 7 m and a diameter of 10 m. It receives 1600 kg/d thickened sludge at 6 percent solids. The specific gravity of the thickened sludge is 1.03. Calculate (a) volatile solids loading factor, (b) digestion period, and (c) capacity per capita if the population served is 19,000. Assume VS in sludge solids is 79 percent.

17-3 An anaerobic sludge digester receives 2000 kg/d solids at 76 percent VS. Calculate total gas production per day using Eqs. (17-2) and (17-3). Compare gas production using other rules of thumb. The population served is 35,000. 1 g of biodegradable solids = 1.42 g BOD_L,$Y = 0.06$, $k_d = 0.04$/d, and $E = 0.79$. Biodegradable solids = 70 percent. Incoming sludge is 5.5 percent solids and sp. gr. = 1.025. Average digestion period is 18 d.

17-4 Pilot plant experiments using complete-mix anaerobic digester without recycle were conducted on an industrial sludge. The data are given below:

Test Run	BOD_L of the Sludge Solids, dS/dt (kg/d)	Net Mass of Active Solids in the Digester X, kg	Sludge Solids Withdrawn from Digester dX/dt (kg/d)
1	450	200	39
2	220	55	21

If the digestion period (θ_c) is 10 d and waste-utilization efficiency E is 0.8, calculate the percent stabilization of BOD_L. Use Eqs. (17-2) and (17-3), and

$$\frac{dX}{dt} = Y\frac{dS}{dt} - k_d X$$

The digester receives 4500 kg/d BOD_L.

17-5 Compute the power requirements for gas mixing in a digester. The volume of the digester is 2000 m^3, velocity gradient for proper mixing is 88/s, and viscosity of sludge at operating temperature is 1.53×10^{-3} N s/m^2.

17-6 An anaerobic digester receives 2000 kg total dry solids per day. The sludge contains 5.8 percent solids at a specific gravity of 1.03. The digested sludge contains 4.2 percent solids and has a specific gravity of 1.02. Calculate the BOD_5 and TS of the digester supernatant. Use the following data:

TVS in raw sludge	= 75 percent
TVS stabilized	= 56 percent
Digested sludge withdrawal rate	= 24 m^3/d
BOD_L of digested sludge	= 0.65 × TVS in digested sludge
BOD_5/BOD_L	= 0.68

17-7 Thickened sludge is heated from 15–36°C before reaching an anaerobic digester. The average quantity of sludge is 100 m^3/d, the specific heat of sludge is 4200 J/kg°C (same for water), and the heat exchange capacity of jacketed heat exchanger is 3060 $kJ/h \cdot m^2 \cdot °C$. Calculate the following:

(a) Length of sludge pipe in the jacket

(b) Volume of hot water recirculated through the heat exchanger if inlet and outlet temperatures are 85°C and 40°C. Assume heat transfer efficiency of 80 percent.

(c) Digester gas needed in the boiler (m^3/d). Assume 75 percent boiler efficiency, and heating value of the digester gas is 24,500 kJ/m^3.

17-8 The TSS content in digested sludge and digester supernatant were developed at sustained loading conditions. These values are compared in Table 17-7 with those that were calculated in the mass balance analysis. Calculate BOD_5, Org.-N, NH_4^+-N, TN, TP, TVSS/TSS, and biodegradable solids/TSS under sustained loading conditions, and compare with those obtained in mass balance analysis.

17-9 What is the purpose of sludge mixing in an anaerobic digester? Describe various methods of sludge mixing and give advantages and disadvantages of each method.

17-10 Discuss different factors that affect the performance of an anaerobic digester. List various tests that must be performed daily for control of the digestion process. What is the significance of each test in relation to the normal digester operation?

17-11 Describe various types of digester covers and advantages and disadvantages of each type.

17-12 Describe various process parameters and advantages and disadvantages of an aerobic digester over anaerobic digester.

17-13 An anaerobic digester is designed to receive 4000 kg solids at 7 percent by weight. The digester roof, bottom, and side wall areas are 450, 470, and 550 m^3, having overall coefficients of heat transfers of 0.9, 0.62, and 0.68 $J/s \cdot m^2 \cdot °C$, respectively. Assume the incoming sludge, digester operating, and average outside temperatures are 25°C, 38°C, and 21°C, respectively. Minor heat losses are 25 percent of total heat loss. Calculate the total heat requirement of the digester. The specific gravity of the incoming sludge is 1.03.

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18

Sludge Conditioning and Dewatering

18-1 INTRODUCTION

Sludge dewatering is necessary to remove moisture so that the sludge cake can be transported by truck and can be applied as biosolids over farm lands, composted, or disposed of by landfilling or incineration. The solid particles in municipal sludge are extremely fine, are hydrated, and carry electrostatic charges. These properties of sludge solids make dewatering quite difficult. Sludge conditioning is necessary to destabilize the suspension so that proper sludge-dewatering devices can be effectively used.

Sludge-dewatering systems range from very simple devices to extremely complex mechanical processes. Simple devices involve natural evaporation and percolation from sludge lagoons or drying beds. Complex mechanical systems utilize sludge conditioning, followed by centrifugation, vacuum filtration, filter presses, and belt filters. The selection of any device depends on the quantity and type of sludge and the method of ultimate disposal. At smaller plants, sludge-drying beds or lagoons are frequently selected. Vacuum filters were used extensively in the past, but their use has declined because of the availability of other improved methods. Centrifugal thickening and dewatering has been applied with varying degrees of success in many applications. Filter presses, while widely used in Europe, were only introduced in the United States in the 1970s. Because of their considerably higher cake solids concentration and operational reliability, they were popular for some time. At the present time, however, the belt filter presses have become the predominant sludge-dewatering system in the United States. They offer continuous operation, have a low energy requirement, and have relatively lower capital and operation costs.

In this chapter various methods of sludge conditioning and dewatering are briefly presented. Step-by-step design procedures, operation and maintenance, and equipment specifications for a sludge-dewatering facility using chemical conditioning and belt filter presses are covered in the Design Example.

18-2 SLUDGE CONDITIONING

Conditioning involves chemical and/or physical treatment of the sludge to enhance water removal. In addition, some conditioning processes also disinfect sludge, control odors, alter the nature of solids, provide limited solids destruction, and improve solids recovery. Several chemical and physical conditioning methods are discussed below.[1-6]

18-2-1 Chemical Conditioning

Chemical conditioning is associated principally with mechanical sludge-dewatering systems such as vacuum filter, filter press, belt filter, and centrifugation. Chemical conditioning is achieved by inorganic or organic chemicals.

Inorganic Chemicals. Ferric chloride and lime are commonly used inorganic chemicals, although ferrous sulfate and alum have also been used. Ferric chloride that hydrolyzes in water is added first, forming positively charged soluble iron complexes. The iron complexes then neutralize the negatively charged sludge solids, thus causing them to aggregate. Ferric chloride also reacts with the bicarbonate alkalinity in the sludge to form hydroxides that cause flocculation. Hydrated lime is usually used in conjunction with ferric iron salts. Lime provides pH control, odor reduction, and disinfection. $CaCO_3$ produced in the reaction of lime and bicarbonate provides a granular structure that increases sludge porosity and reduces sludge compressibility.

The generalized chemical conditioning reactions with ferric chloride, ferric sulfate, alum, and lime are expressed by Eqs. (12-2)–(12-5). Properties of these chemicals are provided in Table 12-5. Addition of inorganic chemicals such as ferric chloride and lime in sludge increase the quantity of dry solids by 20–30 percent.

Both power plant fly ash and sludge incinerator ash have also been used successfully in the conditioning of sludge. Ash improves dewatering of sludge because of the partial solubilization of the metallic constituents in the ash, its sorptive capabilities, and its irregular particle size. When ash is used for sludge stabilization, the quantity of sludge cake is increased considerably.

Organic Chemicals. Organic polymers (polyelectrolytes) are widely used in wastewater treatment and sludge conditioning. These are long-chain, water-soluble, specialty chemicals. These materials differ greatly in chemical composition and functional effectiveness.[3,5,6] Because most sludge solids carry a negative charge, the cationic polymers are the most commonly used polymers for sludge conditioning. Polymers are further categorized by molecular weight (0.5–18 million), charge density (10–100%), and active solids level (2–95%). The use of organic polymers in sludge conditioning is attractive because they (a) do not appreciably increase solids quantity, (b) do not lower the fuel value of the cake, and (c) are safer and easier to handle than the inorganic chemicals.

Organic polyelectrolytes dissolve in water to form solutions of varying viscosity. The electrolytes in solution act by adhering to the sludge particle surfaces, thus causing (1) desorption of bound surface water, (2) charge neutralization, and (3) agglomerization by bridging between the particles.

Dosage. Proper dosage and mixing of chemicals are necessary to achieve sludge conditioning. The chemical dosage depends upon the type of sludge, solid concentration, pH, and alkalinity. The chemical dosage is determined by laboratory studies. Filter leaf tests or Büchner funnel, capillary suction time test (CST), and standard jar tests are commonly used to determine proper chemical dosage, filter yield, and suitability of various filtering media.[3,5-8] The chemical dosage requirements for different dewatering processes and types of sludge are discussed in Sec. 18-3. In general, the dosage of iron salts is 2–6 percent of the dry solids in the sludge. Lime dosage usually varies from 7–15 percent of dry solids. The dosage of polyelectrolytes may vary from 0–0.5 percent of dry solids in the sludge. The amount of fly ash for sludge stabilization is 25–50 percent of the sludge solids.

Mixing. Intimate mixing of sludge with coagulating chemicals is necessary to properly condition the sludge. In order to minimize floc shearing, mixing should provide just enough energy to disperse the conditioning chemicals throughout the sludge. Design criteria and design procedure for the mixing equipment are presented in Sec. 12-5-3 and Sec. 16-9-2, Step C.

18-2-2 Physical Conditioning

Important physical sludge-conditioning methods include elutriation and thermal conditioning. Other less commonly used methods are freezing, ultrasonic vibration, solvent extraction, and irradiation. These methods are briefly discussed below.

Elutriation. Elutriation is washing of sludge in order to remove certain soluble organic or inorganic compounds that would consume large amounts of chemicals for conditioning of sludge. Wastewater effluent is normally used for elutriation. The elutriation of sludge generates large volumes of liquid that contain high concentrations of suspended solids. This liquid, when returned to the plant, increases the organic and solids loadings.

Generally, the volume of washwater is two to six times the volume of sludge. Elutriation tanks are designed to act as gravity thickeners with a solids loading of 39–50 kg/m^2·d (8–10 lb/ft^2·d). The cost of additional tanks and equipment for washing and solids separation may not justify the savings in the cost of chemicals. Additional information on sludge elutriation can be obtained in Refs. 1, 3, 5, and 6.

Thermal Conditioning. The thermal conditioning process involves heating the sludge to a temperature of 140–240°C (248–464°F) in a reaction vessel under pressures of 1720–2760 kN/m^2 (250–400 psig) for periods of 15–40 min. One modification of the process involves the addition of a small amount of air. Heat coagulates the solids, breaks down the gel structure, and reduces the affinity for water. The resulting sludge is sterilized, practically deodorized, and dewatered readily on vacuum filters or filter presses without the need of chemicals. This process is most applicable to biological sludges that are normally difficult to condition or stabilize by chemicals. Because of high capital costs, this process is generally limited to large plants. Process side streams include gases and liquids. The gases are malodorous. Methods of odor control include combustion, ad-

TABLE 18-1 Characteristics of Returned Liquor from Thermal Conditioning of Sludge

Parameters	Concentration
BOD_5, mg/L	5000–15,000
COD, mg/L	10,000–30,000
TSS, mg/L	300–20,000
Total nitrogen, mg/L	600–1000
Total phosphorus, mg/L	150–200
pH	5.0–6.5

Source: Refs. 1 and 2.

sorption, scrubbing, and masking. Liquid side streams contain high BOD and, when returned to the plant, may significantly increase the loading to the aeration basin.[2] The characteristics of liquor from a thermal conditioning system are summarized in Table 18-1.

Other Conditioning Processes. Freezing and thawing improves sludge filterability. In northern regions natural freezing has been used for sludge conditioning.[1] Sludge irradiation also improves filterability. Discussion on irradiation of wastewater may be found in Chapter 14. Electric current, when passed through sludge using graphite anodes and iron cathodes, can also condition the sludge.[1] Ultrasonic vibration has also improved filterability of sludge. All these processes are in experimental stages.[1-3]

18-3 SLUDGE DEWATERING

A number of sludge-dewatering techniques are currently used. The selection of any sludge-dewatering system depends on (1) characteristics of the sludge to be dewatered, (2) available space, and (3) moisture content requirements of the sludge cake for ultimate disposal. When land is available and the sludge quantity is small, natural dewatering systems are most attractive. These include drying beds and drying lagoons. The mechanical dewatering systems are generally selected where land is not available. Common mechanical sludge-dewatering systems include vacuum filter, centrifuge, filter press, and belt filter press. A comparative evaluation of these processes is summarized in Table 18-2. Each method is discussed below.

18-3-1 Drying Beds

Sludge drying beds are the oldest method of sludge dewatering and are still used extensively in small- to medium-sized plants to dewater digested sludge. They are relatively inexpensive and provide dry sludge cake. In recent years, many advancements have been made to the conventional drying beds, and new systems are used on medium- and large-sized plants. These variations of the drying beds are (1) conventional sand, (2) paved, (3) wire-wedge, and (4) vacuum-assisted. Each of these variations of drying beds is discussed below.

Conventional Sand Beds. Typical sand beds consist of a layer of coarse sand 15–25 cm in depth and supported on a graded gravel bed (0.3–2.5 cm) that incorporates selected tiles or perforated pipe underdrains. The uniformity coefficient and effective size of the sand are less than 4.0 and 0.3–0.75 mm, respectively. Each section of the bed (6 m × 30 m) contains watertight walls and a perforated pipe underdrain system. Sludge cake removal is manual: shoveling into wheelbarrows or trucks or a scraper or front-end loader. Sludge is placed on the bed in 20- to 30-cm (8- to 12-in.) layers and allowed to dry. The underdrained liquid is returned to the plant. The drying period is 10–15 days, and the moisture content of the cake is 60–70 percent. Poorly digested sludge may cause odor problems. Depending on the climatic conditions and odor control requirements, the drying bed may be open or covered. Typical design criteria for sludge-drying beds receiving digested primary and secondary sludges are given below:

1. Sludge drying bed area:
   - 0.14–0.28 m^2 per capita (1.5–3.0 ft^2/capita) for uncovered beds
   - 0.10–0.20 m^2 per capita (1.0–2.0 ft^2/capita) for covered beds
2. Sludge loading rate:
   - 100–300 kg dry solids per m^2 per year (20–61 $lb/ft^2 \cdot yr$) for uncovered beds
   - 150–400 kg dry solids per m^2 per year (30–82 $lb/ft^2 \cdot yr$) for covered beds

The sludge cake from drying beds contains 20–40 percent solids; almost 90–100 percent solids capture occurs. The filtrate from anaerobically digested sludge may have a COD and BOD_5 of around 350 and 40 mg/L, respectively. The design details of the sludge-drying beds are given in Figure 18-1. Excellent discussions of sludge-drying beds may be found in Refs. 3 and 6.

Paved Drying Beds. Paved drying beds may be of a drainage type, decanting type, or a combination. The drainage-type paved drying beds are rectangular and are similar to the conventional drying beds with vehicular track for cake removal.[5] A front-end loader is generally used. The paving is made of concrete, asphalt, or soil cement. The lining rests on 20- to 30-cm built-up sand or gravel and should have a minimum of 1.5 percent slope to the drainage area. The beds are generally 6–15 m wide and 21–46 m long with vertical side walls. The unpaved section is 0.6–1 m wide. The drainage pipe is at least 10 cm in diameter and is placed below the unpaved area. The design details of paved drying beds are shown Figure 18-2.

The paved drying beds may also be a decanting type. The sludge feed pipe is vertical and is at the center of the bed. The drawoff pipes are near the edges and are used to decant the supernatant. The settling sludge is agitated periodically by a tractor-mounted horizontal auger or other device to regularly mix and aerate the sludge. The mixing and aeration breaks up the surface crust that inhibits evaporation and percolation. This may give more rapid dewatering than the conventional drying beds.[5] The solid con-

TABLE 18-2 Comparative Evaluation of Various Sludge-Dewatering Processes

Dewatering Method	Land Area Needed	Capital Costs	O&M Costs	Skilled Personnel	Energy Requirement	Chemical Needs	Sludge Variability	Filtrate TSS	Cake Solids	Others
Sludge-drying bed	Large	Low	Low, removal of cake labor-intensive	No	Low	Low	Less sensitive	Low	High	Odors, stabilized sludge applied, influenced by climatic conditions.
Sludge lagoons	Very large	Very low	Low	No	Low	None	No effect	No filtrate	High	Odors, stabilized sludge applied, potential for groundwater pollution, influenced by climatic conditions, mosquito problem, unsightly

Vacuum filter	Large floor area	High	Medium	Medium, need continuous attention	High	Medium	Not flexible	High	Medium	Vacuum pumps may be noisy
Centrifuge, solid-bowl	Low floor area	Medium	High	High demand	High	High	Medium effect	Medium	High	Minimum odors, easy to install, clean appearance
Centrifuge, basket	Low floor area	Medium	High	High demand	High	Medium	Flexible	High	Low	Minimum odor, same machine can be used for thickening and dewatering, clean appearance
Filter press (recessed plate filter)	Large floor area	High	High	High demand	Medium	High	Flexible	Low	High	Batch operation
Belt filter press	Medium floor area	Medium	Medium	Medium demand	Low	Medium	Not flexible, need grinding	Low	High	Hydraulically limited in throughput, short media life

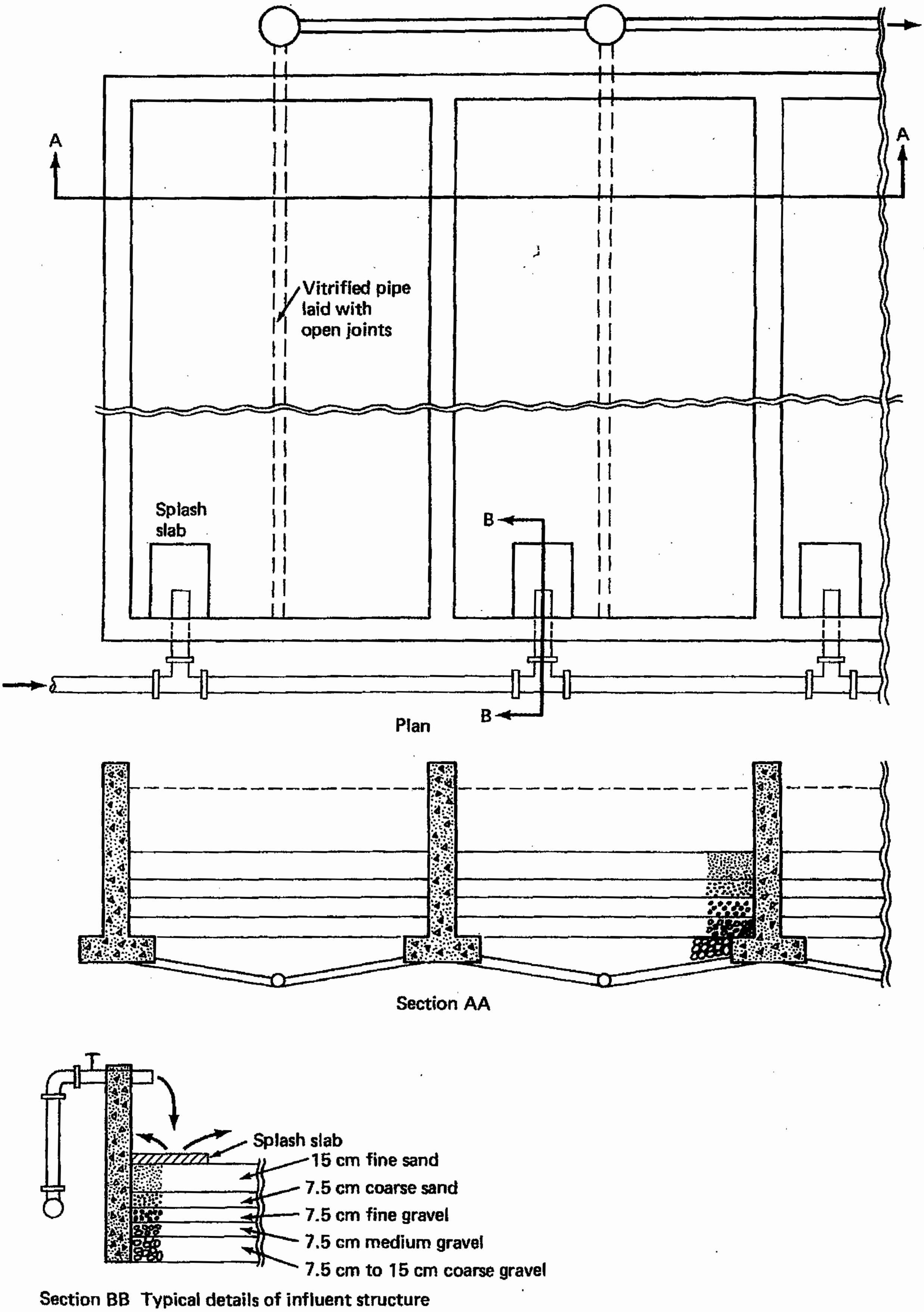

Figure 18-1 Details of Sludge-Drying Beds.

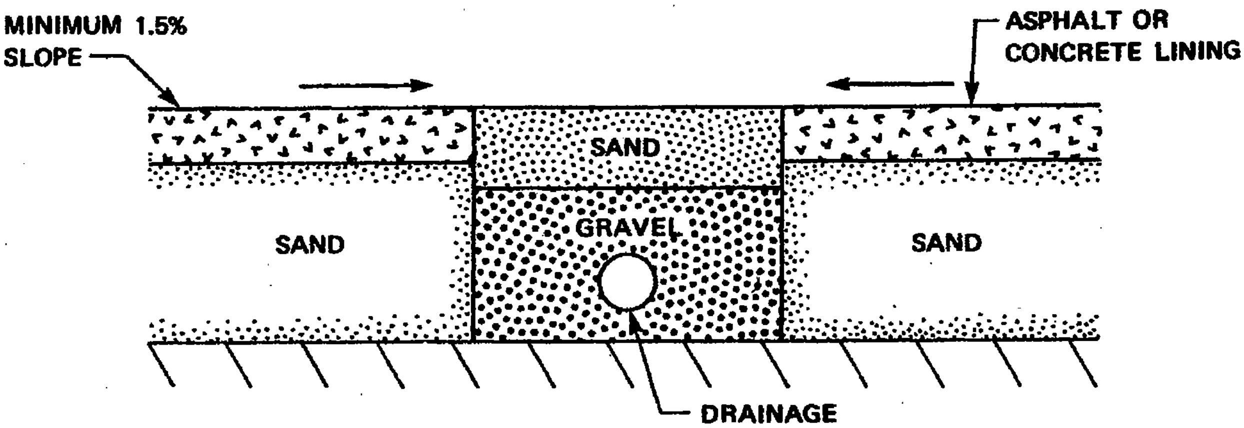

Figure 18-2 Typical Paved Drying Bed Construction (from Ref. 1).

centration may range from 40–50 percent for 30–40 days of drying time in an arid climate for a 30-cm sludge layer.[3]

Wedge-Wire. In recent years drying beds have been designed and built with artificial media. Stainless steel wire wedge and high-density polyurethane formed into panels have been successfully utilized. The bed consists of a shallow, rectangular watertight basin fitted with a false floor of wedgewater interlocking panels. These panels have slotted openings of 0.25 mm. An outlet valve controls the rate of drainage (Figure 18-3). Initially, the outlet valve is closed, and the bed is started by filling the space below and approximately 2–3 cm above the surface of wire-wedge or the panel. This water serves as a cushion when sludge is applied on the top of the false floor. After the bed is filled to a desired depth, initially applied wastewater and drainage water are released at a controlled rate through the drainage pipe. The sludge cake is allowed to concentrate by drainage and evaporation until cake is removed. The reported advantages for the system are (1) no clogging of the media, (2) constant and rapid drainage, (3) higher throughput rate than sand beds, (4) easier removal of sludge cake, (5) ability of difficult-to-dewater sludge to be dried, and (6) ease to maintain.[2] The solid loading rate is 890–1780 kg/m^2·yr (180–365 lb/ft^2·yr).[6]

Vacuum-Assisted. Sludge dewatering is accelerated by applying vacuum to the drying bed. The principal components are (1) a housing with a concrete bottom slab; (2) a layer of support aggregate; (3) a rigid multimedia filter top, which is placed on aggregate in the form of rectangular plates; (4) polymer addition and mixing equipment; and (5) a vacuum pump and filtrate removal system. The operation of a sludge-drying bed involves the application of preconditioned sludge to a depth of 30–75 cm (12–30 in.). The filtrate drains by gravity for approximately 1 hour. The vacuum system is started and maintained at 30–48 kPa (10–25 in. of Hg). When the cake is cracked and vacuum is lost, the suction is stopped. After 1–2 days of drying, the cake is removed by a front-end loader. The plates are washed by a high-pressure hose to remove the residue. A solids level of 14–23 percent is reached in the cake. The solid loading per application ranges from 5–20 kg/m^2. Each application or cycle time is 24 h (22 h for dewatering and 2 h

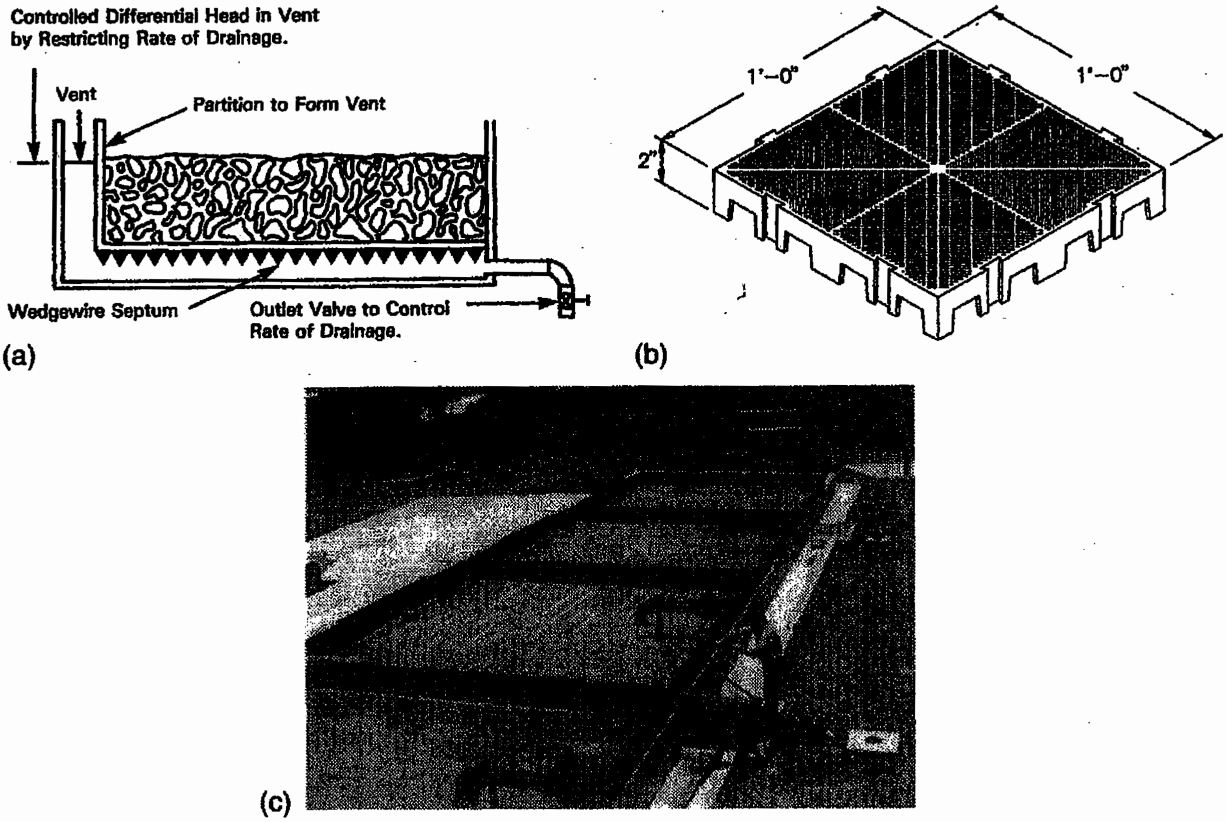

Figure 18-3 Design Details of Artificial Media Sludge-Drying Beds: (a) cross section of wire-wedge sludge-drying bed (from Refs. 4 and 5), (b) interlocking 30 cm × 30 cm high-density polyurethane GFS filter panel, and (c) installation of GFS wedge water filter bed ready for operation (courtesy Gravity Flow System, Inc.)

for cleaning). The polymer dosage depends on sludge characteristics, and the range is 1–10 g/kg dry solids (2–20 lb/ton). The system component of a vacuum-assisted drying bed is shown in Figure 18-4.

18-3-2 Drying Lagoons

Sludge lagoons are an economical method for sludge dewatering where sufficient land is available. They are similar to drying beds because the sludge is periodically removed and the lagoon refilled. Sludge must be stabilized to reduce odor problems.

Sludge-drying lagoons consist of shallow earthen basins. Earthen dykes (0.7–1.4 m high) enclose the sludge lagoon. Sludge 0.7–1.4 m (2–4 ft) in depth is applied. The supernatant is decanted from the surface and returned to the plant. The sludge liquid is allowed to evaporate. Sludge-drying time depends on the climatic conditions and the depth of sludge application. Generally, 3–6 months are required to reach 20–40 percent solids in the sludge cake. Solids capture in drying lagoons is 90–100 percent. Sludge cake is removed by mechanical equipment.

The suggested solids-loading rates for drying lagoons are 37 kg/m^3·yr (2.3 lb/ft^3·yr) of lagoon capacity. Some designers provide a lagoon capacity of 0.3–0.4 m^2/capita (3–4 ft^2/capita) for primary and secondary sludge. The proper design of sludge-drying lagoons

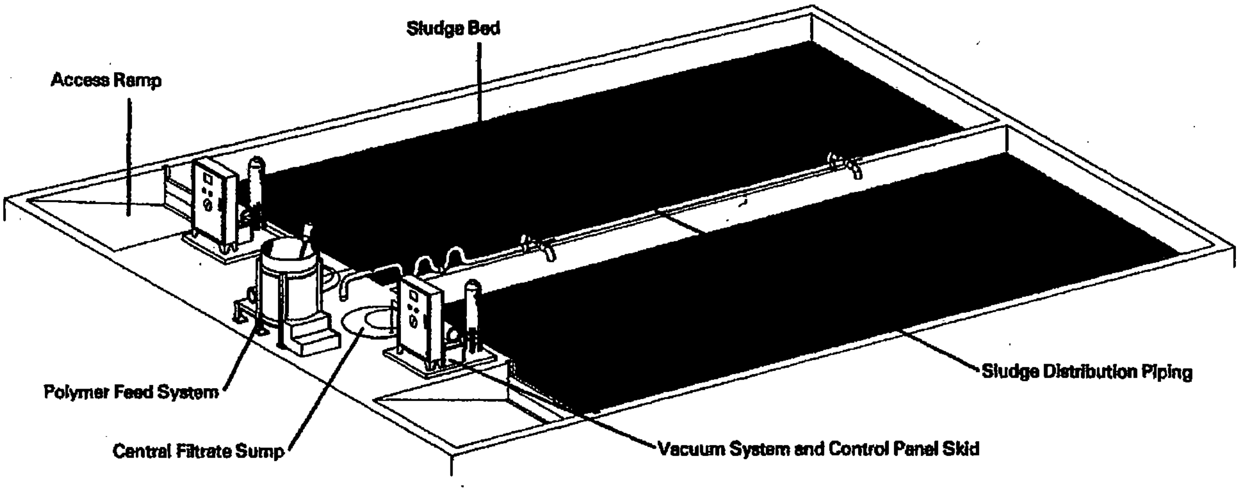

Figure 18-4 The Components of Vacuum-Assisted Drying Bed (courtesy Infilco Degremont Inc.)

requires a consideration of the following factors: climate, subsoil permeability, sludge characteristics, lagoon depth, and area management practices.[1,6,9,10]

18-3-3 Centrifugal Dewatering

The centrifuge uses centrifugal force to speed up the sedimentation rate of sludge solids. Sludge dewatering can be achieved by solid-bowl and basket centrifuges. In a typical unit, the conditioned sludge is pumped into a horizontal or cylindrical "bowl" rotating at 1600–2000 rpm. The solids are spun to the outside of the bowl where they are scraped out by a screw conveyor. The liquid, or "centrate," is returned to the wastewater treatment plant for treatment. The centrifuging process is comparable to the vacuum filtration in cost and performance. Centrifuges are compact and entirely enclosed (which may reduce odors), require small space, and can handle sludges that might otherwise plug filter cloth. The disadvantages include complexity of maintenance, abrasion problems, and centrate high in suspended solids.

The sludge cake from the centrifuge contains 20–35 percent solids, and solids capture of 85–90 percent is achieved. The polymer dosage for sludge conditioning prior to centrifuge is 0.1–0.7 percent of dry solids in the feed. A centrifuge dewatering system is shown in Figure 18-5. Excellent discussions of centrifuge dewatering may be found in Refs. 1–6.

18-3-4 Vacuum Filter

Rotary vacuum filters are widely used for dewatering of both raw and digested sludges. Vacuum filters consist of a cylindrical drum covered with cloth of natural or synthetic

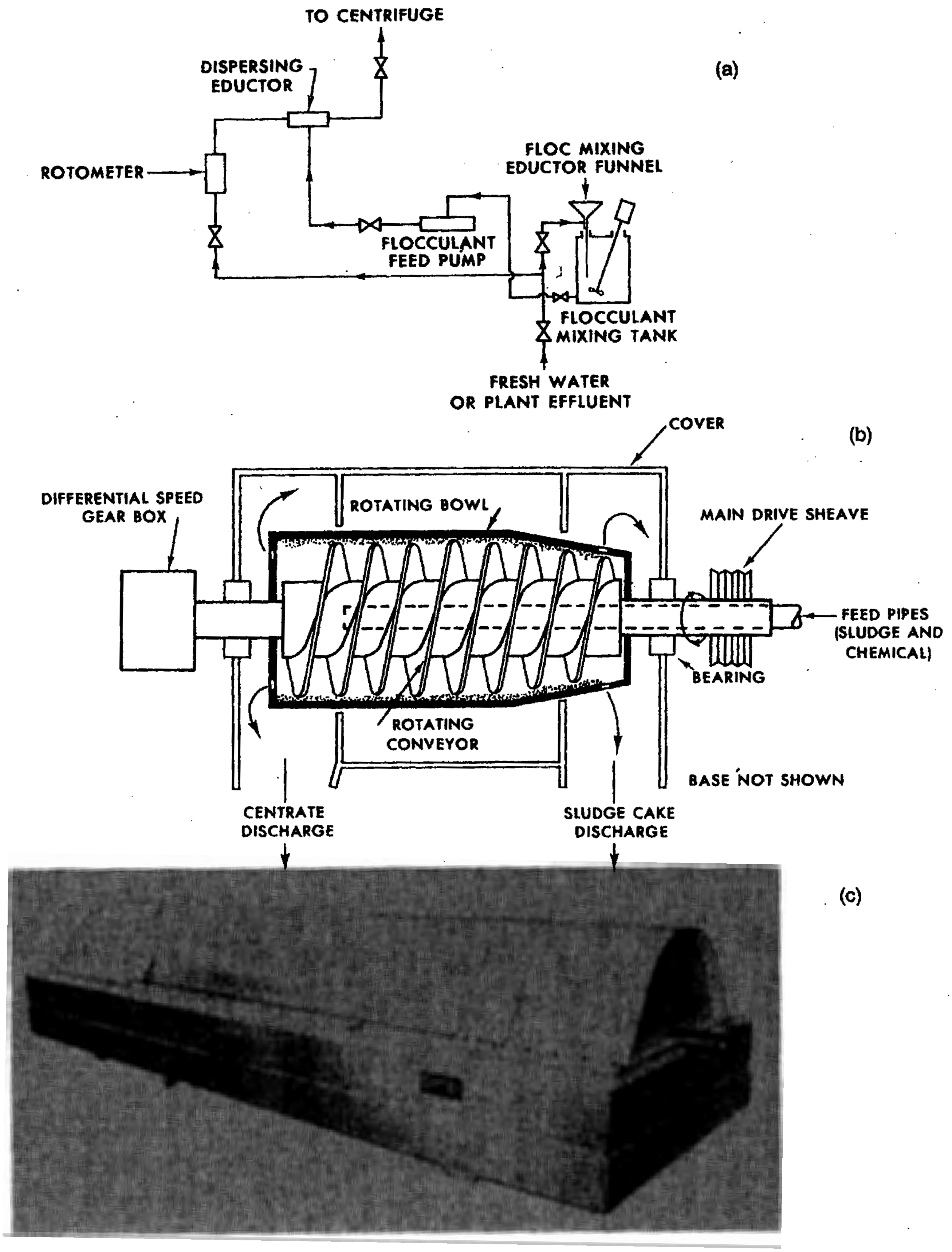

Figure 18-5 Centrifugal Sludge Dewatering System: (a) typical flocculant piping diagram of centrifuge used for sludge dewatering (from Ref. 2), (b) schematic of typical solid-bowl centrifuge (from Ref. 2), (c) continuous flow, solid-bowl centrifuge for sludge dewatering (courtesy Bird Machine Company).

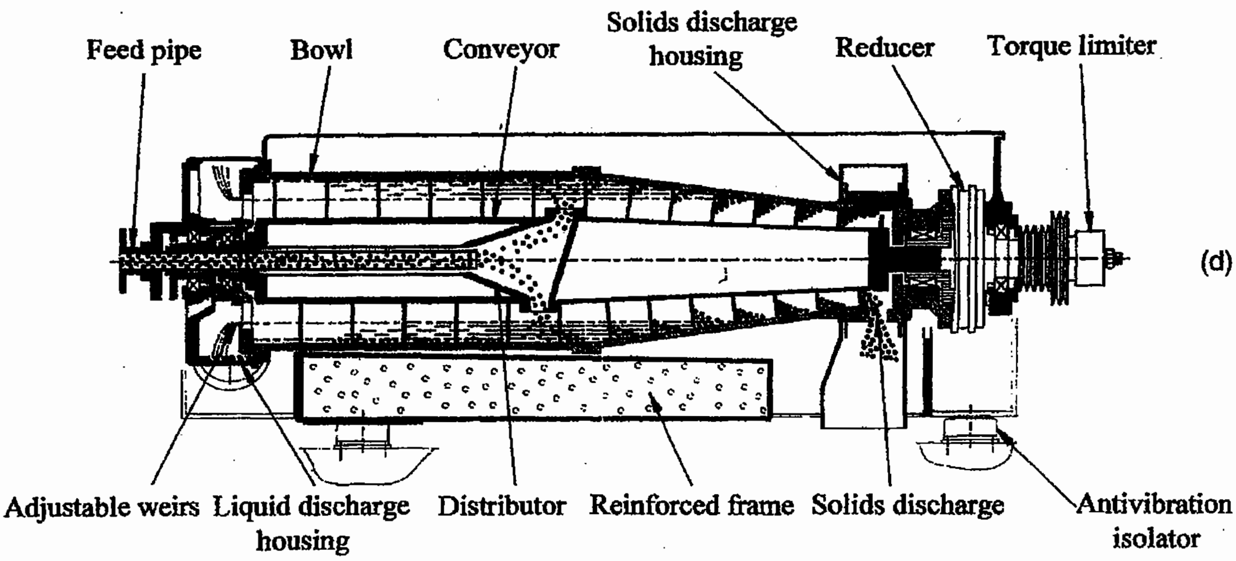

Figure 18-5—cont'd (d) cross section of centrifuge for thickening and dewatering (courtesy Andritz).

fabric. The drum remains partly submerged in a vat of sludge and rotates slowly. An internal vacuum that is maintained inside the drum draws the sludge to the filter medium, and water is withdrawn from the sludge. The cake-drying zone represents 40–60 percent of the drum surface and terminates at the cake discharge zone where the cake is removed. In a drum-type rotary vacuum filter, the sludge cake is scraped off. Compressed air may be blown through the media to release the cake prior to scraping. In belt-type rotary vacuum filters, the covering or media belt leaves the drum, and sludge cake is released by use of two stainless steel coils arranged around the drum. A typical vacuum filter arrangement is shown in Figure 18-6.

A variation of conventional rotary drum filter is the top feed drum filter. In this case the sludge is fed to the vacuum filter through a hopper located above the filter.

The important design factors for rotary vacuum filters include characteristics of conditioned sludge, cake formation time, viscosity, vacuum applied, specific resistance of the sludge cake, type of filter medium, and filter yield. Many equations have been developed to express the filtration rate, specific resistance of sludge, and filter yield. Basic theory of vacuum filtration may be found in some excellent publications.[3-7] Four test procedures used for determining the filterability of sludge are (1) the Büchner funnel method, (2) the filter leaf techniques, (3) capillary time test, and (4) standard jar test. The Büchner funnel method enables determination of the relative effects of various chemical conditioners and the calculation of the specific resistance of the sludge, but it is seldom used for the calculation of required filter area. The filter leaf test is used to determine the required filter area, evaluate filter medium, and permits an accurate prediction of the operation of a full-scale filter. Other tests are used for conditioning information. Detailed procedures on these tests may be found in Refs. 3–7.

Rotary vacuum filters range in size from 5–60 m^2 (50–600 ft^2) and are normally supplied by equipment manufacturers. The system includes vacuum pump, filtrate receiver and pump, filtering medium, and sludge-conditioning apparatus. The vacuum pump requirements are normally 0.5 m^3 of air per min per m^2 of filter area at 69 kN/m^2

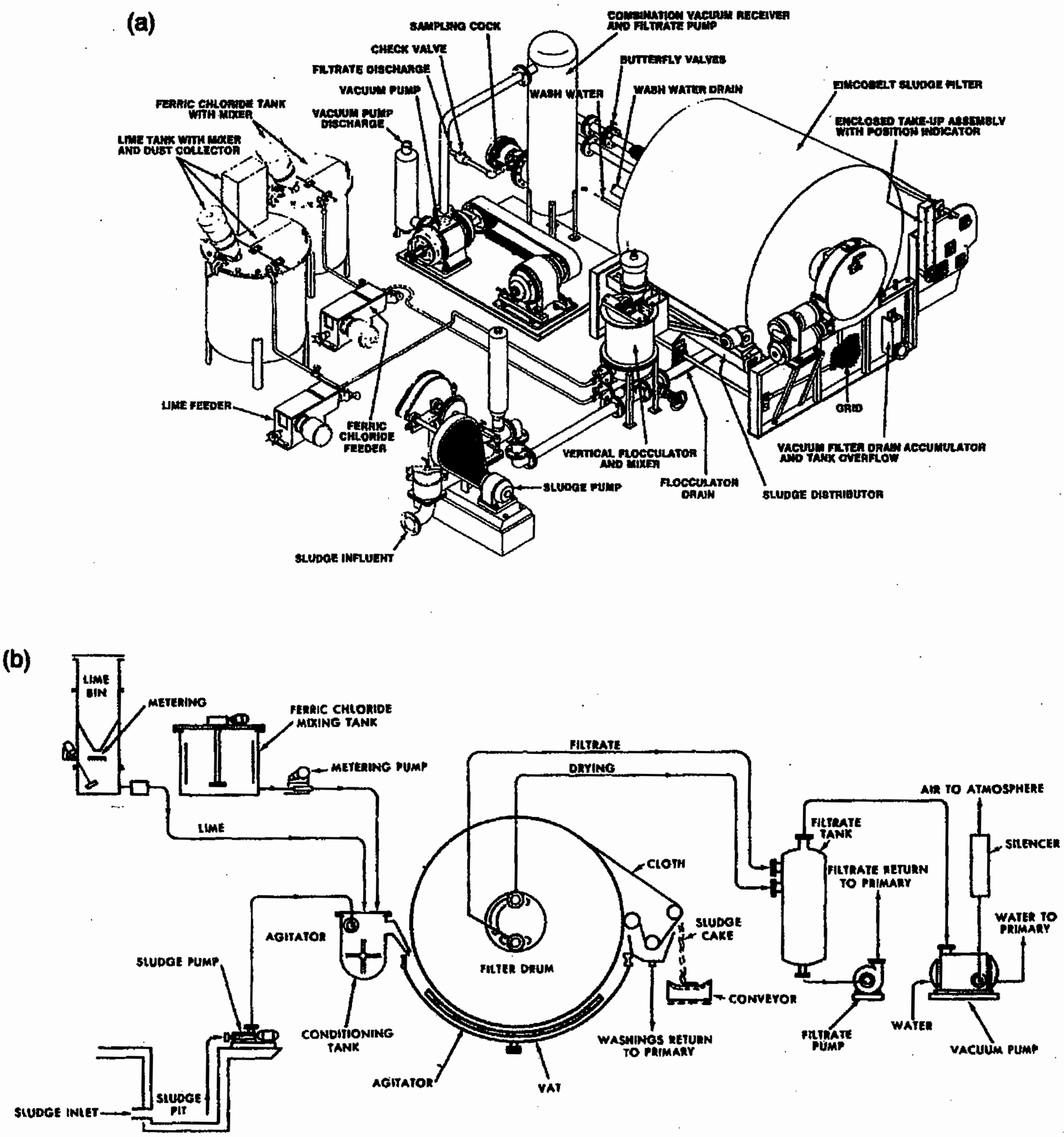

Figure 18-6 Installation Details of Rotary Vacuum Filter Assembly: (a) typical rotary vacuum filter installation (courtesy EIMCO Process Equipment Company) and (b) schematic flow diagram of rotary vacuum filter system (from Ref. 2).

(1.5–2.0 $ft^3/min{\cdot}ft^2$ at 20 in. of mercury vacuum).[1] The typical dewatering performance data for rotary vacuum filters using cloth media is summarized in Table 18-3. Other filter media include cotton, wood, nylon, Dacron, and so on. An evaluation of various filtering media was done at the Chicago Sanitary District.[2] The results indicated that Dacron, a polyester, was most suitable for their use. Other treatment plants have found polypropylene media to be satisfactory. Polyethylene media tend to stretch when wet and require constant operator vigilance of belt tension. Minneapolis-St. Paul has reported a life of

TABLE 18-3 Dewatering Performance Data for Rotary Vacuum Filters Using Cloth Media

Type of Sludge	Feed Solids (percent)	Chemical Dosage (percent)[a] FeCl$_3$	Chemical Dosage (percent)[a] CaO	Filter Yield (kg/m^2·h)[b]	Cake Solids (percent)
Raw sludge					
Primary	4–9	2–4	8–10	17–40	27–35
Primary and activated sludge	3–7	2–4	9–12	12–30	18–25
Primary and trickling filter	4–8	2–4	9–12	15–35	23–30
Anaerobically digested					
Primary	4–8	3–5	10–13	15–35	25–32
Primary and activated sludge	3–7	4–6	15–20	10–25	18–25
Primary and trickling filter	5–10	4–6	13–18	17–40	20–27
Aerobically digested					
Primary and activated sludge	3–6	3–7	8–12	8–20	16–23
Elutriated sludge					
Anaerobically digested					
Primary and activated sludge	4–8	3–6	0–7	15–18	18–25
Thermally conditioned					
Primary and activated sludge	6–15	0	0	20–40	35–45

[a]Chemical dosage percent of dry solids, 1 percent = 10 g/kg (20 lb/ton).
[b]1 kg/m^2·h × 0.205 = lb/ft^2·h.
Source: Adapted in part from Ref. 1.

12,400 h for a Saran medium. Monofilament fabrics are most resistant to blinding and have been used exclusively in recent installations of drum or belt filter.[2,11]

18-3-5 Plate and Frame Filter Press

Plate and frame presses are also called filter presses or recessed plate pressure filters. Typical installation of plate and frame presses is shown in Figure 18-7. These consist of round or rectangular recessed plates that, when pressed together, form hollow chambers. On the face of each individual plate is mounted a filter cloth. In a fixed-volume filter press, the sludge is pumped under high pressure 350–1575 kN/m^2 (50–225 psi) into the

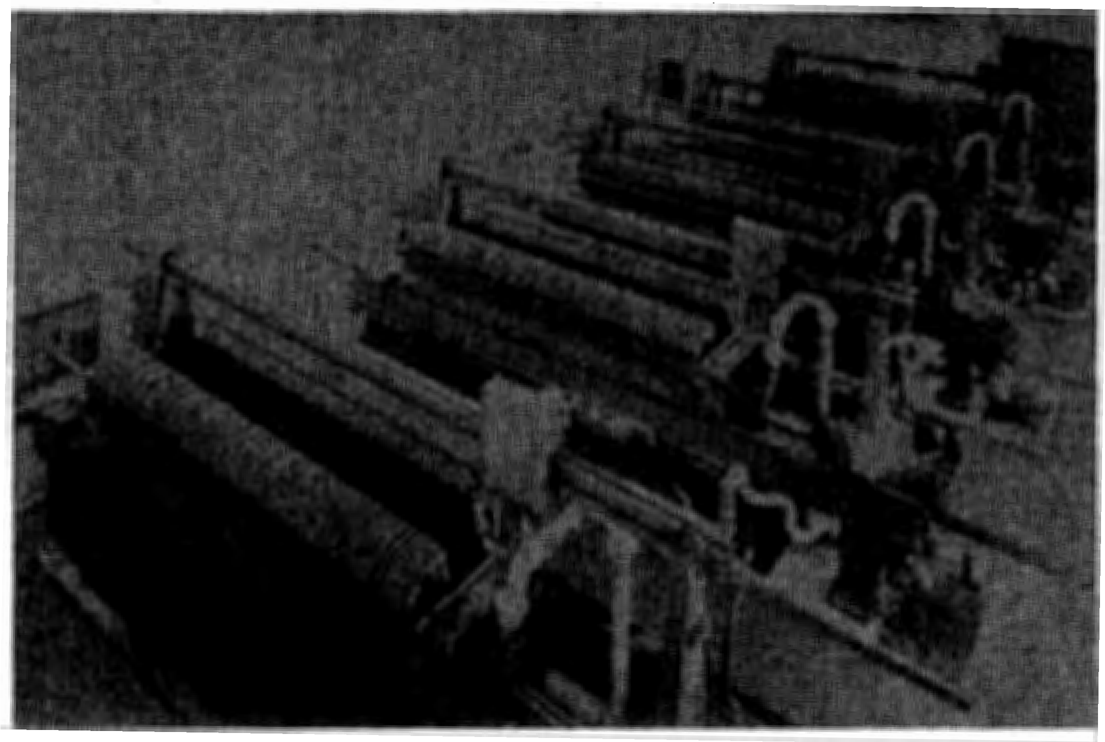

Figure 18-7 Typical Installation of Plate and Frame Filter Press Assembly (courtesy Envirex, U.S. Filter).

chamber. The water passes through the cloth while the solids are retained and form a cake on the surface of the cloth. The sludge filling continues until the press is effectively full of cake. The entire filling operation takes 20–30 min. The pressure at this point is generally the designed maximum and is maintained for a 1- to 4-h period. During this time, more filtrate is removed, and the desired cake solids level is reached (filter presses can attain up to 40 percent solids). The filter is then mechanically opened, and the dewatered cake drops from the chamber onto a conveyor belt for removal. Cake breakers are usually required to break up the rigid cake into conveyable form. Figure 18-8(a) shows the components of a fixed volume recessed plate pressure filter.

A variation of the fixed-volume filter press (discussed above) is the variable-volume recessed plate pressure filter. A diaphragm is placed behind the filter cloth that provides air or water pressure to squeeze the sludge. Generally, 10–20 min is required to fill the press with the conditioned sludge. When the endpoint is reached, the sludge feed pump is automatically turned off. Water or air under high pressure is then pumped into the space between the diaphragm and the plate, thus squeezing the already formed and partially dewatered cake to the desired solids content. At the end of the cycle, the water is returned to a reservoir, plates are automatically opened, and sludge cake is discharged. The operational details of a variable-volume recessed plate filter assembly are shown in Figure 18-8(b) and (c). The typical dewatering performance of filter presses is summarized in Table 18-4. Design example of a plate and frame filter press is covered in Ref. 10.

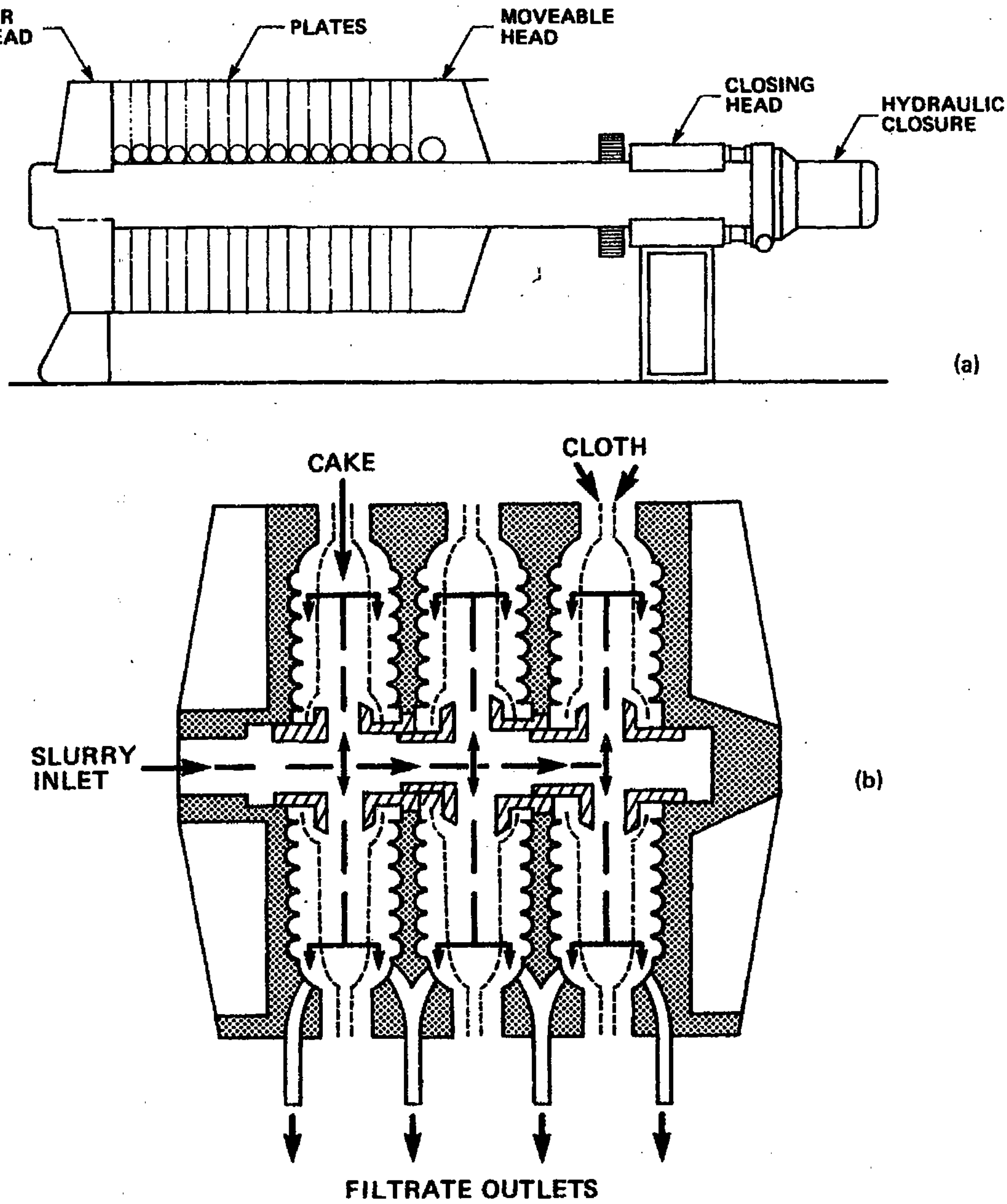

Figure 18-8 Details of Recessed Plate Pressure Filteration System: (a) details of a fixed-volume recessed pressure plate filter assembly (from Refs. 1), (b) cross section of a fixed-volume recessed plate filter assembly (from Ref. 1).

Continued

18-3-6 Belt Filter Press

The belt filter presses employ single or double moving belts to dewater sludge continuously. Belt filter presses are currently very popular in the United States. The main advantages of belt filter presses are a drier cake, low power requirement, and continuous operation. The main disadvantages are short media life and a filtration rate sensitive to incoming sludge.

The belt filtration process involves four basic operational stages: (1) polymer conditioning zone, (2) gravity drainage zone for excess water, (3) low-pressure zone, and

(c)

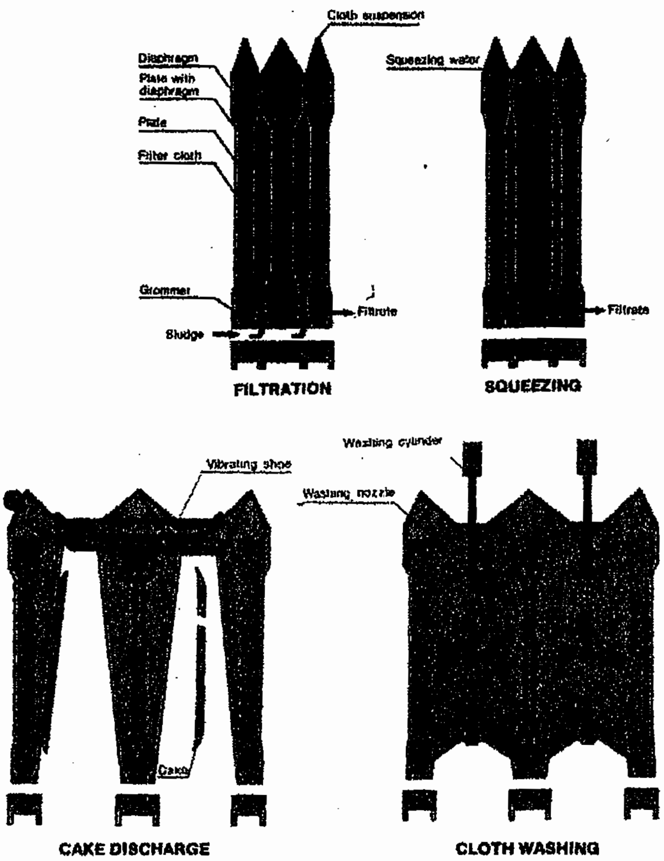

Figure 18-8—cont'd (c) operation of a variable-volume recessed plate filter assembly (courtesy Envirex, U.S. Filter).

(4) high-pressure zone. The system components of a belt filter process are identified in Figure 18-9 and briefly discussed below.[5]

The polymer conditioning zone can be a small tank, approximately 265–379 L (70–100 gal), located 0.6–1.8 m (2–6 ft) from the press; a rotating drum attached to the top of the press; or an in-line injector. Each press manufacturer usually supplies the polymer conditioning unit with the belt filter press.

TABLE 18-4 Typical Dewatering Performance of Plate and Frame Filter Presses

Type of Sludge	Feed Solids (percent)	Chemical Dosage (percent dry solids)		Filter Yield (kg/m²·h)	Cake Solids (percent)
		$FeCl_3$	CaO		
Primary and secondary	4	5	15	5	40
Anaerobically digested, primary and secondary	4	6	16	5	40
Thermally conditioned, primary and secondary	14	0	0	12	60

Source: Ref. 1.

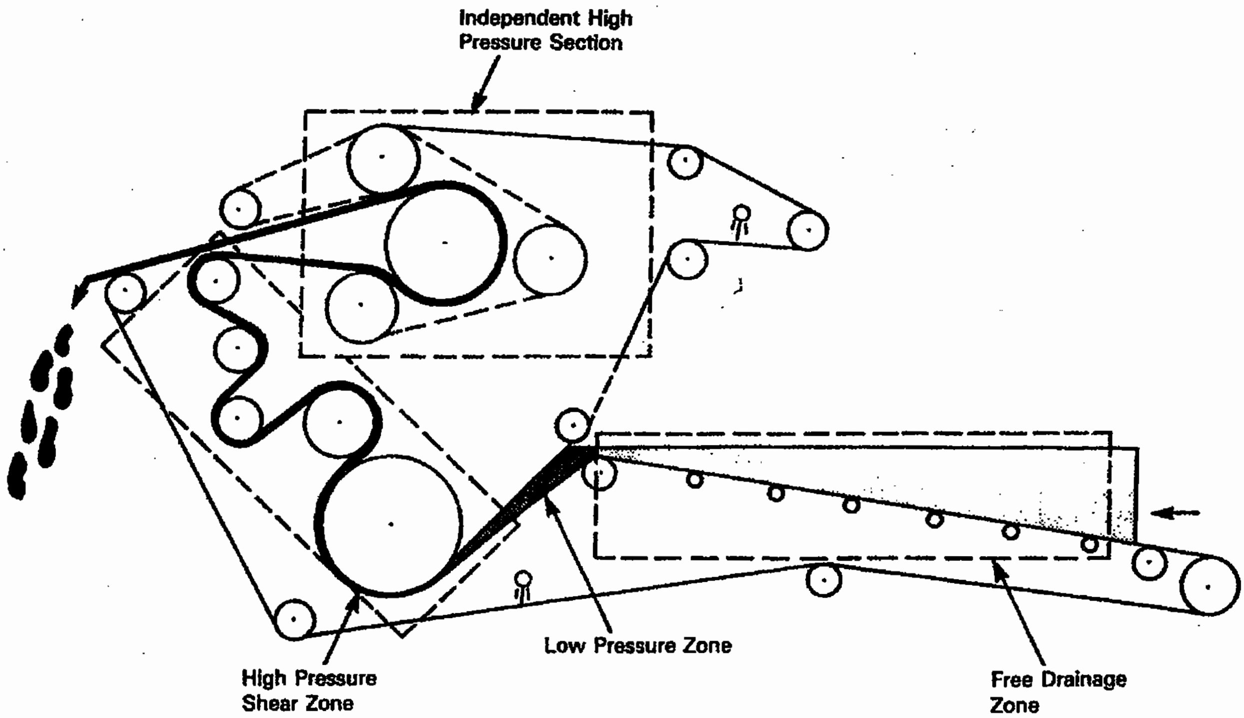

Figure 18-9 Belt Filter Press Used for Sludge Dewatering, Showing Various Basic Stages of Operation (from Ref. 5).

The gravity drainage zone is a flat or slightly inclined belt. In this section, sludge is dewatered by the gravity drainage of the free water. Approximately 5–10 percent increase in solids concentration is expected in the gravity drainage zone from the original feed sludge. Problems such as sludge squeezing out from between the belts and blinding of the belt mesh can occur if the sludge does not drain well in this zone. This free water drainage is a function of sludge type, quality, conditioning, screen mesh, and the design of the drainage zone.

The low-pressure zone, also called the wedge zone, is the area where the upper and lower belts come together with the sludge in between, thus forming the sludge "sandwich." The low-pressure zone prepares the sludge by forming a firm sludge cake that is able to withstand the shear forces within the high-pressure zone.

In the high-pressure zone, forces are exerted on the sludge by the movement of the upper and lower belts relative to each other, as they go over and under a series of rollers with decreasing diameters. Some manufacturers have an independent high-pressure zone that uses belts or hydraulic cylinders to further increase the pressure on the sludge (Figure 18-9), thus producing a drier cake. A dry cake is especially important for plants that use incineration as the final disposal method and need the driest cake possible. The compaction pressure can be widely varied by using a variable belt and roller arrangement, as shown in Figure 18-9.[10] A continuous belt press filter using an endless belt around a system of rollers for sludge dewatering is shown in Figure 18-10. The mechanical components of a belt filter press generally include the following: (1) dewatering belts available in widths of 0.5–3.5 m, (2) rollers and bearings, (3) belt tracking and tensioning system, (4) control and drives, and (5) belt washing system. Each piece of equipment that makes

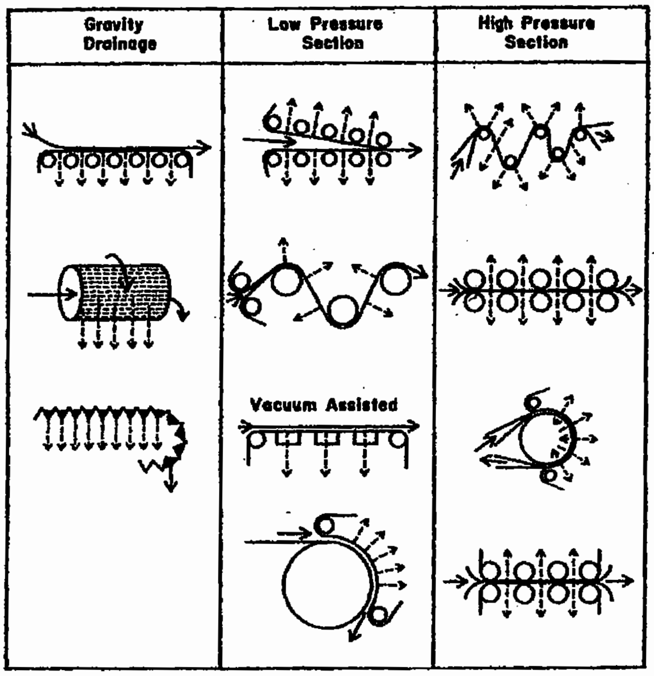

Figure 18-10 Alternative Designs for Obtaining Water Releases with Belt Filter Presses (from Ref. 11; used with permission of National Council of the Paper Industry for Air and Stream Improvement, Inc.).

up the sludge-dewatering system should be interconnected so that each unit is started in the proper sequence. In addition, automatic shutdown of equipment must also be provided in the proper sequence. If a piece of equipment downstream fails, everything upstream of that unit must shut down; i.e., if the press fails for any reason, the polymer and sludge feed pumps must shut down automatically. Automatic shutdown of dewatering equipment should occur for any of the following fault conditions: (1) belt drive failure, (2) sludge conditioning tank failure, (3) belt misalignment, (4) insufficient belt tension, (5) loss of pneumatic or hydraulic system pressure, (6) low belt wash water pressure, and (7) emergency stop. A continuous belt press filter using an endless belt around a system of rollers for sludge dewatering is shown in Figure 18-11. The design and operating data for a belt filter press are summarized in Tables 18-5 and 18-6. In-depth coverage on theory, design, and operation on belt filter presses may be found in Ref. 5.

18-3-7 New Mechanical Dewatering Systems

The concept of filtration has been utilized to develop many other types of dewatering devices. These are (1) modified twin belt press, (2) screw press, (3) Centri press, and (4) rotating gravity concentrator. Brief descriptions of each of these systems are given below.

Modified Twin Belt Press. The modified twin belt press is developed primarily for industrial applications. The system uses a series of wraparound rollers and a series of di-

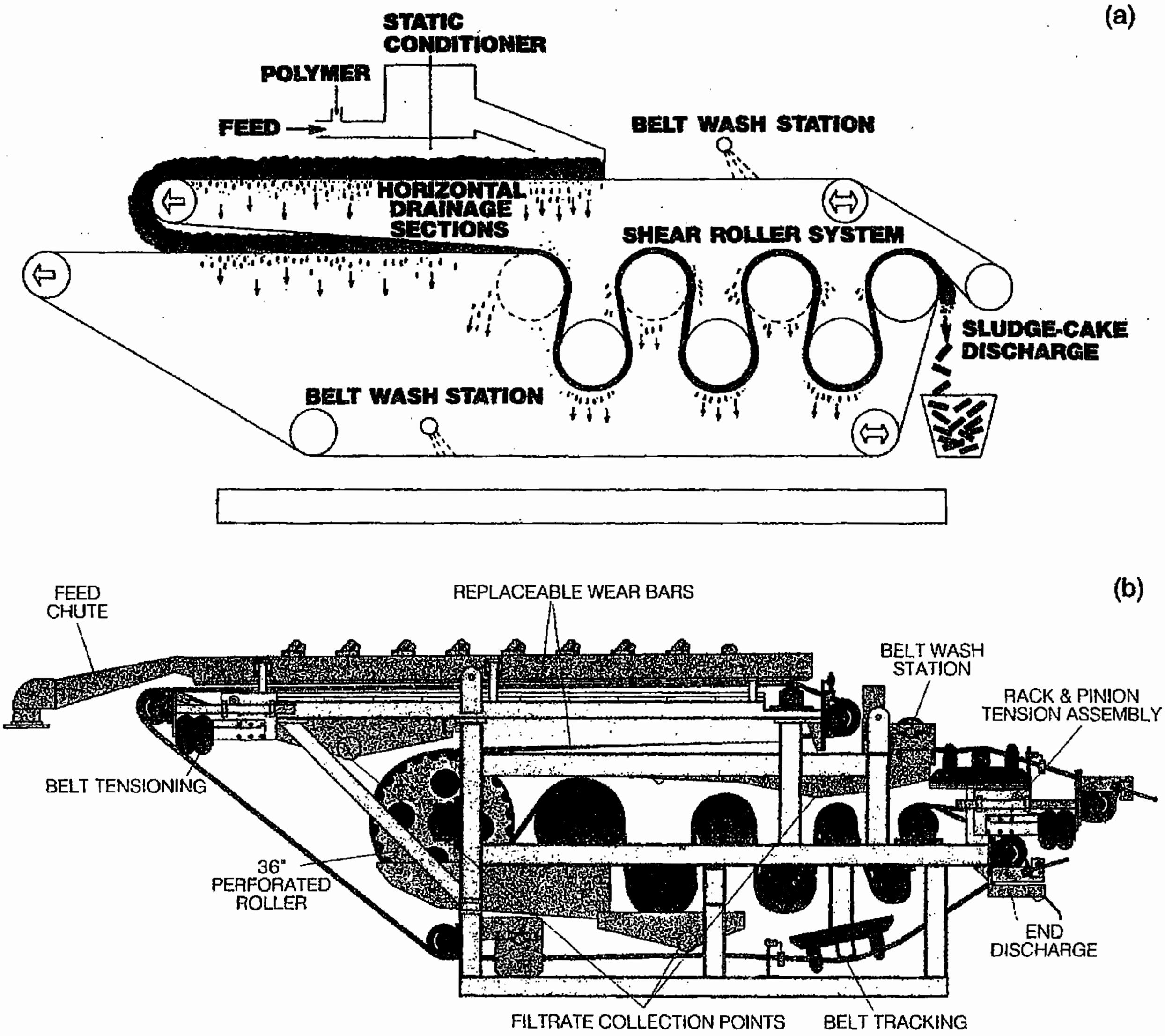

Figure 18-11 Belt Filter Press: (a) operational schematic (courtesy Ashbrook Corporation) and (b) construction detail (courtesy Andritz).

rect rollers. The pressure on the rollers can be varied from zero up to 200 kg/cm. Different types of press aids can be applied to yield dry sludge cake.[5]

Screw Press. The screw press employs a screen surrounded by a perforated steel cylinder. The sludge is pumped inside the screen. The screen moves progressively the dewatered sludge against a containment.[1,2] Polymer dosage is 0.2–0.5 percent of dry solid. The cake solids are 18–25 percent. Almost 90–95 percent solids capture is achieved. The screw could be vertical or horizontal. The functional schematics of a screw press is shown in Figure 18-12.[4,5]

Centri Press. The Centri press offers significant improvements in capabilities of a newly designed solid bowl, continuous flow centrifuge. The improvements are in the area of cake concentration.[5]

TABLE 18-5 Design and Operation Data of Belt Filter Press

Condition	Data
Solids in feed sludge	3–10 percent dry wt.
Solids in cake	20–40 percent dry wt.
Polymer for conditioning	0.2–0.5 percent of dry solids
Total suspended solids in filtrate	100–1000 mg/L
Solids capture	90–95 percent
Sludge-dewatering rate based on belt width	200–700 kg/m belt width·h
Hydraulic throughput based on belt width	2–8 L/m belt width·s

Source: Adapted in part from Refs. 3–6 and 11.

Rotating Gravity Concentrator. A rotating gravity concentrator (also called dual-cell gravity filter) consists of two independent cells. The cells are formed by a fine-mesh nylon filter cloth that travels continuously over the front and rear guide wheels. Dewatering occurs in the first cell, and cake formation takes place in the second cell.

TABLE 18-6 Typical Data for Various Types of Sludges Dewatered on Belt Filter Presses

Type of Sludge	Feed Solids (percent)	Solids Loading Rate (kg/m·belt width·h)	Polymer Dose (g/kg)	Cake Solids (percent)
Raw				
P	3–10	360–680	1–5	28–44
WAS	0.5–4	45–230	1–10	20–35
P + WAS	3–6	180–590	1–10	20–35
P + TF	3–6	180–590	2–8	20–40
Anaerobically Digested				
P	3–10	360–590	1–5	25–36
WAS	3–4	40–135	2–10	12–22
P + WAS	3–9	180–680	2–8	18–44
Aerobically Digested				
P + WAS	1–3	90–230	2–8	12–20
P + TF	4–8	135–230	2–8	12–30
Oxygen Activated				
WAS	1–3	90–180	4–10	15–23
Thermally Conditioned				
P + WAS	4–8	290–910	0	25–50

P = primary sludge; WAS = waste activated sludge; TF = trickling filter sludge.
Source: Adapted from Ref. 5.

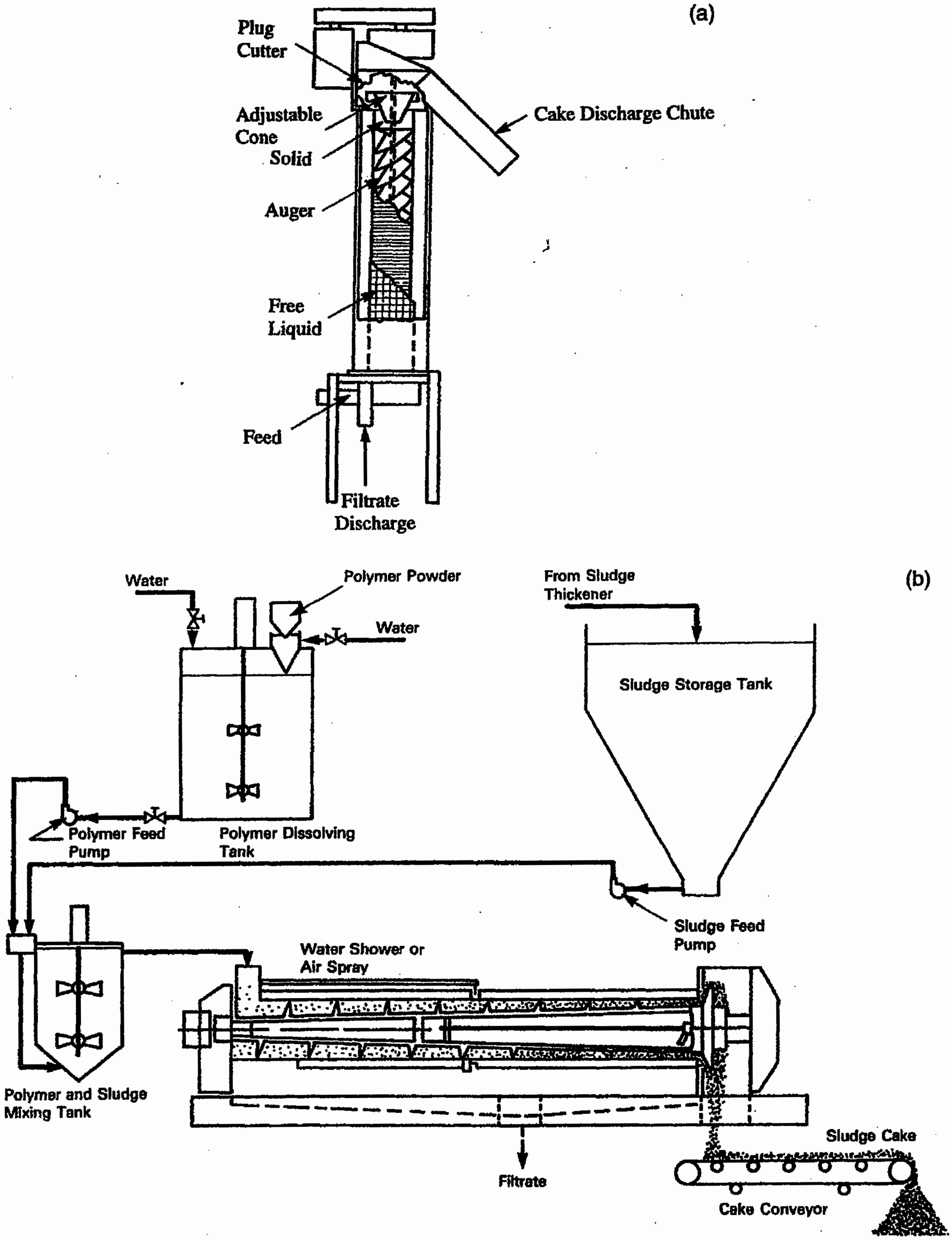

Figure 18-12 Schematic Diagram Showing Screw Press Dewatering System: (a) vertical configuration (from Refs. 4 and 5), (b) flow schematic and components of horizontal screw configuration (from Refs. 4 and 5).

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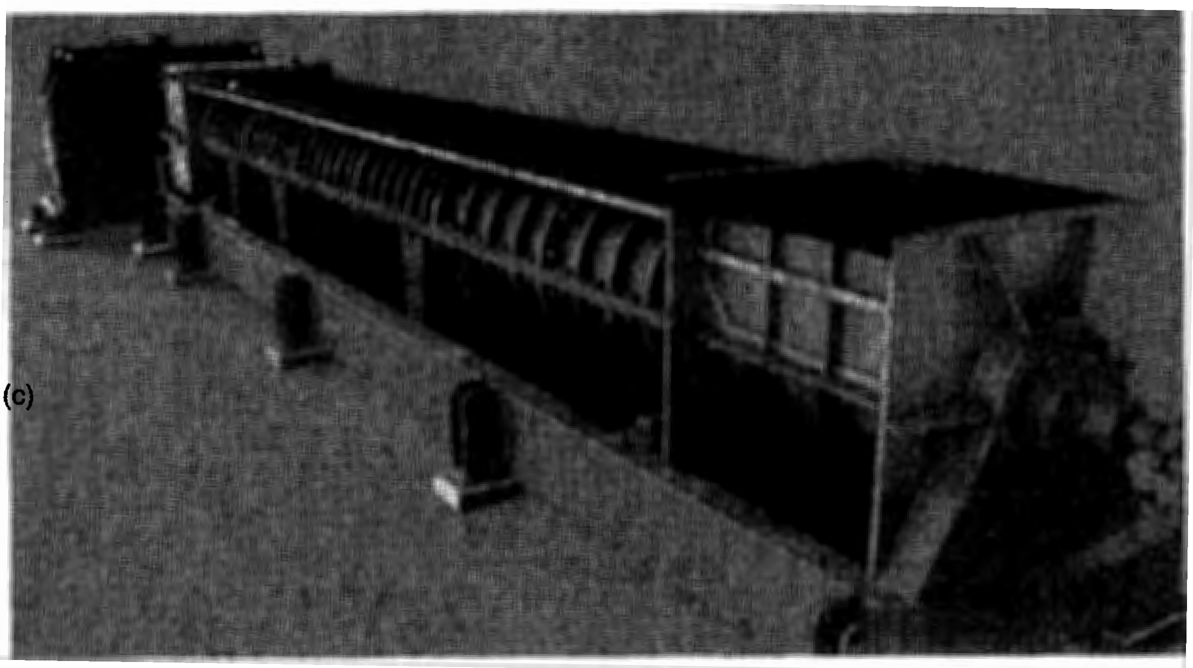

(c)

Figure 18-12—cont'd (c) photograph of a horizontal screw press (courtesy Andritz).

The partially dewatered solids are then carried over the drive roll separator, where a cake of relatively low moisture content is produced. The dewatering is entirely by gravity and without the application of either pressure or vacuum. If more complete dewatering is needed, a multiroll press is used. The sludge cake contains 10–16 percent solids without the multiroll press. The system components are shown in Figure 18-13.[2,5,11]

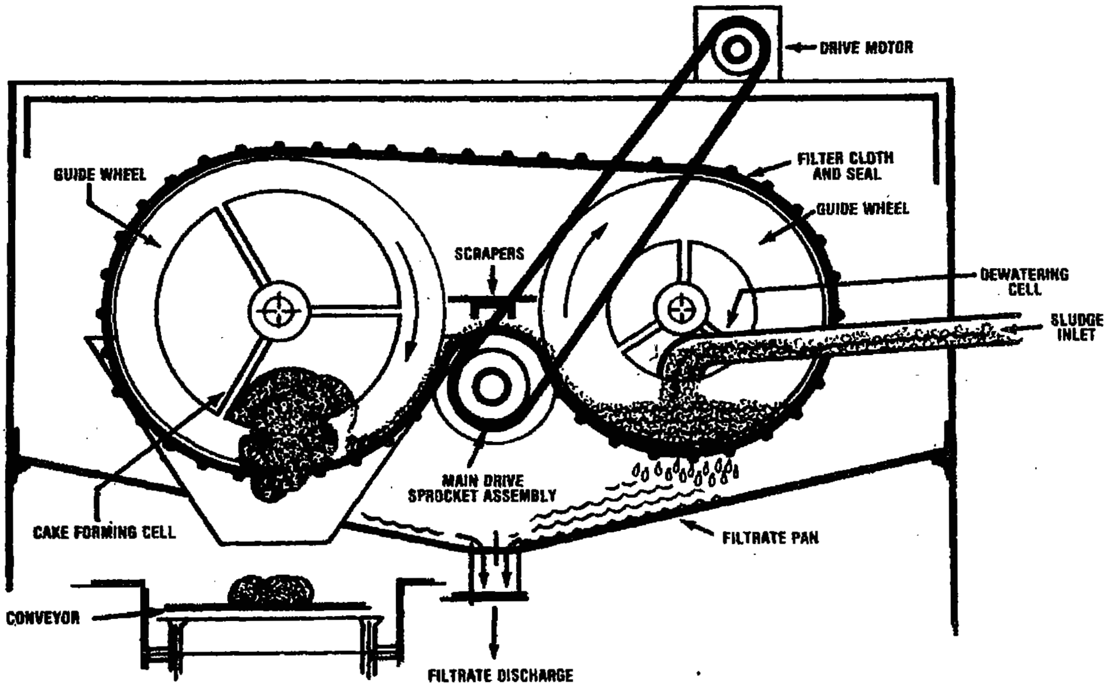
18-4 EQUIPMENT MANUFACTURERS OF SLUDGE-CONDITIONING AND -DEWATERING SYSTEMS

A list of sludge and dewatering equipment suppliers is given in Appendix D. This list includes the suppliers of chemical feeders, mixers, and controls and various types of sludge-dewatering equipment. Important considerations for equipment selection are presented in Sec. 2-10.

18-5 INFORMATION CHECKLIST FOR DESIGN OF SLUDGE-CONDITIONING AND -DEWATERING FACILITIES

The design of a sludge-conditioning and -dewatering facility should be started after the following basic information has been developed and necessary decisions have been made:

1. Develop sludge characteristics. This includes average quantity of sludge dewatered per day, solids concentration, and whether the sludge is raw or stabilized. Material



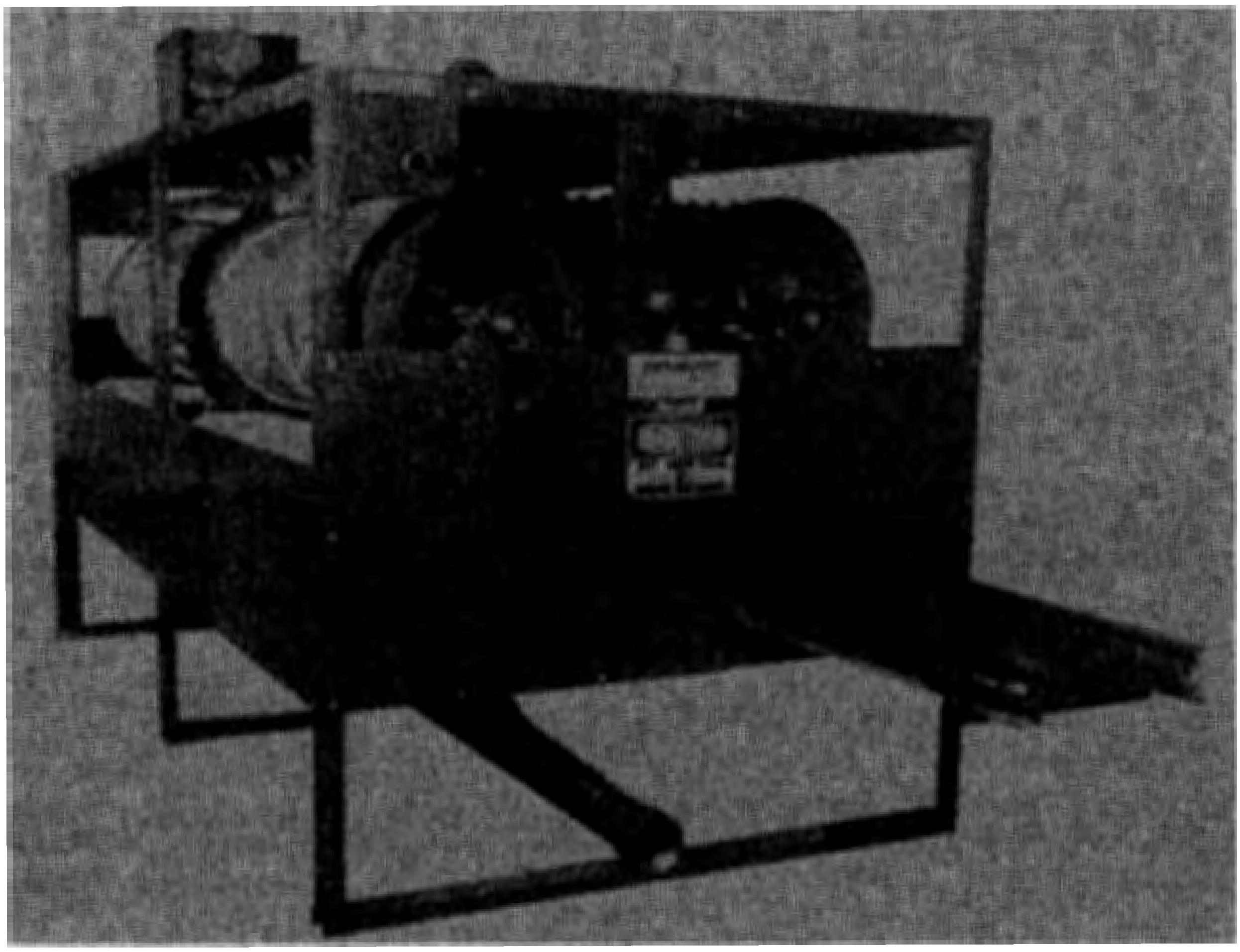


Figure 18-13 Details of Dual-Cell Gravity Filter Assembly (courtesy The Permutit Company, Inc.).

mass balance analysis at average daily design flow must be conducted to establish this information.
2. Select an ultimate sludge disposal method, and specify the moisture content of the sludge cake.
3. Evaluate site conditions such as space, access roads, noise, and other environmental limitations.
4. Establish operational characteristics of the equipment, including energy requirements, specialized maintenance requirements, performance reliability, and simplicity of operation.
5. Establish nature of operation, that is, whether continuous or intermittent.
6. Determine type of chemicals and the dosages needed for proper dewatering. Laboratory bench-scale or pilot testing may be necessary to establish chemical dosages, mixing, and reaction period.
7. Develop design data for the selected equipment for dewatering. This includes operational period, solids-loading rate, or dewatering rate.
8. Obtain the design criteria from the concerned regulatory agency.
9. Obtain manufacturers' catalogs and equipment selection guides.

18-6 DESIGN EXAMPLE

18-6-1 Design Criteria Used

The following design assumptions and criteria are used for the design of a sludge-dewatering facility:

1. The combined primary and secondary sludge is anaerobically digested. The characteristics of anaerobically digested sludge average daily design flow conditions are given below:

Total solids	= 5021 kg/d (Sec. 17-8-2, Step E, 3)
Flow	= 98 m^3/d (Sec. 17-8-2, Step E, 3)
Concentration of solids	= 5.0 percent (Sec. 17-8-1)
pH	= 6.5–7.5
Specific gravity	= 1.03 (Sec. 17-8-1)

   Other constituents in feed sludge are given in Table 17-7.
2. The minimum dry solids allowed in the sludge cake shall be 25 percent.
3. Select belt filter press assembly for dewatering.
4. Based on the pilot testing with a belt filter assembly, the following design data were developed:

   - chemical conditioning

     Optimum cationic polymer dosage = 0.5 percent (5 g/kg dry solids)

   No BOD_5 is added by chemical conditioning, and 80 percent polymer is captured in the sludge cake.

- dewatering equipment

Combined digested sludge (primary + WAS)	
Operating time (continuous operation)	= 8 h/d, 5 d/week
Solid loading	= 250 kg/m belt width·h
Maximum hydraulic throughput loading less than 5 L/m belt width·s	
Cake solids (minimum)	= 25 percent
TSS capture	≠ 95 percent
Sp. gr. of sludge cake	= 1.06

- mechanical requirement

  Filter belt wash water volume (plant effluent) = 35 L/kg TSS in feed sludge

  The filter belt wash water is applied over the belt near the rollers; therefore, all washwater goes directly to the filtrate (without going through the belt).

5. The filtration system shall be housed inside a building. The filter building design shall include filtrate removal, sludge cake removal, chemical feed tanks, chemical feed equipment, chemical storage, and belt filter press assembly. The digested sludge-holding well shall be kept outside the filter building.
6. The number of belt filter press units shall be selected such that the normal filtration process continues when the largest single unit is out of service. When all units are in operation, the maximum filtration capacity shall be 125 percent of the average daily design capacity.

18-6-2 Design Calculations

Step A: Belt Filter Press Sizing

1. Compute the total solids processed per hour of filter operation.

Total solids dewatered = sludge + polymer

Sludge solids $= \dfrac{5021 \text{ kg/d} \times 7 \text{ d/week}}{5 \text{ d/week (operation)}}$ $\quad \dfrac{7029 \text{ kg/d}}{8 \text{ h/d}} = 879 \text{ kg/h}$

$= 7029$ kg/d (879 kg/h)

Polymer in processed solids (assume 80% capture) = 7029 kg/d × 0.005 × 0.8
= 28 kg/d (3.5 kg/h)

Total solids = 7057 kg/d

$$\text{Total solids processed per hour} = \frac{7059 \text{ kg/d}}{8 \text{ h/d}} = 882 \text{ kg/h}$$

2. Compute the belt-filter press size.

$$\text{Efective belt width} = \frac{882 \text{ kg/h}}{250 \text{ kg/m·h}} = 3.5 \text{ m}$$

Provide three belt filter presses, each having a 1.2-m-wide belt (total width = 3.6 m). Provide one identical unit as a standby.

3. Check loading under different operating conditions.

Solids loading $= \dfrac{882 \text{ kg/h}}{3 \times 1.2\text{-m-wide belt}} = 245 \text{ kg/m·h}$

Peak solids handled when four belt filter presses are in operation $= 245 \text{ kg/m·h} \times 1.2\text{-m-wide belt} \times 4 \text{ filters}$

$= 1176 \text{ kg/h}$

Peaking factor under sustained solids loading $= 1176 \text{ kg/h} \div 882 \text{ kg/h} = 1.33$

The system is capable of handling 33 percent additional solids under sustained loading conditions. If a higher quantity of solids is handled under sustained loading, the operation period will be increased from a normal 8-h per day. Because of the 5-day operation of a dewatering facility, a 2- to 3-day sludge storage facility will be needed.

The hydraulic throughput loading on belt width $= \dfrac{98 \text{ m}^3\text{/d} \times 7 \text{ d/week}}{5 \text{ d/week}} \times \dfrac{1}{8 \text{ h/d}} \times \dfrac{1000 \text{ L/m}^3}{(60 \times 60)\text{s/h}} \times \dfrac{1}{3.6} \text{m belt width}$

$= 1.3$ L/m belt width·s [This is within the permissible value (see Design Criteria).]

Step B: Sludge Cake and Filtrate Quality

1. Compute the volume of filter wash water and TSS in wash water.

Volume of belt filter wash water $= 35 \text{ L/kg} \times 5021 \text{ kg/d} \times \dfrac{1}{1000 \text{ L/m}^3} = 176 \text{ m}^3\text{/d}$

TSS in belt filter wash water $= 10 \text{ g/m}^3 \times 176 \text{ m}^3\text{/d} \times \dfrac{1}{1000 \text{ g/kg}} = 1.8 \text{ or } 2 \text{ kg/d}$

2. Calculate TSS in the sludge cake and volume of the sludge cake.

TSS removed $= 5021 \text{ kg/d} \times 0.95 = 4770 \text{ kg/d}$ (596 kg/h)
Organic polymer removed $= 5021 \text{ kg/d} \times 0.005 \times 0.8 = 20 \text{ kg/d}$
Total TSS in filter cake $= 4770 \text{ kg/d} + 20 \text{ kg/d} = 4790 \text{ kg/d}$

Volume of sludge cake $= \dfrac{4790 \text{ kg/d}}{0.25 \times 1060 \text{ kg/m}^3} = 18 \text{ m}^3\text{/d}$

3. Calculate the filtrate TSS and flow rate.

TSS in filtrate = TSS in digested sludge + TSS in belt wash water

+ polymer remaining in filtrate
− TSS lost in sludge cake
= 5021 kg/d + 2 kg/d + 5021 kg/d × 0.005
× (1 − 0.8) − 4770 kg/d
= 258 kg/d

Volume of filtrate = volume of digested sludge + volume of belt wash water
+ volume of polymer − volume of sludge cake
= 98 m^3/d + 176 m^3/d + small volume − 18 m^3/d
= 256 m^3/d

4. Calculate the overall solids capture efficiency of the belt filter.

Total solids reaching filter = 5021 kg/d + 20 kg/d = 5041 kg/d
Total solids lost in filtrate = 258 kg/d − 2 kg/d[a] = 256 kg/d

$$\text{Overall efficiency of belt filter} = \frac{(5041 - 256)\text{ kg/d}}{5041\text{ kg/d}} = 95\text{ percent}$$

5. Calculate other constituents in the sludge cake and filtrate.
The values of BOD_5, various forms of nitrogen and phosphorus, and other constituents in sludge cake and filtrate have been calculated in the mass balance analysis. The step-by-step calculations for the first iteration are provided in Sec. 13-11-4, Step A, 11 (sludge cake) and Step A, 12 (filtrate). The final results are summarized in Table 13-13 under streams 14 and 15. These values for sludge cake and filtrate are provided in Table 18-7. There may be a slight difference (less than 0.5 percent) in these values if calculated from the increased sludge quantity reaching the dewatering facility. The revised values may be obtained by repeating the steps given in the material mass balance analysis.

Step C: Belt Filter Press Building and Equipment Layout The belt filter press building is designed to house the chemicals, feeders, mixing, and conditioning tanks, pumps, and filter assembly. The digested sludge and sludge feed pumps are located outside the building.

1. Compute the digested sludge storage tank (outside the building).
Provide storage for three days.

Flow rate of digested sludge = 98 m^3/d (Table 17-7)
Volume of sludge storage tank = 98 m^3/d × 3 d = 294 m^3

Provide one square tank with sludge hopper. The design details are shown in Figure 18-14. The tank is 7 m × 7 m. The vertical and conical sections are 4.2 m and 5 m deep, respectively. The base of the conical section is 1 m × 1 m.

[a]The filter belt wash water is applied over the belt near the rollers. Therefore, the washwater does not go through the filter.

TABLE 18-7 Characteristics of Sludge Cake and Filtrate Obtained from a Belt Filter Press (Table 13-13).

Parameter	Sludge Cake (kg/d) (stream 14)	Filtrate (stream 15)	
		kg/d	mg/L
Flow, m^3/d	18 (18[a])	255 (256[a])	—
TSS	4778 (4790[a])	257 (258[a])	1004 (1007[a])
BOD_5	1516	82	320
Org.-N	304	16	63
NH_4^+-N	8.1	36	141
NO_3-N	0	0	0
TN	312	57	204
NPP	64	3.6	14
PP	126	—	—
TP	190	3.6	14
TVSS/TSS, ratio	0.55	—	—
Biodegradable solid/TSS, ratio	0.33	—	—
Org.-N/TSS ratio	0.12	—	—
NPP/TSS	0.025	—	—

[a]Values in parentheses are calculated in Steps B, 2 and B, 3 and are provided for comparison. These values are higher because of a sustained loading effect.

$$\begin{aligned}\text{Total volume of the tank} &= (7\text{ m} \times 7\text{ m} \times 4.2\text{ m}) + 1/3 \times 5\text{ m}\,[A_1 + A_2 + \sqrt{A_1A_2}]\\ &= 205.8\text{ m}^3 + 1/3 \times 5\text{m}\,[49\text{ m}^2 + 1\text{ m}^2 + \sqrt{49 \times 1\text{ m}^2}]\\ &= 205.8\text{ m}^3 + 95\text{ m}^3\\ &= 300\text{ m}^3\end{aligned}$$

Provide 0.6 m freeboard.

2. Provide a sludge comminutor.
   Sludge grinding is recommended with a belt filter press. Provide a sludge comminutor to grind and macerate the digested sludge.
3. Compute the size of the polymer storage facility.
   Provide 30-d storage at average conditions.

$$\text{Polymer needed per average day} = 0.005 \times 5021\text{ kg/d} = 25\text{ kg/d}$$
$$\text{Total quantity needed per month} = 25\text{ kg/d} \times 30\text{ d} = 750\text{ kg}$$

Provide a storage area attached to the building to store the polymer in bags. Unload polymer daily into the hopper for feeding to the polymer mixing tank.

$$\begin{aligned}\text{Quantity of polymer needed per operating day} &= 25\text{ kg/d} \times \frac{7\text{ d/week}}{5\text{ operating d/week}}\\ &= 35\text{ kg/d}\end{aligned}$$

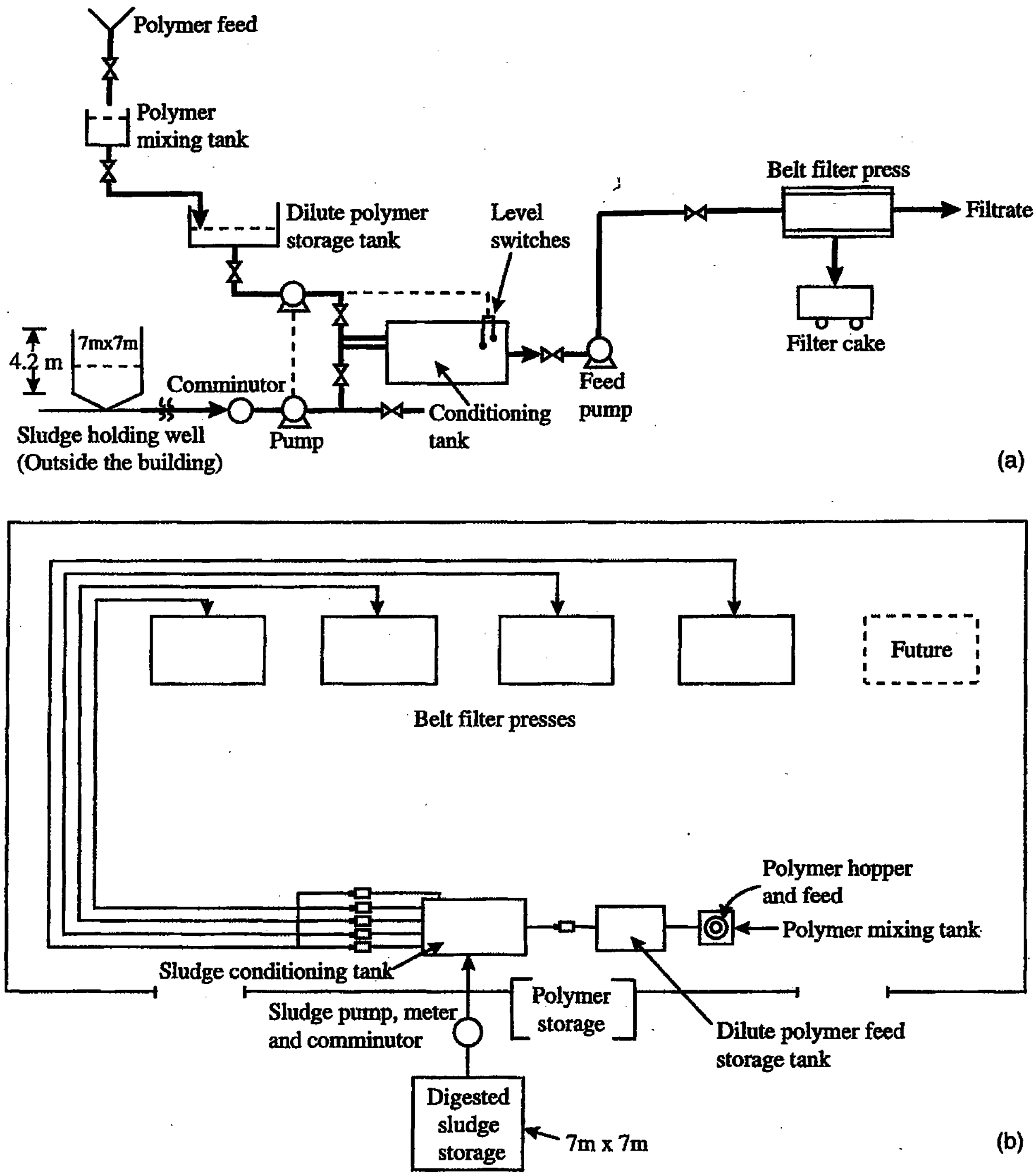


Figure 18-14 Details of Belt Filter Press Building for Sludge Dewatering: (a) typical flow schematic of in-line conditioning system, (b) plan of filter press building for the Design Example.

Continued

(c) (d) (e)

Figure 18-14—cont'd (c) photograph of a typical belt filter press installation, (d) photograph of a typical chemical conditioning and pumping installation, and (e) photograph of a typical polymer storage facility.

Provide one hopper for feeding polymer into the mixing tank.

The polymer solution goes into a dilute polymer storage tank and then into the sludge-conditioning tank.

$$\text{Assume bulk density of polymer} = 321\ \text{kg/m}^3\ (20\ \text{lb/ft}^3)$$

$$\text{Volume of hopper} = \frac{35\ \text{kg/d}}{321\ \text{kg/m}^3} = 0.11\ \text{m}^3 (110\ \text{L})$$

4. Compute the size of the digested sludge pump to the conditioning tank.
   The conditioning tank is connected to four belt filter presses. Assuming three filter presses are operating,

$$\begin{array}{l}\text{Total sludge pumped}\\ \text{per operating day}\\ \text{per pump}\end{array} = \frac{98\ \text{m}^3/\text{d} \times 7\ \text{d/week}}{5\ \text{d/week (operation)}}$$

$$= 137\ \text{m}^3/\text{d}$$

Provide two identical variable flow pumps, each with a maximum pumping capacity of 140 m^3/d or 97 Lpm (26 gpm).

5. Compute the size of the conditioning tank.
   Provide the in-line conditioning tank with a 30-min detention time when all three filters are operating.

$$\text{Volume of conditioning tank}^{b} = 140\ \text{m}^3/\text{h} \times \frac{30\ \text{min}}{1440\ \text{min/d}}$$

$$= 3.0\ \text{m}^3$$

   Provide the rectangular tank with level switches to control the operation of sludge and polymer feed pumps.
6. Compute the size of the conditioned sludge feed pump to the belt filter presses.
   Each belt filter press receives the conditioned sludge by a separate pump. There are a total of four belt filters and four pumps (one unit is a standby).

$$\text{Capacity of each pump} = \frac{137\ \text{m}^3/\text{d}}{3} = 46\ \text{m}^3/\text{d}$$

$$= 46\ \text{m}^3/\text{d} \times 1000\ \text{L/m}^3 \times \frac{1}{1440\ \text{min/d}}$$

$$= 32\ \text{Lpm}\ (8.5\ \text{gpm})$$

   Provide variable flow pump each with a maximum capacity of 50 Lpm (11 gpm).

Step D: Design Details. The design details of the filter press assembly, chemical feed systems, and conditioning units are given in Figure 18-14.

18-7 OPERATION AND MAINTENANCE AND TROUBLESHOOTING AT BELT FILTER PRESS

Important operation and maintenance considerations of filter press assembly are briefly described below. Additional details may be found in Refs. 10–13.

18-7-1 Common Operational Problems and Troubleshooting Guide

1. If dewatered sludge is not dry enough, the probable cause may be that the sludge application rate is too high, belt speed is too high, or there is an incorrect polymer dose. Check the sludge pumping rate and adjust the influent rate. Check and adjust the belt speed. Conduct a jar test procedure to determine the optimum polymer dose.
2. Excessive belt wear is an indication of improper alignment of the rollers. Check the tracking of the belt to see if it creeps to one side. Make proper adjustment of the rollers. Often, sludge buildup on the bottom of the belt on rollers may cause improper roller alignment. Check the operation of the automatic belt adjuster and re-

[b]Some polymers may require a certain time period after mixing. Sufficient storage volume must be provided for this purpose. In-line conditioning tank with each filter assembly is desirable.

place or repair faulty adjuster mechanism. Also, check the bottom of the belt and clean solids that may have accumulated.

3. A high solids carryover in the filtrate may be caused by an incorrect polymer dose or solids running off the edge of the filter belt. Check the dilution water feed rate and the polymer mixing and dosing system. Conduct a jar test to adjust the dosage. Check the influent sludge pumping rate and belt travel rate. Reduce the sludge pumping rate and adjust the belt travel rate if necessary.
4. Oil leaks may be caused by seal failure. Check and replace the oil seal.
5. Noisy or hot bearings or universal joint is an indication of excessive wear caused by improper alignment or lack of lubrication. Replace, lubricate, or align joint or bearings as required.
6. Extrusion of sludge in the gravity zone may be caused by worn-out rubber seals. Replace the seals. If the gravity zone has poor drainage, the problem may be caused by poor flocculation or belt blinding. Check the polymer feed system. Check and correct the spray nozzles for belt cleaning.
7. Extrusion of solids from a wedge section may be caused by poor flocculation, the belt speed being too slow, or the throughput being too high. Check and make proper corrections.
8. Extrusion in the high-pressure zone may be caused by poor flocculation or too much pressure on the slurry by the belts. Check the polymer feed system, decrease belt tension, and increase belt speed.
9. A bulge in the high-pressure section may be caused by too much water in the cake, or the belt may be blinded. Check the polymer feed, reduce belt speed, and check spray nozzles.
10. Belt slippage on the drive roller may be caused by too low a belt tension or too much sludge in the machine. Increase the belt tension, shut down the machine, and remove the excess sludge.
11. Cake sticking to the belt may result from poor flocculation or misaligned or worn-out scraper (doctor) blades. Check the polymer feed, and align and adjust scraper blade pressure.
12. Belts wrinkling or folding may be caused by poor distribution of feed, incorrect belt tension, the cake being too thick, or plows or baffles being mislocated. Take the proper steps to correct the problem.
13. The overload relay from the drive system may kick on because of an overloaded drive or overloaded relay. Adjust belt tension or replace the relay.
14. If improper filter belt is specified, the cake will stick, causing belt cleaning difficulties. Change the belt and specify a relatively coarse belt media.
15. Cake transport conveyor may not operate properly if the slope exceeds 15°.

18-7-2 Routine Operation and Maintenance

A good preventive maintenance program will reduce breakdown and make the operation clean and pleasant.

1. Examine the belt routinely for abnormal wear, damage, holes, or tears. The life expectancies of the belt and the pin wires vary from application to application. Keep

spare belts of each length and spare pin wire for each belt. Also, keep a record of date of installation of the belt, the source of the belt, specifications, estimated hours of operation, and the nature of any belt failures.

2. Keep a constant check on the following operating conditions: (a) sludge feed rate, (b) polymer feed rate, (c) belt speed, (d) sludge and polymer mixer, (e) plow position, and (f) spray wash system, especially nozzles and their setting.
3. Inspect daily the following routine maintenance items and take immediate corrective action if an unusual situation that may have developed: (a) roll coating for tears, (b) wedge and wedge setting, (c) condition of bearings, (d) air compressor, (e) belt tracking device, and (f) pneumatic system oiler.
4. Perform the following routine maintenance at specified time intervals: (a) grease bearing, (b) check lubricant level in the main drive reducer, (c) change main drive reducer oil, and (d) repack main drive motor bearings.

18-8 SPECIFICATIONS

The specifications of filter press equipment are briefly presented to describe the general features of the equipment. Detailed specifications should be developed in consultation with the equipment suppliers.

18-8-1 General

The contractor shall furnish and install four identical belt filter presses and associated piping, valves, controls, wiring, and appurtenances as specified and shown on the drawings. The belt filter press system specified in this section shall be provided by a single manufacturer to ensure coordination and compatibility of equipment.

18-8-2 Submittals

1. The contractor shall submit for review a complete assembly, foundation, and installation drawings, together with detailed specifications. The information submitted for review shall cover materials used, power drive assembly, parts, instrumentation devices, and other equipment accessories.
2. Owner's manuals.
3. A manufacturer's certificate of satisfactory installation.
4. Only those manufacturers capable of providing irrefutable evidence of a minimum of 10 years' experience in the manufacture and installation of the exact model or equivalent model meeting the filtration area requirements shall be considered.

18-8-3 Belt Filter Press

1. The belt filter press shall be designed to extract water from the sludge after conditioning of the sludge with an appropriate flocculant. This process of dewatering shall be accomplished by the combination of chemical conditioning of the sludge, drainage of free water in the horizontal gravity zone, the gentle compression of the sludge in the wedge zone, and the compression of the stabilized

solids in the pressure/shear zone. Each belt filter press shall have a 1.2 m effective dewatering width, and a combined effective filtration width of 4.8 m (including one standby unit). All moving wetted parts and all wetted parts on which moving parts contact shall be fully corrosion-resistant for the material being processed. All components of the belt filter press shall be designed to withstand all stresses that may occur during erection and operation.

2. The belt filter press construction shall allow easy access to internal components; operational adjustments and routine maintenance shall be possible without taking the machine out of service. Any disassembly required for maintenance and repair shall be possible within the clearances shown on the drawings.
3. The belt press shall be delivered to the job site as a completely assembled package and ready for service after connection of piping, wiring, and utilities. All pipings provided with the belt filter press shall be schedule 80 PVC or flexible hose.
4. The belt filter press shall be designed such that the "sludge" side of the belt does not come into contact with the roller face in order to prevent the accumulation of material on the roller assemblies.

18-8-4 Components

Each belt filter press shall include a structural frame, sludge/polymer mixer, gravity drainage section, pressure/shear dewatering section, filtrate collection/drainage system, dewatering belts, belt drive assembly, pneumatic/hydraulic system, belt tracking systems, belt tensioning system, scraper (doctor) blades and cake discharge, belt wash stations, roller assemblies, bearings, electrical components, system controls, and any other specified and necessary components.

Structural Frame. The design of the structural frame shall be based on a minimum belt tension of 10 kg/linear cm (50 lb per linear in.) of belt width. The structural frame shall be constructed of welded and bolted structural steel members. The frame shall be designed such that roller assemblies can be removed from the side or end of the belt press without removing structural members or repositioning more than one roller assembly.

The structural members shall be a wide flange beam, conforming to the standard specifications for structural steel (ASTM A36) and shall meet deflection yield point requirements. The structural frame shall be "welded in place" and designed to lift the fully assembled belt press.

Sludge/Polymer Mixer. The belt filter presses shall be provided with a sludge/polymer mixing tank for mixing and flocculation of feed sludge conditioned with polymer.

Gravity Drainage Section. The belt filter press shall be provided with a sludge inlet assembly consisting of a distribution chute and underflow leveling weir designed to uniformly distribute the conditioned feed sludge across the entire working width of a horizontal gravity drainage section consisting of a required minimum working belt area.

Pressure/Shear Dewatering System. The belt filter press shall have a pressure/shear dewatering zone wherein which increasing pressure and shearing forces are applied to the sludge. The wedge dewatering zone shall consist of a minimum required area, utilizing the combined surface area of each belt actually contacting the sludge while the wedge opening is in a maximum position. The "S" rolls in the pressure/shear zone shall be positioned to facilitate access to the internal working areas of the belt filter press for wash down, maintenance, and process optimization.

Filtrate Collection/Drainage System. The belt filter press shall be provided with drainage pans and piping to collect and discharge dewatered filtrate from the gravity drainage and pressure/shear dewatering sections. All filtrate shall be captured and contained by the drainage pans without spilling onto the floor.

Dewatering Belts. The press shall incorporate the use of two dewatering belts. The belts shall be seamed and fabricated to uphold the wear resistance, tensile stress equal to five times the normal maximum dynamic tension, provide optimum dewatering with minimum blending, and provide a minimum life of 2000 h of continuous operation.

Belt Drive Assembly. The belt drive assembly shall consist of a speed reducer, panel-mounted variable frequency drive controller for belt speed adjustment.

Pneumatic/Hydraulic System. The belt filter press shall be provided with pneumatic or hydraulic belt tracking and tensioning systems to ensure reliable operation. The belt tracking and tensioning systems shall be of the continuous and nonincremental tracking type. The hydraulic control system shall consist of pumps, reservoirs, coolers, filters, and pressure relief valves.

Belt Tracking System. The belt tracking system shall automatically and continuously align and maintain the belt position on the rollers during operation of the belt filter press. The belt position shall be monitored by a stainless steel sensing arm that shall continually contact the belt edge.

Belt Tensioning System. The belt filter press tensioning system shall be capable of adjusting belt tension to a maximum of 10 kg/linear cm (50 lb per linear in.) of belt width. Belt tension adjustments shall be manually controlled and shall be capable of adjustment while the belt filter press is operating. A pressure gauge shall be provided for both upper and lower belt tension regulators.

Scraper (Doctor) Blades and Cake Discharge. The belt filter press shall be provided with a scraper blade to assist the separation of cake from the belt at the point of cake discharge. The blade and blade holder shall be designed with sufficient stiffness to prevent warping, bowing, or distortion of the blade. The belt filter press shall be equipped with a minimum of 14-gauge stainless steel discharge chute, with a minimum 1:1 slope, to guide the discharge cake onto the sludge conveyor or disposal container.

Belt Wash Stations. The belt filter press shall be provided with an upper and lower belt wash station, which shall clean the full width of the belts after the cake has been discharged.

Roller Assemblies. All rollers shall be statically balanced and machined to ensure total concentricity.

Bearings. The shafts of all rollers shall be supported by greaseable type, self-aligning, spherical roller bearings housed in a sealed, splashproof, horizontal split case, cast closed pillow block housing.

Electrical Components. The belt filter press shall be supplied with the following NEMA 4X rated components: fiberglass terminal box, emergency stop trip cords (one on each side), and a belt tracking limit switch (one on each side).

System Controls. The belt filter press shall be supplied with a NEMA 4X stainless steel control panel. The control panel shall include a main disconnect, control power transformers, a variable frequency drive for the belt filter press main drive, control relays, alarm relays, selector switches, pilot lights, and pushbuttons, as required for a complete integrated control system. This control panel shall be suitable for 460-VAC, 3-phase, 60-Hz service.

18-8-5 Performance Test

The performance testing for the belt filter press shall be performed after installation. After plant startup, the manufacturer shall conduct a performance test using the owner's sludge to determine the actual system operating conditions and verify that the unit meets the minimum requirements specified.

18-9 PROBLEMS AND DISCUSSION TOPICS

18-1 Assume that 2000 kg of digested sludge solids reach drying beds for dewatering each day. The total solids in the sludge are 5 percent and specific gravity is 1.025. The sludge-drying beds are uncovered, and solids loading is 110 kg/m^2 year. Calculate (a) the number of beds if each paved bed is 4 m $\times$ 20 m, (b) how many beds will be filled each day if the average sludge application rate is 25 cm, and (c) what the sludge drying bed area is in m^2 per capita if digested sludge production is 0.05 kg per capita per day.

18-2 A vacuum filter is designed for dewatering combined digested sludge. The influent digested sludge contains 5 percent solids and specific gravity is 1.025. The volume of influent sludge is 200 m^3/d. Laboratory tests show that filter yield is 20 kg/m^2·h and optimum chemical dosage is 9.0 percent lime and 3 percent ferric chloride. The filter cake has 22 percent solids, including conditioning chemicals. If 75 percent of the added chemicals are fixed into the cake solids and solids capture efficiency is 90 percent, calculate (a) the dimensions of the vacuum filter, (b) the chemical dosage kg/d and kg/kg of solids, and (c) the weight of filter cake produced per day (kg/d). Assume filter operation is 16 h/d and the specific gravity of the filter cake is 1.06.

18-3 Calculate the number of variable volume recessed plate filter units to dewater 5000 kg/d digested sludge under average flow conditions. The filter operation is 8 h/d and 5 d/week. The lime, ferric chloride, and polymer dosages are 6, 1.3, and 0.5 percent of dry solids, respectively. The filter cycle time is 80 min, and the filtration rate is 15 kg/m^2·h. The filter area of each chamber is 0.5 m^2. Also calculate how many chambers in each filter unit shall be provided so that 100 percent filtration can be achieved with one unit out of service, and 124 percent filtration capability is available when all units are in operation.

18-4 A belt filter is designed for dewatering of sludge from a municipal wastewater treatment plant. The conditioned digested sludge that reaches the belt filter is 3360 kg/d at 6 percent solids and has a specific gravity of 1.03. The design solids loading rate is 200 kg per meter belt width per hour. The sludge cake is at 22 percent solids and has a specific gravity of 1.06. Assume that filter operation is 5 d/week. Calculate (a) the effective belt width and number of belt filters if available belt is 1.0 m wide; (b) the volume of filter cake if solids capture efficiency is 95 percent; and (c) the volume of filtrate, assuming belt wash water is 30 L/kg TSS in sludge and TSS concentration in washwater is 15 mg/L; and (d) the concentration of TSS in filtrate.

18-5 The TSS and BOD_5 contents of sludge cake and filtrate calculated under sustained loading conditions are given in Table 18-6 and compared with the values obtained from mass balance analysis summarized in Table 13-13. Calculate Org.-N, NH_4^+-N, TN, TPP, TP, TVSS/TSS, and biodegradable solids/TSS for sustained loading and compare with those obtained from mass balance analysis.

18-6 Determine the volume and concentration of TS and BOD_5 in return flow from a dewatering facility using filter presses. The average volume of digested sludge reaching the dewatering facility is 100 m^3/d. The specific gravity of the sludge is 1.025, total solids are 5 percent, and the biodegradable portion of total solids is 35 percent. The filters operate 8 h/d and 5 d/week. The cycle time is 60 min, and there are five filter units. Lime and polymer dosages are 6 and 3 percent of dry solids, respectively. Assume 75 percent added chemicals are fixed into the sludge cake. Biodegradable solids/BOD_L = 1.42, and BOD_5/BOD_L = 0.68. Also calculate the quantity and volume of sludge cake if the average moisture content in the sludge cake is 75 percent by weight and specific gravity is 1.06. Solids capture efficiency of filter presses is 88 percent. The lime and polymer solutions used in chemical conditioning are 10 and 5 percent, respectively. The water used per cycle for filter washing and leakage through the diaphragms is 4 m^3 per cycle per filter.

18-7 Anaerobically digested sludge is conditioned using hydrated lime and polyelectrolytes. Calculate the volume of conditioned sludge (m^3/d) and capacity of the sludge feed pump (m^3/h) used to pump conditioned sludge into the recessed plate filter press. Use the following data:

Average flow of anaerobically digested sludge	= 50 m^3/d
Solids in sludge	= 4000 kg/d
Lime dosage	= 5.5 percent of dry solids
Lime slurry solution	= 12 percent
Polyelectrolyte	= 2 percent of dry solids
Polyelectrolyte solution	= 5 percent

Sludge pump operating 8 cycles/d and 0.5 h/cycle
Filter press operation 6 d per week and 8 h/d

18-8 Describe various sludge-conditioning methods. Give advantages and disadvantages and the application of each method.

18-9 Describe various sludge-dewatering methods. Which methods would be best for the dewatering of sludge from a two-stage lime treatment process?

18-10 1000 kg digested sludge per day reach the sludge-drying earthen cells for dewatering. The total solid in the sludge is 7 percent, and specific gravity of the sludge is 1.025. The solids loading rate for the cells is 37 kg/m^3-yr. Calculate (1) the number of cells if each cell has a surface area of 0.02 hectare, 1 m depth, and side slope of 1 : 1, (2) in how many days each cell will be filled, and (3) the cell surface area in m^2 per capita if digested sludge production rate is 0.05 kg per capita per day.

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19

Sludge Disposal

19-1 INTRODUCTION

Ensuring the safe disposal of municipal sludge and other residues, such as screenings, grits, and skimmings, is considered an integral part of good planning, design, and management of municipal wastewater treatment facilities. Acceptable sludge disposal practices include (1) conversion processes (incineration, wet oxidation, pyrolysis, composting, etc.) and (2) land disposal (land application and landfilling).

Experience has shown that conversion processes are costly because they are either energy- or labor-intensive. Composting produces soil conditioners that can be used by farmers and homeowners and at city parks and highway medians. Composting is also labor-intensive and can only be successful if there is a market for the soil conditioner. Land application of municipal sludge requires a large amount of land. This method has been successfully practiced for decades and has received renewed interest in recent years. In situations where some sludges may cause accumulation of toxic chemicals in land or in crops grown on land, landfilling may be an attractive alternative for disposal of such sludges and other residues.

In this chapter a general overview of various methods of sludge disposal and reuse is presented. Since land application for beneficial uses and landfilling of municipal sludges are widely used practices in the United States, these topics are covered in greater detail. A step-by-step procedure for planning, design and operation of land application of municipal sludge is given in the Design Example.

19-2 OVERVIEW OF SLUDGE DISPOSAL PRACTICES

Both conversion and land disposal methods require sludge processing. Thickening, stabilization, conditioning, and dewatering are the common methods of sludge processing. These processes essentially prepare the sludge for safe ultimate disposal. Discussion of thickening, stabilization, conditioning, and dewatering of sludge may be found in Chapters 4 and 16–18.

The sludge conversion processes and land disposal methods are discussed below.

19-3 SLUDGE CONVERSION PROCESSES

Sludge conversion processes are generally thermal techniques and are intended to reduce the solids required for final disposal or to recover a resource. Composting is thermal or biological conversion of organic matter into a usable soil conditioner. The conversion and resource recovery processes are summarized in Table 19-1.[1] A general description is given below.

19-3-1 Incineration

Incineration involves drying of sludge cake, followed by complete combustion of organic matter. A minimum temperature of 700°C is needed to deodorize the stack emissions. Excess air 50–100 percent over the stoichiometric air requirement is necessary. Natural gas or fuel oil is provided as an auxiliary fuel for ignition and to maintain the proper temperature. Often, sludge is incinerated with municipal solid waste and other residues. Two major incineration systems are the multiple-hearth furnace and the fluidized-bed reactor. The multiple-hearth furnace contains several hearths arranged in a vertical stack. Sludge cake enters the top and proceeds downward through the furnace from hearth to hearth. The fluidized-bed incinerator utilizes a hot sand reservoir in which hot air is blown from below to expand and fluidize the bed. Air pollution control equip-

TABLE 19-1 Summary of Conversion Processes for Disposal of Municipal Sludges

Conversion Process	Recommended Pretreatment	Codisposal of Other Residue[a]	Additional Processing Requirements
Incineration	Thickening and dewatering	Yes	Ash landfilling
Wet oxidation	Thickening	No	Separation of ash, treatment of returned liquid, landfilling of ash
Pyrolysis	Thickening	No	Utilization of by-products (gas, liquid, carbon, etc), disposal of residues
Recalcining	Thickening and dewatering	No	Recovery of lime, landfilling of ash
Composting by heat drying	Thickening and dewatering	No	Utilization or sale of compost
Composting by microbial action	Thickening, digestion, and dewatering	No	Utilization or sale of compost

[a]Other residues include screenings, grits, and skimmings.
Source: Adapted in part from Refs. 1–3.

ment is needed in both cases to clean the emission. A discussion of sludge incineration may be found in Refs. 4 and 5.

19-3-2 Wet Oxidation

Wet air oxidation (Zimmerman process) is also used to incinerate the sludge. Dewatering of sludge is not required. The sludge and sufficient air is pumped into a reactor where high temperature (200–300°C) and high pressure (5–20 kN/m^2) are maintained. The organic matter is oxidized in a liquid phase. The liquid and solid residues are separated by settling or filtration. The liquid is high in BOD, nitrogen, and phosphorus, which must be returned to the plant. The ash is landfilled. A general discussion of wet oxidation may be found in Refs. 1, 6, and 7.

19-3-3 Pyrolysis

Pyrolysis, or destructive distillation, is heating of organic matter in an oxygen-free atmosphere. At high temperatures of 300–700°C and in the absence of oxygen, the organic matter undergoes cracking, producing gases, oil, tar, and charcoal. All three components are combustible and can be used as an energy source. Currently, pyrolysis of municipal solid wastes on a large scale is in practice. Sludge pyrolysis or copyrolysis with municipal solid wastes will probably be used in the future.[1,6,8]

19-3-4 Recalcining

The recalcining and reuse of lime sludges in wastewater treatment plants is new. A high rate of lime addition in one or two stages produces large quantities of sludge. Recalcining involves heating of dewatered lime sludge to about 1000°C in a multiple-hearth furnace. Moisture and CO_2 are driven off. Often, CO_2 is used in the recarbonation process. At the bottom outlet of the furnace, large pieces of recalcined lime are obtained, which are ground and taken to the storage area. Lime recovery at the wastewater treatment plant may not be feasible, owing to the presence of undesirable impurities, particularly nitrogen and phosphorus. A portion of lime sludge must be wasted to avoid buildup of impurities. The cost of recalcining at a wastewater treatment plant must be weighed against the cost of new lime, savings in cost from reclaimed lime, and reduction in sludge quantities for disposal.[4,5,9]

19-3-5 Composting by Heat Drying

The purpose of heat drying is to remove the moisture from the wet sludge and partially combust the organics. Sludge drying occurs at temperatures of approximately 370°C. At this temperature, part of the volatile matter is removed, and the sludge is stable enough for use as compost. The gases evolved in the drying process are reheated to about 750–800°C to eliminate odors. The most common types of heat-drying systems for sludge are (1) flash dryers, (2) spray dryers, (3) rotary dryers, (4) multiple-hearth dryers, (5) multiple-effect evaporators, and (6) the Carver-Greenfield process.[1,4] The sludge after drying is processed into a soil conditioner. Drying permits grinding, weight reduction, and prevention of continued microbial decomposition.

19-3-6 Composting by Microbial Action

Composting is a process in which organic matters undergo biological decomposition to produce a stable end product that is acceptable as a soil conditioner. There are two methods of composting: open windrow and mechanical process. Windrow composting uses long, narrow piles 1.2 m high and 2.5 m wide. These piles are turned every few days. Moisture is maintained at 55–70 percent. It takes 2–3 weeks to produce stable compost. Mechanical systems of composting produce stable compost in 5–10 d. Co-composting of liquid and dewatered sludge with municipal solid wastes is also used. There has been a limited market for compost in this country. A discussion of composting may be found in Refs. 4–6, 10, and 11.

19-4 LAND APPLICATION OF MUNICIPAL SLUDGE (BIOSOLIDS)

Land application of municipal sludge or biosolids has been practiced in many countries for centuries. The interest in land application of biosolids has increased in recent years because of constraints and increasing costs of many other disposal alternatives. Smaller communities tend to use land application more than larger communities do. It is estimated that the quantity of biosolids marketed or distributed free for land applications may reach 40 percent of total solids production.[12] The land application options of biosolids include

1. Agricultural utilization to enhance production of food-chain and non-food chain crops
2. Forest land utilization to enhance forest productivity
3. Land reclamation utilization to revegetate and reclaim land disturbed by strip-mining
4. Dedicated land disposal, with or without vegetation, for the primary purpose of sludge disposal. Crop production (if any) is of secondary importance. Landfilling is excluded.

A combination of two or more options mentioned above may also be used. Direct land disposal of digested liquid sludge to cropland has the advantage of eliminating the dewatering cost, but the disadvantage is the large volume of sludge that must be transported and applied over land. Small cities often haul the liquid sludge in trucks, while large cities utilize pumping. Dewatered sludge may be disposed of by spreading over farmlands and plowing after it has dried. Wet sludge may be incorporated into the soil directly by injection.

The above four types of land disposal options (and combinations of two or more) are common methods for land disposal of sludge. Each of these options has specific requirements in terms of the quality and quantity of sludge that can be utilized over a selected site. In the following sections the quality and quantity of sludge for land application and site evaluation and selection are presented first, followed by four types of land disposal options.

TABLE 19-2 Concentration Limits on Sludge Quality and Application[13]

Pollutant	Ceiling Concentration[a] (mg/kg)	Monthly Avg. Concentration[b] (mg/kg)	Annual Pollutant Loading Rate[c] (kg/ha/yr)	Cumulative Pollutant Loading Rate[d] (kg/ha)
Arsenic	75	41	2.0	41
Cadmium	85	39	1.9	39
Chromium	3000	1200	150	3000
Copper	4300	1500	75	1500
Lead	840	300	15	300
Mercury	57	17	0.85	17
Molybdenum	75	—	—	—
Nickel	420	420	21	420
Selenium	100	36	5.0	100
Zinc	7500	2800	140	2800

[a]Bulk sewage sludge sold or given away shall not be applied to the land if concentration exceeds ceiling concentration.
[b]If bulk sewage sludge is applied to a lawn or a home garden, the concentration of each pollutant shall not exceed the given monthly average concentration.
[c]If sewage sludge is sold or given away for application to the land, either the concentration of pollutant should not exceed the monthly average concentration, or the annual pollutant loading rate shall not exceed the given value.
[d]If bulk sewage sludge is applied on agricultural land, forest, or a public contact site, either the monthly average concentration of each pollutant shall not exceed, or the cumulative loading rate for each pollutant shall not exceed, the given value.
All mg/kg concentrations are on dry weight bases.
Note: kg/ha × 0.89 = lb/acre.

19-4-1 Quality and Quantity of Sludge for Land Application

Regulatory Control. The U.S. Environmental Protection Agency's Part 503 Sludge Regulations impose significant monitoring, reporting, and recordkeeping requirements on POTWs. These requirements will help ensure environmental quality with long-term application of sludge or biosolids on land. A sludge is considered clean if the concentration of certain pollutants is not exceeded. Additionally, certain pathogens and vector reduction requirements must also be met for land application. The requirements for pathogens vary, depending on the classification of the sludge, either Class A or Class B.[a] The concentration limits on heavy metals in sludge for land application are provided in Table 19-2.[13]

[a]The Class A or Class B requirements are based on a number of alternative testing procedures and methods, including density of fecal coliforms, density of *Salmonella,* temperature and pH maintained for the period of time used in sludge processing, and viable helminth ova or enteric viruses. Class A pathogen requirements shall be met when bulk sewage is applied to a lawn or a home garden or sold or given away in a bag or other container for application to the land. The Class B pathogen requirements and site restrictions shall be met when bulk sewage sludge is applied to agricultural land, forest, a public contact site, or a reclamation site.

Characteristics of Sewage Sludge. Reliable information on sludge composition is needed to select and design land disposal methods. Many factors influence the composition of sludge. Among these are proportion of industrial and residential input, urban runoff, pretreatment requirements, and treatment processes used. Several studies have been conducted to develop data on chemical characteristics of sewage sludge in various states.[14-17] The mean values of many chemical constituents for digested sludge are provided in Table 16-1.

Pathogenic organisms such as bacteria, viruses, protozoa, and parasitic worms are almost always present in raw sewage. Sludge stabilization processes destroy the great majority of these pathogens. Nevertheless, most stabilized sludge will still contain some pathogens, and safeguards are necessary to protect against possible contamination of operating personnel, general public, and crops. Significant pathogen kill is expected after sludge is applied over land.

19-4-2 Site Evaluation and Selection

Proper site selection is perhaps the most important factor for selection of land application options (agricultural use, forest, land reclamation, and dedicated disposal use). Additionally, proper site selection will reduce future environmental problems, monitoring requirements, overall program costs, and public reaction. The factors to be considered in site selection are many and are quite complex. A two-phase approach for site evaluation and selection are given in Figure 19-1.[12] For preliminary screening purposes (Phase I), the physical characteristics, land availability, and rough estimate of land area requirements for each feasible option is made using the loading rate. The final site selection (Phase II) involves careful final evaluation. The land area requirements are based on design loading rates plus buffer zones and other land area requirements. Physical site characteristics of concern are topography, soil permeability, site drainage, depth of groundwater, subsurface geology, proximity to critical areas, and accessibility.[4] Typical guidelines used for site evaluation are summarized in Table 19-3.

19-4-3 Agricultural Land Utilization

As mentioned in Sec. 19-4-1, the agricultural land utilization is one of the four options of land disposal of municipal sludge. This utilization option is designed for nitrogen and/or phosphorus need for the crop. The application is at agronomic rates (usually on an annual basis) to ensure acceptance by the farmers and to minimize environmental monitoring. As a general guide, the sludge application rate may be 2–70 mt/ha·yr (1–30 T/acre·yr).[b] The sludge application is annually between harvesting and planting. Scheduling can be complex with large systems. This option (1) reduces or eliminates commercial fertilizer, (2) may improve production, and (3) with proper site selection and application rates, the potential harmful impacts on phytotoxicity, ground and surface water contamination, and public health or nuisance may be avoided. The typical minimum depths to groundwater for agricultural use is given in Table 19-3. The sludge application

[b]mt = metric ton or 1000 kg; mt/ha·yr = mt per ha per year; mt/ha × 0.445 = T/acre.

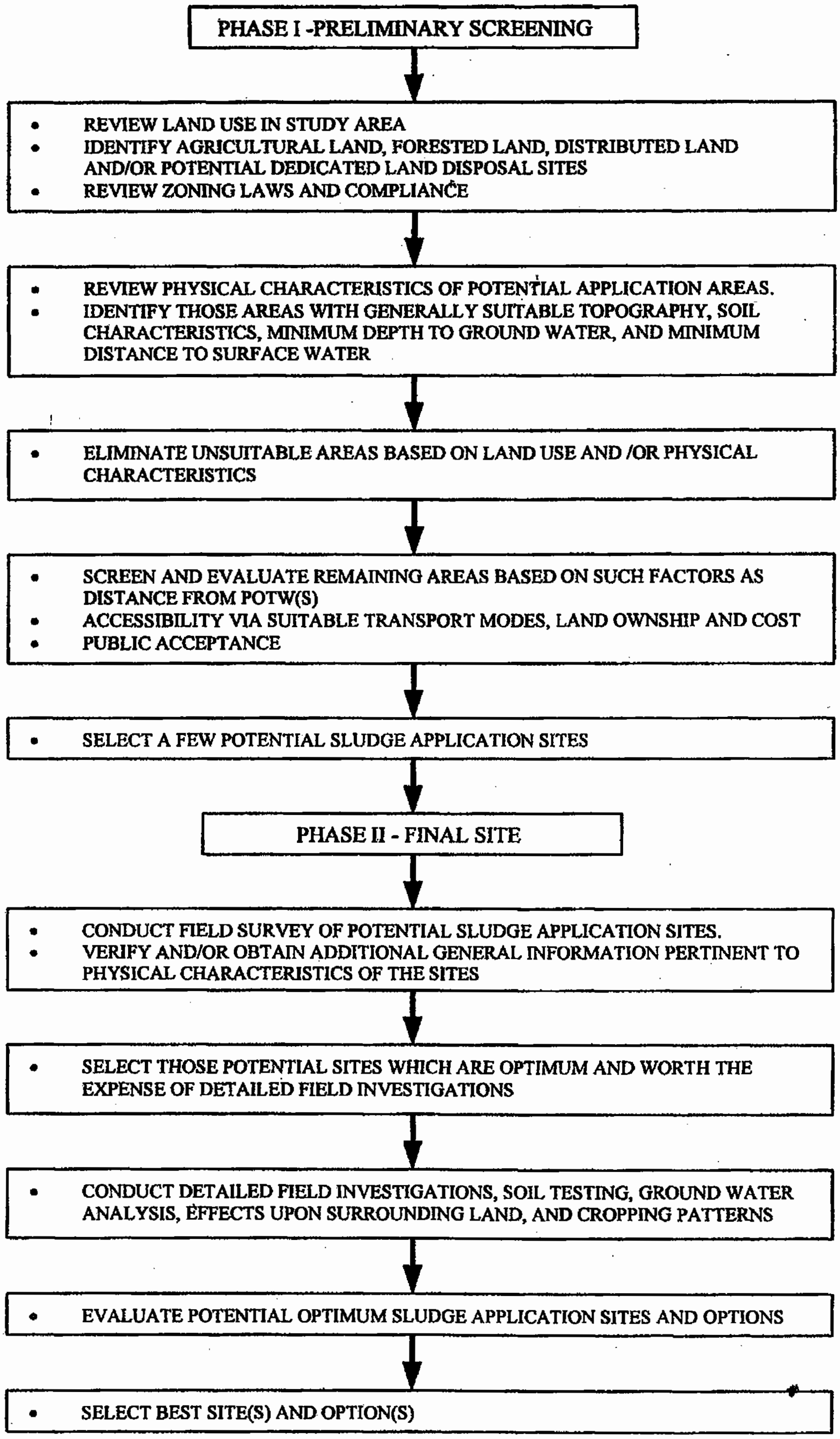

Figure 19-1 Two-Phase Approach to Sludge Application Site Identification, Evaluation, and Selection (from Ref. 12).

TABLE 19-3 *Soil Limitations for Sewage Sludge Application to Agricultural Land at Nitrogen Fertilizer Rates*

	Degree of Soil Limitation		
Soil Features Affecting Biosolids Use	**Slight**	**Moderate**	**Severe**
Slope[a]	<6%	6–12%	>12%
Depth to seasonal water table	>1.2 m	0.6–1.2 m	<0.61 m
Flooding and ponding	None	None	Occasional to frequent
Depth to bedrock	>1.2 m	0.6–1.2 m	<0.61 m
Permeability of the most restricting layer above a 1-m depth[b]	0.24–0.8 cm/h	0.8–2.4 cm/h	<0.08 cm/h
		0.08–0.24 cm/h	>2.4 cm/h
Available water capacity	>2.4 cm	1.2–2.4 cm	<1.2 cm

[a]Slope is an important factor in determining the runoff that is likely to occur. 0–3% is ideal, no concern for runoff, 3–6% is acceptable, slight risk for erosion; 6–12% medium runoff expected, surface application of dewatered sludge is usually acceptable; greater than 12% may have rapid to very rapid runoff. Runoff control measures may be necessary

[b]Permeability rating (cm/h) is as follows: <0.15 very slow, 0.15 to 0.5 slow, 0.5 to 1.5 moderately slow, 1.5 to 5.1 moderate, 5.1 to 15.2 moderately rapid, 15.2 to 51 rapid, >51 very rapid. 1 ft = 0.3048 m; 1 in = 2.54 cm.

Source: Adapted from Refs. 3 and 12.

rate is generally controlled by regulatory guidelines or nutrients loading rates to meet the regulation requirements. Generally, nitrogen is the nutrient of principal concern. The pollutant loading, nitrogen limitation, phosphorus limitation, and land requirements are expressed by several equations presented in later sections.

Pollutant Loading. The cumulative amount of sludge on a dry weight basis that can be applied is calculated from Eq. (19-1). The limiting values of various contaminants are given in Table 19-2.

$$S_m = \frac{L_m}{C_m \times 1000} \tag{19-1}$$

where

S_m = maximum amount of dry solids in sludge that can be applied over the useful life of the site, mt/ha
L_m = maximum cumulative amount of pollutant that can be applied over the useful life of the site as established by regulatory laws (Table 19-2), kg/ha
C_m = concentration of concerned pollutant in sludge, decimal fraction (e.g., for 100 mg/kg, $C_m = 0.0001$)
1000 = kg/mt dry solids in sludge

Nitrogen Limitation. Nitrogen in sludge is mostly in the organic form and is slowly mineralized. Therefore, plant-available nitrogen has two components: (1) nitrate and ammonia nitrogen, which becomes available within the first year, and (2) organic nitrogen, which is released slowly in time. The plant-available nitrogen is calculated from Eq. (19-2):

$$N_p = 1000\,[(\mathrm{NO_3^-\text{-}N}) + K_v\,(\mathrm{NH_4^+\text{-}N}) + f\,(\mathrm{Org.\text{-}N})] \tag{19-2}$$

where

N_p = plant-available nitrogen during the application year, kg N/mt dry solids per year
NO_3^--N = nitrate nitrogen in sludge, decimal fraction of dry solids
NH_4^+-N = amimia nitrogen in sludge, decimal fraction of dry solids
Org.-N = organic nitrogen in sludge, decimal fraction of dry solids
K_v = volatilization factor for ammonia, 0.5 for surface or sprinkler application of liquid sludge, and 1.0 for dewatered sludge or incorporated liquid sludge
f = mineralization factor for Org.-N in sludge in the first year (Table 19-4), faction of Org.-N
1000 = kg/mt dry solids in sludge

It should be noted that N_p calculated from Eq. (19-2) is for the first year only. Additional nitrogen will be available from mineralization of Org.-N during the subsequent years. Two methods are used to calculate the mineralization rate of Org.-N for subsequent years: (1) stepwise calculations using corresponding values of f given in Table 19-4 and adding the values and (2) a simpler alternate method using Eq. (19-3).

$$N_t = K_t N_o \tag{19-3}$$

where

N_t = fraction of Org.-N mineralized in the year under consideration, kg/mt of dry solids
K_t = mineralization factor for the year under consideration, kg/mt·N_o. The values of K_t are given in Table 19-4.
N_o = Org.-N expressed as percent of dry solids

To illustrate both methods, an example is used. Suppose an anaerobically digested municipal sludge has NO_3^--N = 0, NH_4^+-N = 1.5%, Org.-N = 3%, and NH_4^+-N volatilization factor $K_v = 0.5$, calculate Org.-N mineralization in 2 years by both methods.

1. Stepwise calculations: N_p (1st yr) [Eq. (19-2)] = 1000 [(0) + 0.5 (0.015) + 0.2 (Table 19-4) (0.03)] = 13.5 kg/mt

   N_p (2nd yr) is obtained by adding Org.-N mineralized in the second year to the first year value. Org.-N mineralized in the second year from step calculation is as follows:

   - Org.-N originally present = 1000 × 0.03 = 30 kg/mt
   - Org.-N mineralized in first year = 0.2 (Table 19-4) × 30 kg/mt = 6 kg/mt
   - Org.-N remaining at the end of the first year = (30 − 6) kg/mt = 24 kg/mt
   - Org.-N mineralized in the second year = 0.1 (Table 19-4) × 24 kg/mt = 2.4 kg/mt
   - N_p' second year = 13.5 kg/mt + 2.4 kg/mt = 15.9 kg/mt

2. Simplified method: The above results can also be obtained from Eq. (19-3):

$$N_t = 0.8 \text{ kg/mt (Table 19-4)} \times 3 = 2.4 \text{ kg/mt}$$
$$N_p \text{ second year} = 13.5 \text{ kg/mt} + 2.4 \text{ kg/mt} = 15.9 \text{ kg/mt}$$

Total plant-available nitrogen provided by sewage sludge for a given application rate over a target period can therefore be calculated from Eqs. (19-2) and (19-3). The calculation procedure for sludge application based on nitrogen requirements is provided in the Design Example.

Phosphorus Limitation. The annual phosphorus application rate may also be based on the P requirements of the crop grown. The P fertilizer needs of the crop grown are determined from the soil tests for the available P and the yield level of the crop grown. The

TABLE 19-4 Estimated Percentages and Amounts of Org.-N Mineralized after Sludge of Various Types Are Applied to Soils

Time after Sludge Application (years)	Unstabilized Primary and Waste Activated		Aerobically Digested		Anaerobically Digested		Composted	
	f[a] Fraction of Org.-N	K_t[b] (kg/mt)	f Fraction of Org.-N	K_t (kg/mt)	f Fraction of Org.-N	K_t (kg/mt)	f Fraction of Org.-N	K_t (kg/mt)
0–1	0.4	4.00	0.3	3.00	0.2	2.00	0.10	1.00
1–2	0.2	1.20	0.15	1.05	0.1	0.80	0.05	0.45
2–3	0.1	0.48	0.08	0.45	0.05	0.36	0.03	0.25
3–4	0.05	0.22	0.04	0.21	0.03	0.21	0.03	0.25
4–5	0.03	0.12	0.03	0.16	0.03	0.20	0.03	0.24
5–6	0.03	0.12	0.03	0.15	0.03	0.19	0.03	0.23
6–7	0.03	0.12	0.03	0.15	0.03	0.19	0.03	0.23
7–8	0.03	0.11	0.03	0.15	0.03	0.18	0.03	0.22
8–9	0.03	0.11	0.03	0.15	0.03	0.18	0.03	0.21
9–10	0.03	0.11	0.03	0.15	0.03	0.17	0.03	0.21

[a]Fraction of Org.-N based on dry solids.
[b]kg N released per metric ton of sludge applied per percent Org.-N (N_o) in the dry solids. Multiply kg/mt by 2 to obtain lb/T.
Source: Adapted from Ref. 12.

amount of sludge applied is then equated to the P fertilizer requirements. The sludge application rate is calculated from Eq. (19-4):

$$S_P = \frac{C_P}{P_P \times 1000} \tag{19-4}$$

where

S_P = application rate of sludge to satisfy P fertilizer needs of crop, mt/ha
C_P = plant P needs, kg/ha
P_P = concentration of P in sludge, decimal fraction
1000 = kg/mt dry solids in sludge

Land Requirement. The land requirement is calculated from the design sludge loading rate. The design loading rate may be based on nitrogen, phosphorus, cadmium, or any other critical constituent. Eq. (19-5) is used to calculate the land area requirement:

$$A = \frac{Q_s}{R_d} \tag{19-5}$$

where

A = land area required, ha
Q_S = total sludge production, mt dry solids/yr
R_d = design sludge loading rate, mt/ha·yr

Design Considerations for Agricultural Use of Sludge. The amount of N, P, and K available in the sludge should be compared with the fertilizer needs of the crops to achieve the desired yields. If this comparison shows that one or more nutrients are suboptimal, the appropriate amount of commercial fertilizer should be added. To protect the productivity of soils and to determine long-term accumulation of Cd in the crops, sludge applications are terminated when the cumulative amounts of Cd (sometimes also Pb, Zn, Cu, and Ni) exceed a specific limit based on the CEC of the soil. The recommended limits are given in Table 19-2. The other monitoring requirements include soil pH, concentration of P and K in the soil, nitrate in groundwater, and cadmium in crops. Other design considerations are sludge application methods, sludge storage and scheduling, and climate. Design procedure for land application of municipal sludge is covered in the Design Example.

19-4-4 Forest Land Utilization

The sludge application to forest soils is also a major utilization/disposal option for sludge. Forested lands are abundant and well distributed throughout most of the United

States and are close to standard metropolitan areas.[12] Three categories of forested lands may be available for sludge disposal.

- recently cleared land prior to planting
- newly established plantations (about 3 to 10 years old)
- established forests

The availability of site, non-food crop, recycling of nutrients, and no cost to acquiring land are the major benefits. The normal sludge application range in forest land is 10–220 mt/ha (4–100 T/acre) per year, depending upon soil, tree species, and sludge quality. The sludge application may be one time over an interval of several years. Dramatic improvement in vegetative growth has been reported. Potential harmful impacts on vegetation, surface and groundwater, and human health may be avoided by proper selection of sludge, application rate, and management practices. The minimum distance to water depths for drinking water and excluded aquifers are 2 m and 1 m, respectively. Proper site selection, calculations for heavy metals accumulation, and limitations in nitrogen and phosphorus loading rates may be performed using the procedure discussed in the previous section.[12]

19-4-5 Land Reclamation and Utilization

The surface mining of coal and other minerals has generated mine spoils and drastically disturbed the marginal lands. These lands are unable to support vegetation because of nutrient deficiencies, lack of organic matter, physical properties (stony, clayey, etc.), chemical properties (pH, heavy metals), and topography. The sewage application range is 7–450 mt/ha (3–200 T/acre) per year depending upon the soil and site conditions, and application is one time. Additional applications are at 5- to 10-year intervals. The sludge nutrients are beneficially recycled, which eliminates commercial fertilizer, allows soil to support vegetation, and retards erosion. Proper sludge application and management may improve existing surface and groundwater pollution and other environmental problems. The minimum distances to drinking water and excluded aquifers are 1 m and 0.5 m, respectively. Detailed site investigations for topography, soil chemistry, climate, sludge nutrients, heavy metals, and sludge loading rates must be made on case-by-case study.

19-4-6 Dedicated Disposal

The primary purpose of dedicated land disposal site is long-term sludge spreading. Production of agricultural crops or improvement of soil characteristics are secondary to the sludge disposal activity. The sludge application rate is 220–900 mt/ha (100–400 T/acre) per year, depending upon soil, climate, etc. The application rate may be 5–20 days on a routine basis. Usually, the land cannot be used for agriculture. Sludge nutrients are not the primary objective. There is a potential danger for degradation of surface and groundwater quality. Therefore, proper site selection and substantial monitoring are required. The minimum distances to drinking water and excluded aquifers are 1 and 0.5 m. There are several types and designs used for dedicated land disposal sites. Proper site selection, evaluation of environmental constraints, nutrients, heavy metals, climate, vegetation grown, and sludge loading rates are all considered in the selection process and site development. Information on these topics may be found in Ref. 12.

19-4-7 Monitoring Requirements

Most state and local regulatory agencies specify monitoring requirements for a specific project, and they must be conducted. The conservative design approaches reduce the need for monitoring of soils, crops, and surface and groundwaters. The basic rationale for using sludge at agronomic rates are to substitute for commercial fertilizer, and also the site monitoring requirements are not as extensive. If the pH of the soil is maintained above 6.5, groundwater monitoring may not be required. As a general rule, the following parameters may require monitoring:

1. Soil: pH, K, P, some heavy metals
2. Groundwater: nitrate, fecal coliforms
3. Surface water: BOD, nitrogen, phosphorus, fecal coliforms
4. Crop: cadmium

Frequency may depend upon amounts applied, depth to groundwater, and climatological conditions.

19-5 SLUDGE DISPOSAL BY LANDFILLING

Raw or digested sludge is buried if a suitable site is available within an economical hauling distance. Sludge is often co-disposed at municipal sanitary landfills along with other solid wastes. Site selection and design considerations require that the sites have a minimum distance from populated areas, leachate protection, runoff and erosion control, and protection against gas movement. A monitoring program and future land use planning is essential. Landfilling of municipal sludge and other residues from wastewater treatment facilities is the most widely used method in the United States. It is expected that this practice will continue to be a major disposal option in this country. Therefore, the rest of this chapter is devoted to detailed discussion on planning, design, and operation of landfills for municipal sludge.

Planning, design, and operation of municipal sludge landfills requires detailed investigation. In addition to soil erosion, dust, vectors, and noise and odor problems, leachate and gases continue to be produced for years to come. There is the danger of ground and surface water contamination, as well as migration of explosive gases to the nearby structures if landfills are not properly designed and operated. Therefore, considerations should be given to the following factors for proper planning, design, and operation of sludge landfills:

1. Regulations and permits
2. Sludge characteristics
3. Site selection

4. Methods of sludge landfilling
5. Design considerations
6. Operation and maintenance
7. Monitoring of completed landfills

Each of these factors is discussed below.

19-5-1 Regulations and Permits and Public Participation

In the *Federal Register* of October 9, 1991, the U.S. Environmental Protection Agency published 40 CFR Parts 257 and 258, *Solid Waste Disposal Facility Criteria; Final Rule.*[18] The Final Rule of the EPA amends regulations under 40 CFR 257 and 403 and adopts a new Part 40 CFR 503 to establish requirements for the final use and disposal of sewage generated from POTWs. Sludge that is disposed of or co-disposed with municipal solid waste in a sanitary landfill will not have to meet numerical pollutant limits as with beneficial use regulated under 40 CFR 503. However, sludge must not be hazardous and must not contain free liquid. Toxicity Characteristics Leaching Procedure (TCLP) or Paint Filter Liquid Test (PFLT) may be necessary.[19] Environmental protection requirements for landfill such as liner installation and groundwater monitoring under Subtitle D are sufficient to protect the ground and surface waters.

Many regulatory agencies require a permit for construction and operation of sludge landfills. The design of the fill must follow the permit requirements, which are generally available from the federal, state, or local agencies. The federal requirements relevant to sludge landfills are contained in the criteria for the classification of solid waste disposal facilities.[18-21] The criteria address the following topic areas:

1. Environmentally sensitive areas
2. Surface water
3. Groundwater
4. Odors
5. Safety

The objective of a public participation program in the establishment of sludge landfills is to give the public a participatory role throughout planning, design, and operation. The public and concerned citizens must therefore be kept informed through news releases, mailing lists, and public hearings regarding the status of the site selection, development of design criteria, and operation phases of the project. A public participation program for the Design Example has been presented in Chapter 6. Additional details may be found in Refs. 1 and 22.

19-5-2 Sludge Characteristics

Not all sludges are suitable for landfilling because of either odors or operational problems. Moisture contents and chemical composition of sludge solids are important in the design and operation of sludge landfills. Sludges having solids contents of 15 percent will not support cover material, and therefore, dry soil as a bulking agent must be used. Soil-bulking operations are generally not cost-effective if the solids content in sludge is less than 15 percent. Severe operational problems could occur at the landfills if sludge-dewatering facilities are unable to produce sludge cake at a desired solids content.

Sludges from chemical precipitation facilities create different types of problems. These sludges are viscous, sticky, and slippery (particularly if polymers are used). Handling of these sludges is difficult even when bulking agents are used. Undigested sludges produce large amounts of methane gas that pose serious explosion problems to the nearby structures.

Heavy metals in wastewater generally come from industrial discharges. Pretreatment regulations were instituted to limit such discharges. A large portion of the heavy metals is removed by conventional treatment processes and is concentrated in the sludge. Typical concentrations of heavy metals in municipal sludge are summarized in Table 16-1. Heavy metals from sludges do not leach out as long as the pH remains above 10. For this reason and others, lime is added to the sludge prior to dewatering (Chapter 18). As anaerobic decomposition progresses in the fill, organic acids are produced that lower the pH, and release of heavy metals may begin.

Nitrogen release from landfills is also of particular interest. Nitrogen in the form of ammonia, nitrite, and nitrate are relatively mobile and cause serious risk of groundwater pollution.

In determining the suitability of sludges for landfilling, considerations must be given to (1) the source and characteristics of wastewater, (2) wastewater treatment processes, (3) sludge-handling and disposal methods, (4) concentration of heavy metals, (5) organic contents, and (6) nitrogen content. Each of these factors should be evaluated carefully in light of landfill site, surface and groundwater hydrology, and geology of the area. Proper safeguard techniques must be utilized in the design and operation of the fill to ensure minimal environmental consequences.

19-5-3 Site Selection

The technical considerations involved in site selection of sludge landfills include factors that span many disciplines. These disciplines include engineering, land use, social and political science, and economics. These disciplines are briefly discussed below.

Engineering

Useful Life. The selected site should last for several years to justify the cost of permanent structures. The land needed for access roads, buffer land, and stockpiles should be considered within the gross total site area. The calculation procedure for estimating the fill life may be found in Ref. 3.

Topography. Flat sites (slopes less than 1 percent) tend to pond. Steeper sites (slopes greater than 20 percent) could erode, thus creating operational difficulties.

TABLE 19-5 Properties of Soils and Their Suitability for Sludge Landfilling

Soil Type	Grain Size (mm)	Permeability (cm/s)	CEC (meq/100 g)	Suitability for Landfilling
Clay	0.002 and less	10^{-8}–10^{-6}	Over 20	Excellent. Impermeable liner may not be needed.
Silt loam	0.002–0.05	10^{-6}–10^{-3}	12–20	Fair. Impermeable liner may be needed.
Sandy soils	0.05–0.25	10^{-3}–10^{-1}	1–10	Poor. Impermeable liner is needed for protection of groundwater.

Source: Adapted in part from Ref. 1.

Surface Water. Surface runoff resulting from landfill areas must comply with the NPDES permit issued for the facility. Proper control methods must be considered in the design.

Soil and Geology. Soil serves as a cover and bulking material, attenuates potential contaminants, and controls runoff and leachates; therefore, the chemical, physical, and hydrological properties of soils must be studied. Important properties are pH, cation exchange capacity (CEC), texture, structure, distance from the groundwater table, and permeability. A detailed discussion of soil characteristics for sludge landfilling may be found in Ref. 1. Basic information is summarized in Table 19-5. Fine-textured soils are considered desirable for landfilling because they have lower permeability and a higher attenuation capacity against heavy metals and other pollutants. Sites operating in clay and clay loams, for instance, have operated successfully with as little as 1–2 m (3–6 ft) of soil separating sludge deposits from the highest groundwater table. In permeable soils, a leachate collection system must be installed.

Highly organic (peaty) soils and formations with faults, major fractures, and joint sets are not recommended for sludge landfilling. Impervious liners with a leachate collection and treatment system must be used in such situations.

Groundwater. The information concerning depth of groundwater (high and low water tables), the hydraulic gradient, the quality of groundwater, current and projected water use, and location of primary recharge zones should be obtained as part of the site selection process.

Site Access. The existing and future access roads should be accessible in all weather conditions. The probable impacts of traffic on the neighborhoods should be fully evaluated.

Ecology. Sludge landfills should not be located in environmentally sensitive areas such as wetlands, flood plains, permafrost, critical habitats of endangered species, and recharge zones of sole source aquifers. Vegetation at the site should also be considered. Vegetation can serve as a buffer and reduce dust, noise, odor, and visibility.

Land Use. The current and future land use patterns of the site should be fully studied. The state, regional, county, and local planning agencies and zoning authorities should be consulted for site evaluation and selection.

Social and Political Science. Waste disposal sites have serious social and political implications. The effect of disposal sites on the neighborhood and the extent of public resistance or acceptance should be fully evaluated. These issues will certainly come up during public participation programs discussed in Chapter 6.

The archaeological or historical significance of the land involved in the potential site should be ascertained. These issues are generally evaluated in the environmental impact assessment report (see Chapters 2 and 6).

Economics. Early in the site selection process, the relative costs for different sites should be developed for comparison purposes. These costs involve land acquisition, permanent structure, utilities, equipment purchase, and operation. A detailed discussion of economics may be found in Ref. 1.

19-5-4 Methods of Sludge Landfilling

The purpose of this section is to identify, describe, and develop design criteria for different methods of sludge landfilling. Common methods are (1) trenching (narrow and wide) and (2) area fill (mound, layer, and dike containment). Each of these methods is briefly discussed below. This discussion is adapted in part from Refs. 1, 2, 23, and 26.

Trenching. Stabilized or unstabilized sludge is placed in a trench and covered with soil. Trench operations are more specifically categorized as narrow and wide. Narrow trenches have widths less than 3 m (10 ft); wide trenches have widths greater than 3 m (10 ft). The width of the trench is determined by the solids content of the receiving sludge, its capability of supporting cover, the need for using the filled area for soil stockpiles, and the operating equipment and haul vehicles. Design considerations should include provisions to control leachate and gas migration, dust, vectors, and/or aesthetics. Leachate control measures include the maintenance of 0.6 to 1.5 m (2–5 ft) of soil thickness between trench bottom and highest groundwater level or bedrock (0.6 m for clay to 1.5 m for sand) or impervious liners and leachate collection and treatment system. Installation of gas control facilities may be necessary if buildings are nearby. These topics are covered in Sec. 19-5-5.

Narrow Trench. Sludge is placed in a single application, and then a single layer of cover soil is applied atop. Trenches are usually excavated by equipment based on solid ground adjacent to the trench. Backhoes, excavators, and trenching machines are particularly useful. The excavated material is usually immediately applied as cover over an adjacent sludge-filled trench. Sludge is placed in trenches either directly from haul vehicles through a chute extension or by pumping. The main advantage of a narrow trench (0.6–1 m) is its ability to handle sludge with a relatively low solids content (15–20 percent). Instead of sinking into the sludge, the cover soil bridges over the trench and receives support from undisturbed soil along each side of the trench. One to three meters width is more appropriate for sludge with a solids content of 20–28 percent, which is high enough to support cover soil. The basic design factors and trench operation are presented in Table 19-6 and Figure 19-2(a).

Wide Trench. Wide trenches are usually excavated by equipment operating inside the trench. Track loaders, draglines, scrapers, and track dozers are suitable equipment. The excavated material is stockpiled on solid ground adjacent to the trench for subsequent application as cover material. If the sludge is incapable of supporting equipment, cover is applied by equipment based on undisturbed ground adjacent to the trench. A front-end loader is suitable for trenches up to 3 m (10 ft); a dragline is suitable for trench widths up to 15 m (50 ft). If sludge can support equipment, a track dozer applies cover from within the trench. Sludge is placed in trenches by one of the following methods: haul vehicles directly entering the trench or haul vehicles dumping from the top of the trench. Dikes can be used to confine sludge to a specific area in a continuous trench. The basic design factors are given in Table 19-6. Wide-trench operation is shown in Figure 19-2(b).

Area Fill. Area fill is a sludge disposal operation in which sludge is placed above the original ground surface and subsequently covered with soil. To achieve stability and soil-bearing capacity, sludge is mixed with a bulking agent, usually dry soil. The soil absorbs excess moisture from the sludge and increases its workability. The large quantities of soil needed may require importing from elsewhere. Provisions must be made to keep the dry soil stockpiled. Installation of a liner is generally required for protection of groundwater, and provisions are made for surface drainage control, gas migration, dust, vectors, and/or aesthetics. Some of these topics are covered in Sec. 19-5-5. Area fills are more specifically categorized as follows.

Area Fill Mound. Sludge is mixed with a bulking agent, usually soil, and the mixture is hauled to the filling area where it is stacked in mounds approximately 2 m (6 ft) high. Cover material is then applied in a 1-m (3-ft) thickness. The cover thickness may be increased to 1.5 m (5 ft) if additional mounds are applied atop the first lift. The appropriate sludge/soil bulking ratio and soil cover thickness depend on the solids content of the sludge as received, the need for mound stability, and the bearing capacity as dictated by the number of lifts and equipment weight. Lightweight equipment with swamp pad tracks is appropriate for area fill mound operation; heavier wheel equipment is appropriate in transporting bulking material to and from stockpiles. Construction of earthen containments is useful to minimize mound slumping and for sloping sites. Design factors are summarized in Table 19-6 and Figure 19-3(a).

TABLE 19-6 Design Criteria for Sludge Landfills

Design Factors or Conditions	Trench		Area Fill		
	Narrow	Wide	Mound	Layer	Dike
Sludge solids, percent	15–28	20–28	⩾20	⩾15	20–28
Bulking ratio,[a] $\frac{\text{vol. of soil}}{\text{vol. of sludge}}$	—	—	0.5–1.00	0.25–1.00	0.25–0.50
Cover thickness[b]					
Intermediate, m	Not used	Not used	1	0.1–0.3	0.3–0.6
Final, m	0.6–1.2	1–1.5	1–1.5	0.6–1.2	1–1.2
Imported soil needed	No	No	Yes	Yes	Yes
Trench width, m[c]	0.6–3	>3	—	—	—
Sludge characteristics	Stabilized or unstabilized	Stabilized or unstabilized	Stabilized	Stabilized or unstabilized	Stabilized
Hydrology	Deep groundwater and bedrock	Deep groundwater and bedrock	Shallow groundwater and bedrock	Shallow groundwater and bedrock	Shallow groundwater and bedrock
Ground slope, percent	<20	<10	Suitable for steep terrain as long as level area is prepared for mounding	Suitable for medium slope but level ground preferred	Suitable for steep terrain as long as a level area is prepared inside dikes
Sludge application [in actual fill area, m^3/ha(yd^3/acre)]	2200–10,000 (1200–5600)	6000–27,400 (3200–14,500)	5500–27,000 (3000–14,000)	3500–17,000 (2000–9000)	9000–28,000 (4800–15,000)
Equipment	Backhoe with loader, excavator, trenching machine	Track loader, dragline, scraper, track dozer	Track loader, backhoe with loader, track dozer	Track dozer, grader, track loader	Dragline, track dozer, scraper

[a]More bulking agent is needed at lower sludge solids.
[b]Higher depths of intermediate and final covers are used at lower sludge solids.
[c]Narrower trench is used at lower sludge solids.
Source: Adapted in part from Ref. 1.

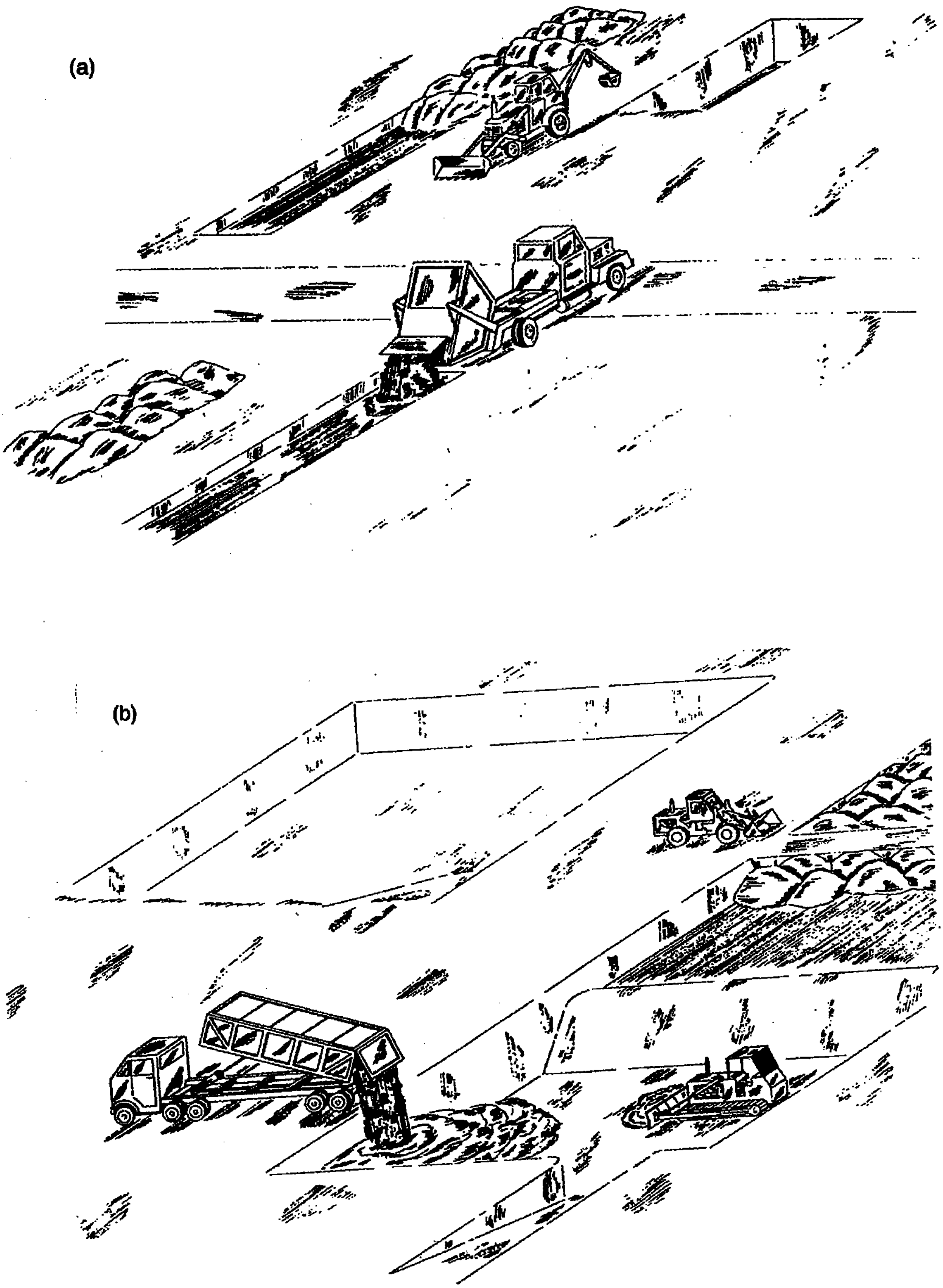

Figure 19-2 Operational Details of Sludge Disposal by Landfilling Using Trench Method: (a) narrow-trench landfilling operation and (b) wide-trench landfilling operation with interior dikes (from Ref. 1).

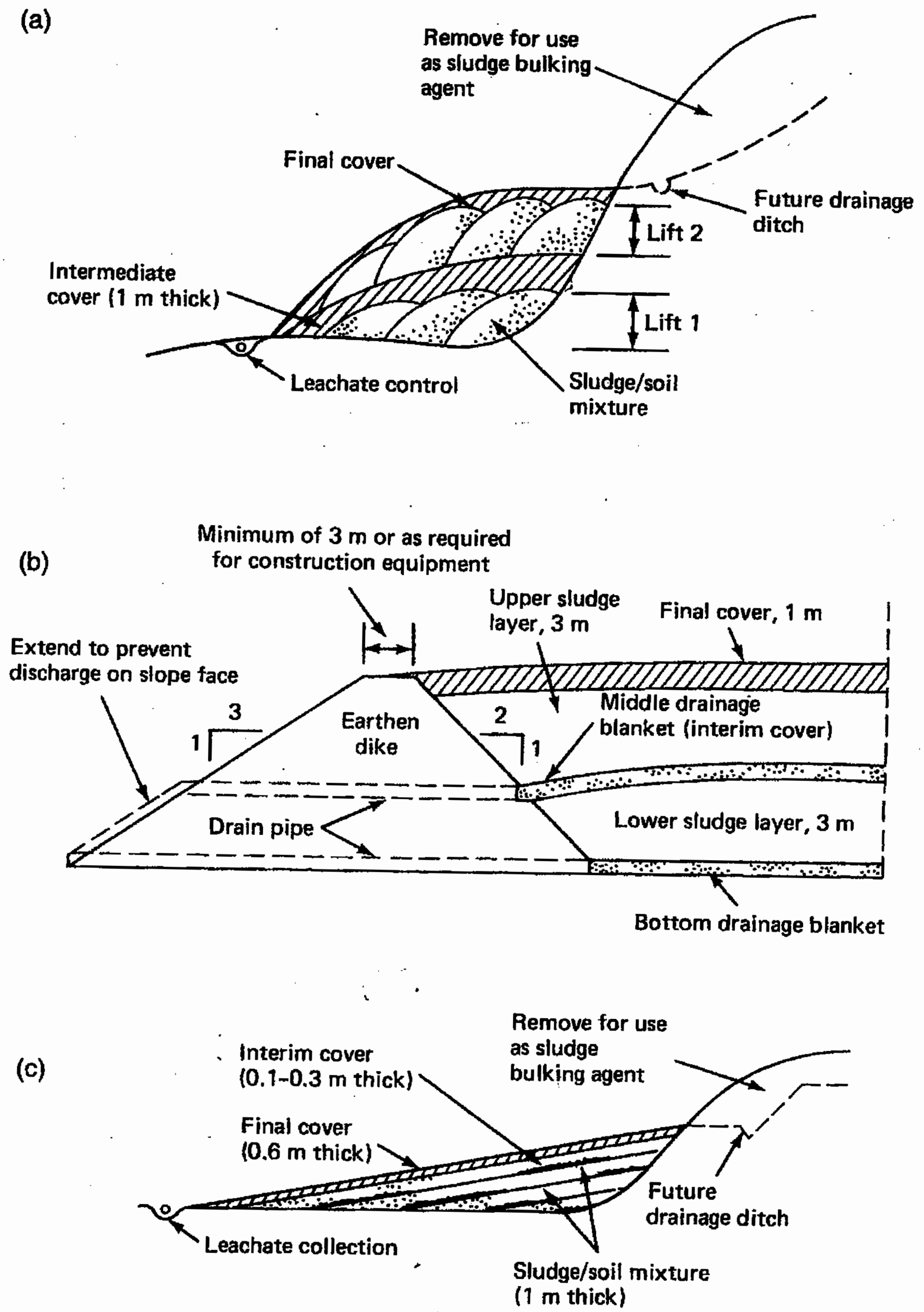

Figure 19-3 Design Details of Sludge Landfill by Area Fill Method: (a) cross section of typical area fill mound operation, (b) cross section of typical area fill layer operation, and (c) cross section of typical dike containment operation (from Ref. 1).

Area Fill Layer. Sludge is mixed with soil on- or off-site and spread evenly in consecutive layers 15 cm to 1 m (0.5–3 ft) thick. Interim cover between layers may be applied to 15- to 30-cm (0.5–1 ft) thick applications. Layering may continue to an indefinite height before final cover is applied. Lightweight equipment with swamp pad tracks is appropriate for area-fill layer operations; heavier wheel equipment is appropriate for hauling soil. Slopes should be relatively flat to prevent sludge from flowing downhill.

However, if sludge solids content is high or sufficient bulking soil is used, the effect can be prevented, and layering may be performed on mildly sloping terrain. Design factors are given in Table 19-6 and Figure 19-3(b).

Diked Containment. Dikes are constructed on level ground around all four sides of a containment area. Alternatively, the containment area may be placed at the toe of the hill so that the steep slope can be utilized as containment on one or two sides. Dikes would then be constructed around the remaining sides. Access is provided to the top of the dikes so that haul vehicles can dump sludge directly into the containment. A 0.3- to 1-m (1–3 ft) intermediate cover may be applied at certain points during the filling; a 1- to 2-m (3–5 ft) final cover should be applied when filling is discontinued. Cover material is applied either by a dragline based on solid ground, atop the dikes, or by track dozers directly on top of the sludge. Usually, operations are conducted without the addition of soil bulking agents, but occasionally soil bulking is added. Typical dimensions are 15–30 m (50–100 ft) wide, 30–60 m (100–200 ft) long, and 3–10 m (10–30 ft) deep. Design parameters are presented in Table 19-6 and Figure 19-3(c).

19-5-5 Design Considerations

The sludge landfills should be designed to ensure adequate protection of the environment and cost-effective utilization of site, storage volume, equipment, and soil. The important environmental factors that should be considered in the design include (1) protection of groundwater and leachate control, (2) gas control, (3) stormwater management, and (4) other design factors (access road, odor, dust, vector control, and aesthetics). Each of these impacts and methods to minimize them are discussed below.

Protection of Groundwater and Leachate Control. Leachate is produced from the filled areas because of excess moisture in the sludge and rainwater entering the fill. Leachate may enter the water system, essentially through two pathways: (a) percolation of leachate laterally or vertically through soil into the groundwater and (b) runoff of leachate outcroppings into the surface water.

Leachates contain high concentrations of organic and inorganic pollutants that may cause serious damage to the water quality.[1,21,24-27] Basic design safeguards are as follows:

1. Slope the surface of the landfill or provide an impervious liner with a sand drainage layer so that the rainwater drains away from the filled area. The surface profile of a completed landfill is shown in Figure 19-4.
2. Divert storm water around the landfill.
3. The Subtitle D Regulations require installation of a composite liner system, leachate collection system, and cover system for containing the leachate. The liner design and construction are based on specific site conditions. The earlier *natural attenuation landfills* were designed on the concept that the leachate would be attenuated (purified) by the soil beneath the landfill and by the groundwater aquifer. This concept has been discarded by most designers because serious groundwater contamination problems

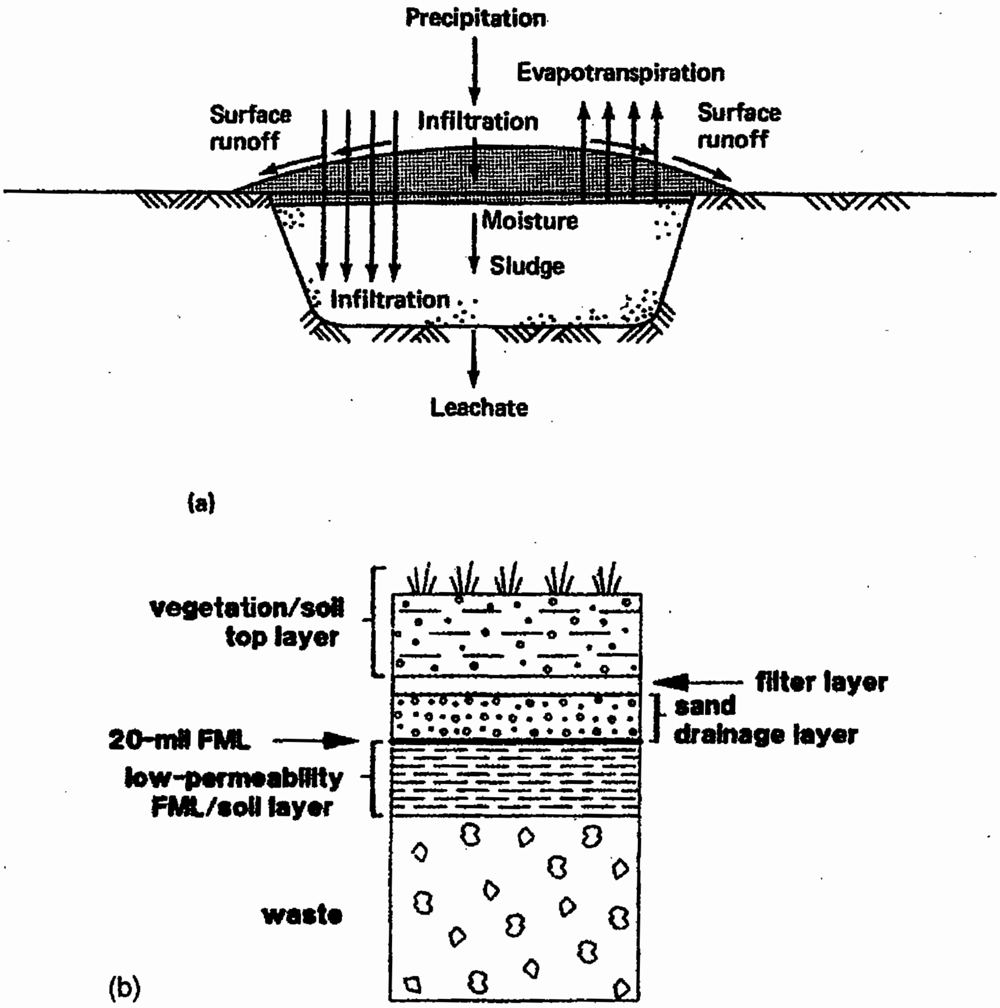

Figure 19-4 Percolation Control into a Landfill: (a) surface profile of a completed landfill with water balance (from Ref. 1) and (b) cover with flexible membrane liner (FML) and sand drainage layer (from Ref. 21).

developed, even many years after completion of landfilling. The new landfill design concept is based on *containment landfills*. These landfills may be single lined or double lined, depending upon the site-specific requirements. These requirements do not apply to existing units. The final design criteria provide owners and operators with two basic design options: (1) a site-specific, single-lined landfill design that meets the performance standards and is approved by the state or (2) a composite liner design. These two design options are shown in Figure 19-5. The single-lined landfills are constructed using either clay or a flexible membrane liner (FML) made of synthetic material [Figure 19-5(a) and (b)]. The composite or lined landfill designs may be constructed from one FML and one clay liner or both FMLs. There may be one leachate or two leachate collection systems. The lower leachate collection (or detection) system is not expected to collect leachate unless there is a leak in the liner or a driving head builts up over the upper liner. This may result from blocked or insufficient flow-carrying capacity of the upper leachate collection system. Several composite liner and multiple liner systems are shown in Figure 19-5(b) and (c). The liner for sludge land-

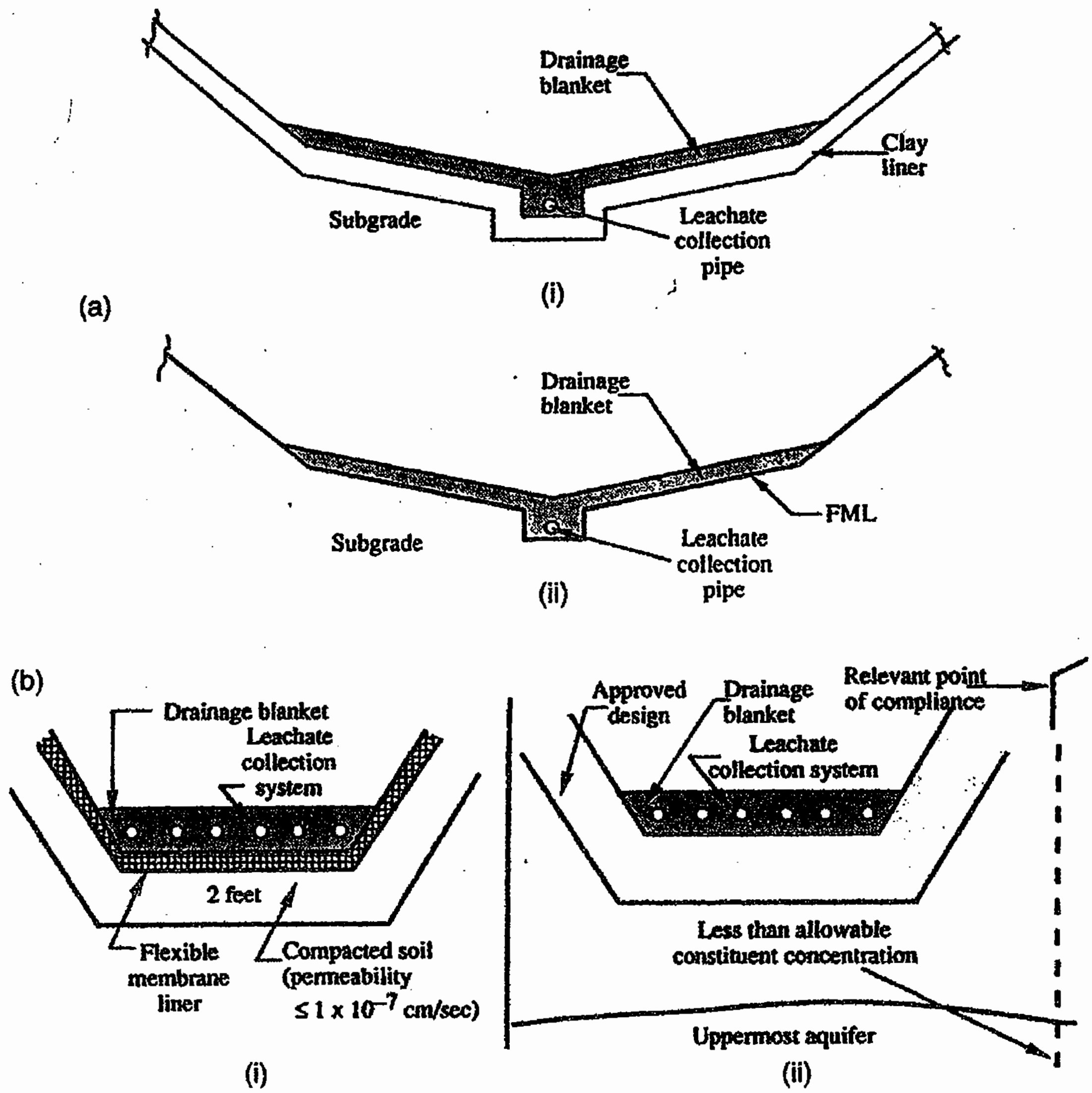

Figure 19-5 Details of Landfill Impervious Base Construction: (a) cross section of a single-lined landfill: (i) clay liner, (ii) flexible membrane liner (from Ref. 21); (b) design criteria for new municipal solid waste landfill (MSWLF) units and lateral expansions: (i) composite liner and leachate collection system, (ii) design that meets performance standard approved by state (from Ref. 18). *Continued*

fills may be of many different types, some of which are listed in Table 19-7.[6,21,27-30] Methods of treatment of leachate include pumping to existing sewers, evaporation, recycling through the landfill, and on-site treatment.[6,21,31-34]

4. If unlined landfill is permitted, provide sufficient depth of suitable soil between the groundwater table and the bottom of the fill. Contaminants in the leachate can be attenuated when passing through the soils by physical-chemical and/or biological processes.[1] The cation exchange capacity of different soils is given in Table 19-5.

Gas Control. Gas is produced by the decomposition of organic matter in sludge. Generally, methane is 50–65 percent and carbon dioxide is 30–45 percent. The remaining portion includes nitrogen, oxygen, water vapors, ammonia, and hydrogen sulfide. The rate of gas generation depends on the characteristics of the sludge, physical and chemi-

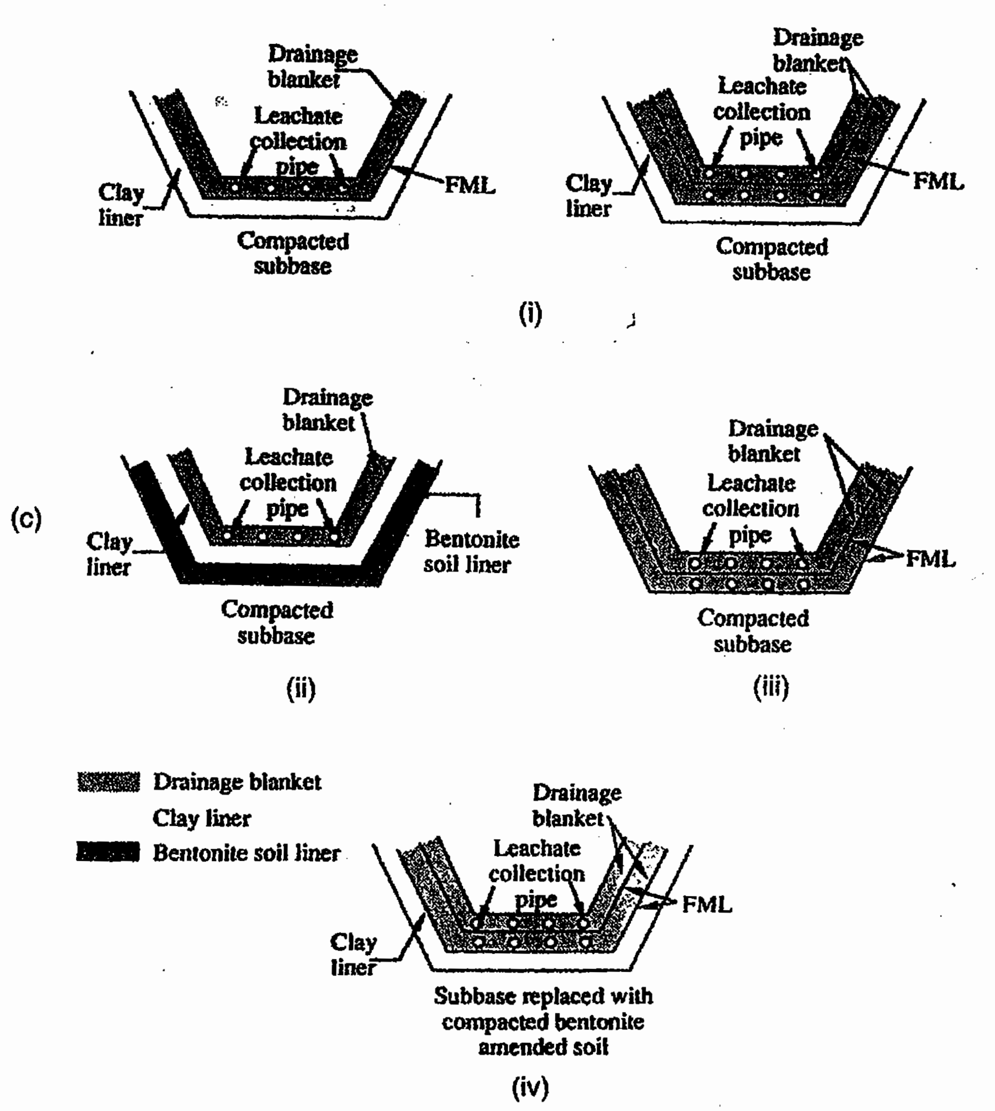

Figure 19-5—cont'd (c) composite liner with drainage pipes: (i) synthetic membrane liner above and clay liner below with one and two leachate collection systems, (ii) two clay liners, (iii) two synthetic liners, and (iv) multiple liner system.

cal conditions in the fill, and microbial activity. Undigested sludge produces gas 0.5–1.0 m^3/kg (8–16 ft^3/lb) of dry solids. Digested sludge produces much less gas. The gas is generated over an extended period.

Methane in air at 5–15 percent concentration constitutes an explosive mixture; therefore, its migration toward existing structures should be prevented. Gas control measures include (1) impermeable methods and (2) permeable methods. Both methods are shown in Figure 19-6. In recent years, there has been some effort to install gas recovery systems at large sanitary landfills.[1,35-37] The gas collection system may be *passive* or *active*. In a passive system the gas movement is by internal pressure, while in active systems the gas movement is by induced negative pressure or vacuum [Figure 19-6(a) and (f)].[6,21]

Storm Water Management. All upland drainage should be collected and directed around the landfill. Open earthen ditches, concrete- or stone-lined channels, and corru-

TABLE 19-7 *Classification of Liners for Landfill*

Type	Description
Natural soil and clay systems	Native clays, engineered soils formed on-site
Admixed liners	Formed on-site
Asphalt concrete	Hot mixture of asphalt cement and mineral aggregate
Soil cement	Mixture of portland cement, water, and selected soil
Soil asphalt	Mixture of soil and liquid asphalt
Sprayed-on linings Air-blown asphalt Emulsified asphalt Urethane-modified asphalt Rubber and plastic latexes	Liners are formed in the field by spraying the material on a prepared surface.
Soil sealants	Permeability of soils is reduced by application of sealants by spraying or pressure injection.
Flexible membrane liners (FMLs) or geomembranes Liners Butyl rubber Chlorinated polyethylene (CPE) Chlorosulfonated polyethylene (CSPE) Elasticized polyolefin (ELPO) Elasticized polyvinyl chloride (PVC-E) Epichlorohydrin rubber (CO, ECO) Ethylene propylene rubber (EPDM) Neoprene (chloroprene rubber—CR) Polyethylene (PE), High density polyethylene (HDPE), and very low density polyethylene (VLDPE) Polyvinyl chloride (PVC) Thermoplastic elastomer (TPE)	These are prefabricated liners made of polymeric material used to serve functions such as (1) liners, (2) geotextile, (3) geogrids, (4) geonets, and (5) geocomposites.
Geosynthetic clay liner	These liners are fabricated from sodium bentonite in between a woven and a nonwoven geotextile.
Geotextiles	These geomembranes are used to (1) separate layers of two different soils, (2) as filters over the drainage material, (3) as a cushion to protect synthetic liners, (4) drainage layer, and (5) reinforcement for soil veneer stability or slope stability.
Geogrids	Used for soil reinforcement on steep slopes
Geonets	Used in place of drainage layer
Geocomposites	Combination of synthetic materials. One composite drainage layer is made of geonet sandwich between geotextiles.

Source: Adapted in part from Refs. 21 and 27–30.

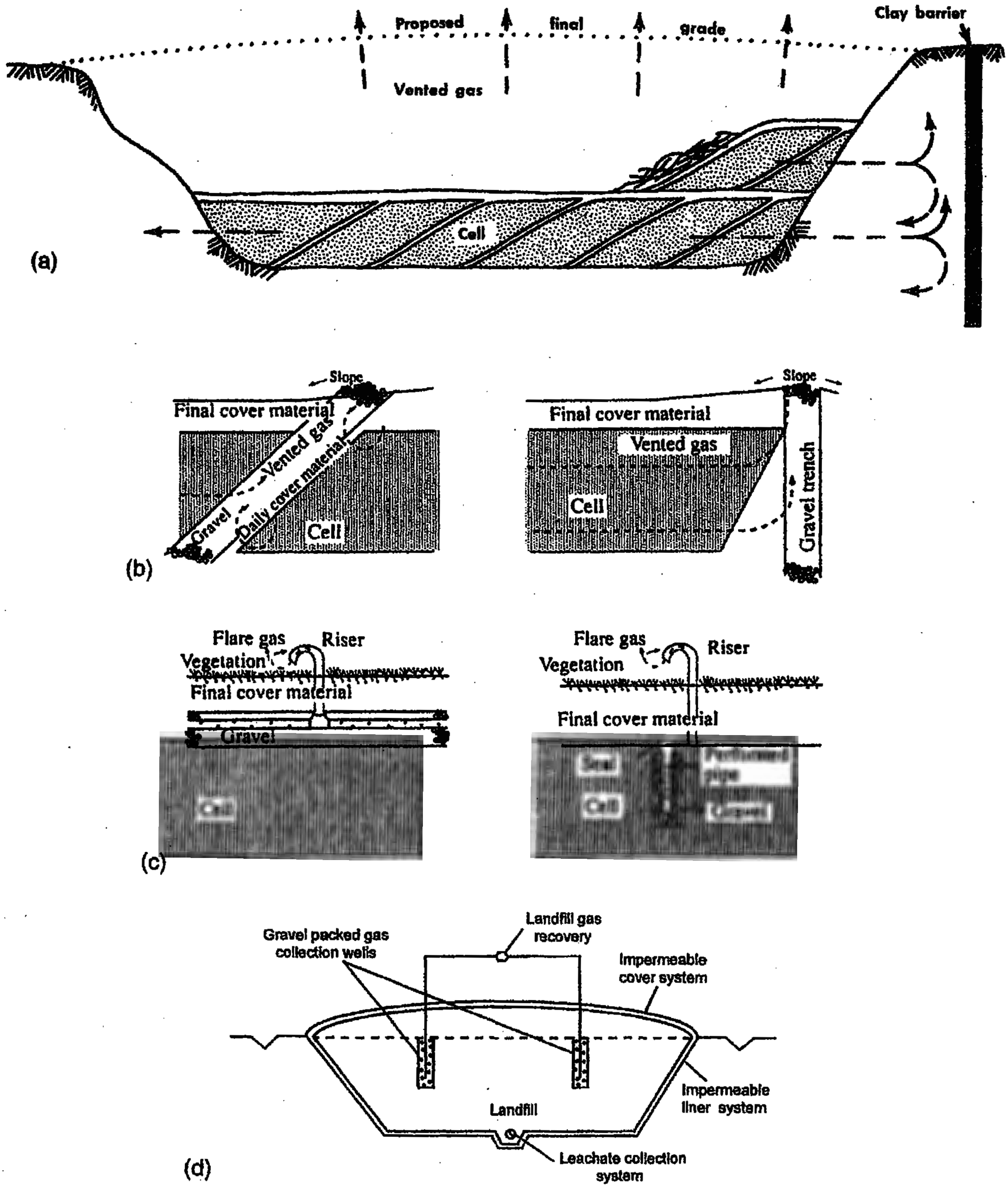

Figure 19-6 Gas Migration Control in Sludge Landfills: (a) impermeable method of gas migration control, (b) permeable method using gravel-filled trench inside the landfill and along the boundary, (c) gas venting from impervious cover and gravel-filled trench inside the landfill, (d) impermeable liner and cover and gas recovery components.

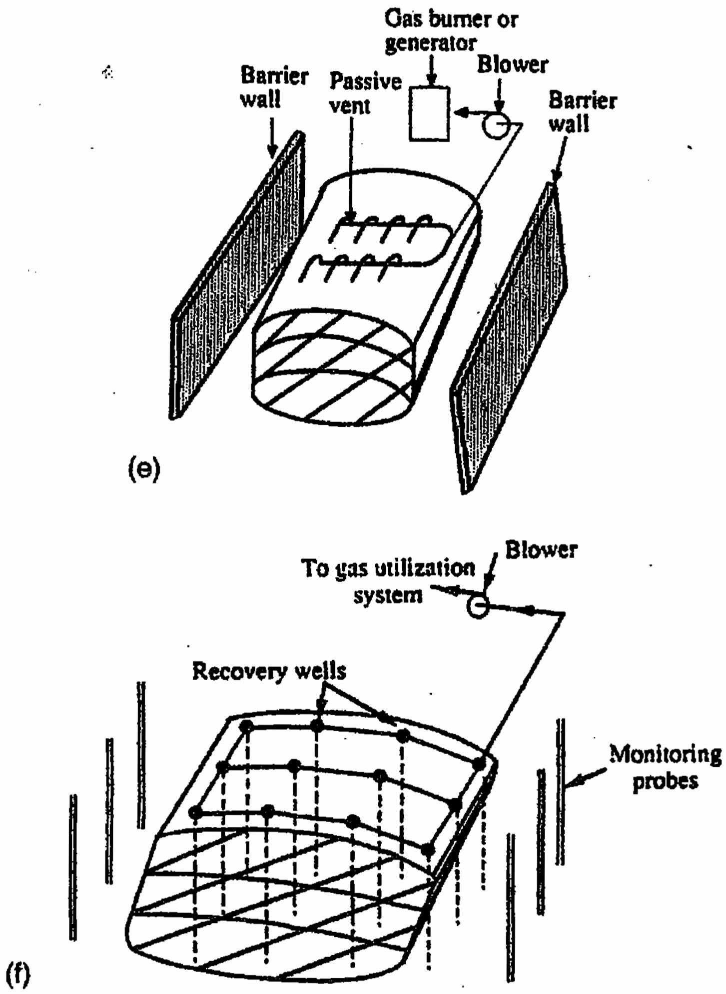

Figure 19-6—cont'd (e) passive gas recovery system, and (f) active gas recovery system.

gated metal pipe are used for drains. Various types of surface drains are shown in Figure 19-7.

The surface of the sludge landfill should be well graded to promote runoff and inhibit ponding. Slopes greater than 2 percent and less than 5 percent are recommended. Grades higher than 5 percent may encourage erosion. Siltation ponds are also used to settle solids.

Other Design Factors. Other important factors for efficient design and operation of sludge landfills include the following:

1. Provide all-weather access roads 6–7 m wide. To minimize dust, access roads should be gravel paved.
2. Consider on-site availability of soil and its use for bulking and covering. To minimize dust, all covered areas should be vegetated soon after completion. As an alternative, water should be applied to dusty roads.
3. Utilize available fill volume to the fullest extent.
4. Provide toilet facilities for employees.
5. Provide utilities, including water, sewer, electrical, and telephone.

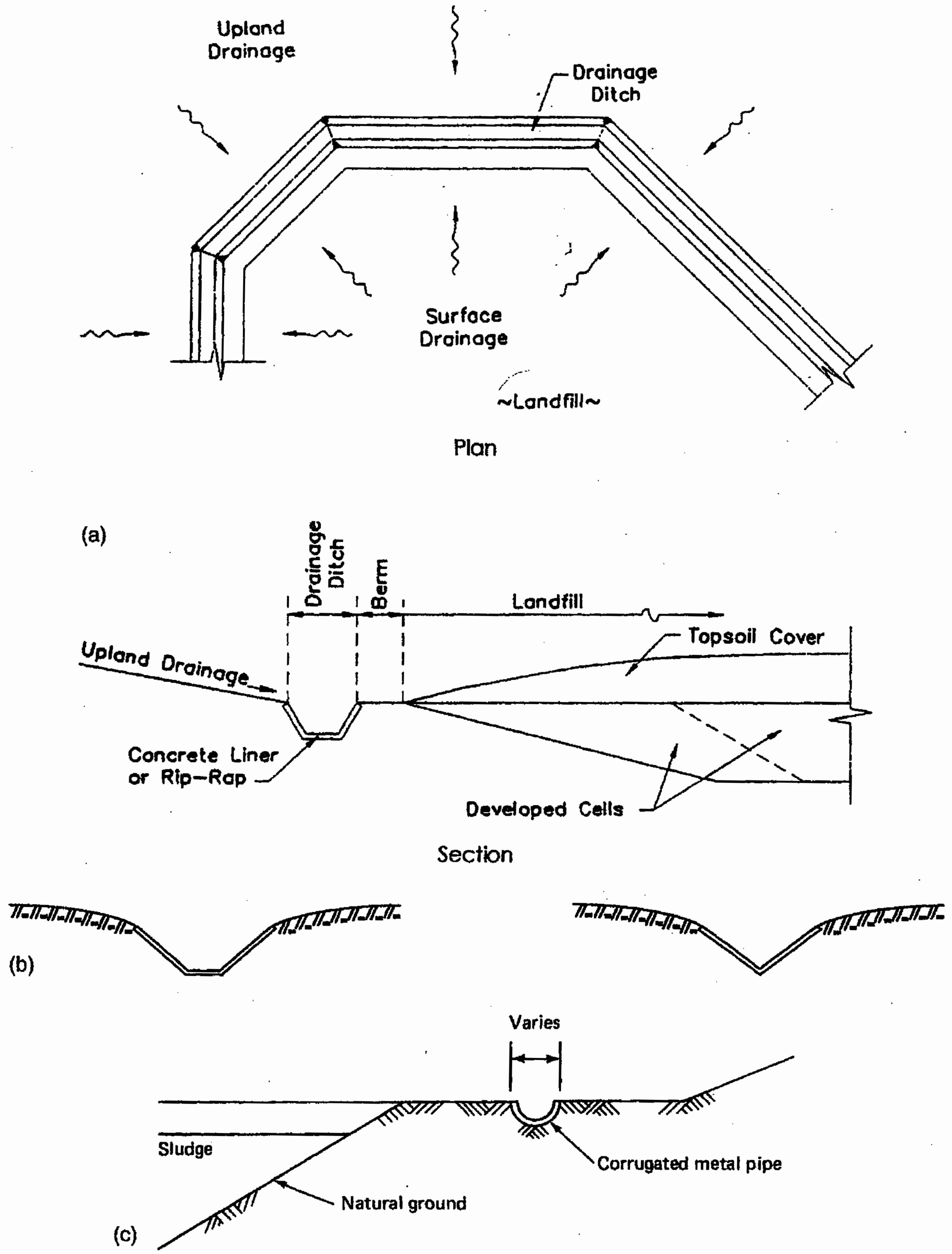

Figure 19-7 Various Types of Surface Drains: (a) surface diversion ditch, (b) trapezoidal and vee section-lined channels, (c) corrugated metal pipe channel.

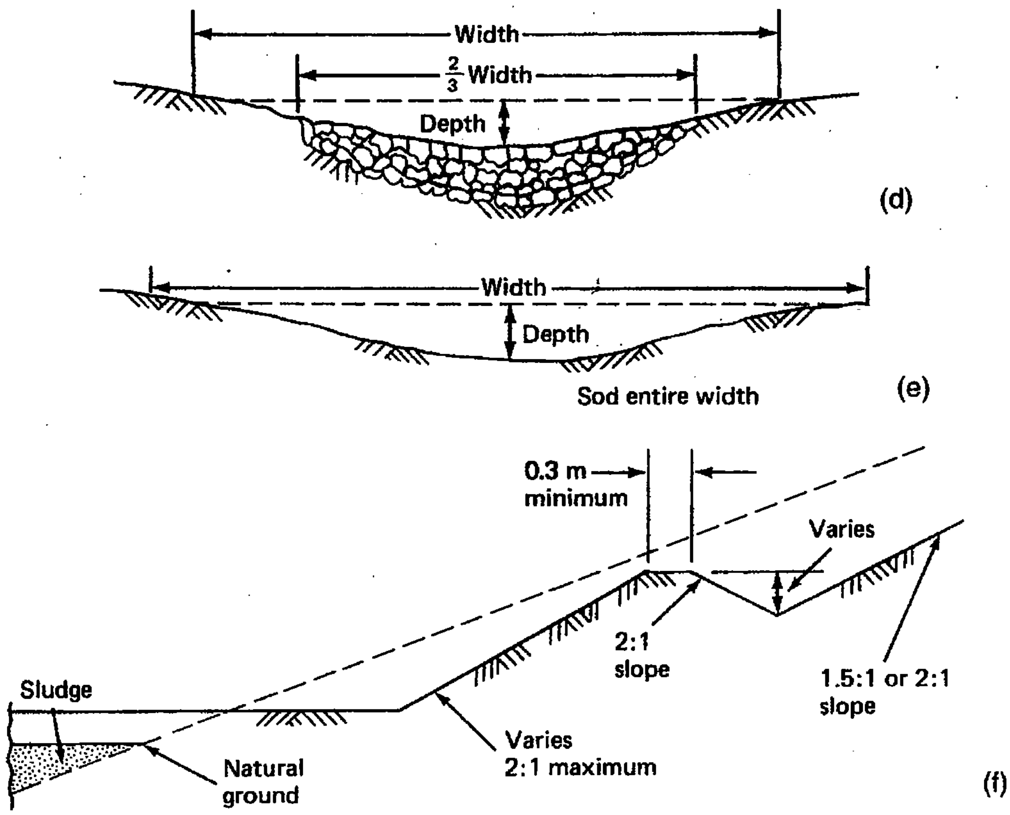

Figure 19-7—cont'd (d) stone-lined channel, (e) sodded earthen channel, and (f) earthen drainage channel on sloping embankment.

6. Provide fence.
7. Provide proper lighting.
8. Provide wash rack for equipment.
9. Do not leave uncovered sludge for extended time periods. Covering with well-compacted soil will reduce odors and control flies.

19-5-6 Operation and Maintenance

A sludge-landfilling operation is considered as an ongoing construction project that should implement the design plan. An effective operation requires a choice of equipment compatible with sludge characteristics, site conditions, landfilling method, operation plan, and contingency plans.

A sludge-landfilling operation is divided into two parts: (1) specific operational procedures and (2) general operational procedures. The specific operational procedures depend on the landfilling methods and include site and base preparation, sludge unloading, and sludge handling and covering. A typical equipment selection guide is summarized in Table 19-8.

The general operational procedures include (1) environmental control practices (spillage, erosion, dust, mud, vectors, odors, noise, aesthetics, health, and safety); (2) inclemental weather practices; (3) hours of operation; and (4) special wastes such as grit, screenings, ash, etc. A detailed discussion on operational procedures for different types of fills may be found in Ref. 1.

TABLE 19-8 Typical Equipment Selection Guide for Sludge Landfill

Sludge Cake Processed (m-ton/d)[a]	Trenching Machine	Backhoe with Loader	Excavator	Track Loader	Wheel Loader	Track Dozer	Scraper	Dragline	Total Equipment
Narrow trench									
up to 10		1							1
10–50		1				1[b]			2
50–100			1[b]			1			2
100–250	1	1			1				3
250–500	2	1			2[b]				5
Wide trench									
up to 10				1					1
10–50				1[b]		1[b]			2
50–100				1		1			2
100–250						1	1[b]		2
250–500				1[b]		2[b]	1		4

Area fill, mound							
up to 10		1					1
10–50	1[b]	1					2
50–100	1[b]	1		1[b]	1[b]		4
100–250	1[b]	1	1	1	1		5
250–500	1	1	1	1	1		5
Area fill, layer							
up to 10				1			1
10–50				1	1[b]		2
50–100				1	1[b]		2
100–250				2[b]	1[b]		3
250–500			1[b]	2	1		4
Area fill, dike							
up to 10				1			1
10–50				1[b]		1	2
50–100				1[b]	1[b]	1	3
100–250				1	1	1	3
250–500				2[b]	1	1	4

[a]mt/d = metric ton/d = 1000 kg/d.
[b]May not have 100 percent utilization.
Source: Adapted from Ref. 1.

19-5-7 Monitoring of Completed Landfills

The purpose of monitoring a completed landfill is to detect adverse environmental effects that may develop in the future. This includes monitoring (1) groundwater, (2) surface water, and (3) gas movement. Monitoring ground and surface water involves sample collection to establish baseline data, detection of contamination, and development of mitigation procedures. Common analytical parameters are pH, electrical conductivity or total dissolved solids (TDS), nitrate, chloride, total organic carbon (TOC) or COD, heavy metals (especially lead, cadmium, and mercury), and methylene-blue-active substances.

Gas monitoring is generally needed near the property line of the existing structures. The gas-sampling device generally consists of gas probes that are installed at various depths within a single hole, facilitating measurement of methane concentration.

For proper closure of the entire landfill or segment of the fill that has been filled to capacity, the following operational procedures must be used. These procedures can be conducted concurrent with ongoing site operation:

1. No sludge should be left exposed.
2. Maximum settlement will occur in the first year; therefore, the area should be regraded to ensure proper drainage. All depressions and cracks must be filled. The final cover shall not be less than the specified thickness.
3. Check the sediment and erosion controls, and modify the surface grade for best results.
4. Seed the area with the appropriate mixture of grasses.
5. Outline timetable to ensure that the following features are inspected on a regular basis: settlement, grading and vegetation cover, sediment and erosion control, leachate and gas control, monitoring of ground and surface water, fencing, and vandalism.

19-6 INFORMATION CHECKLIST FOR DESIGN OF SLUDGE DISPOSAL FACILITY

Design of a sludge disposal facility, whether land application or landfilling, consists of completing existing information and generating new information on the sludge and site conditions. Some of this information is collected during the site selection phase, some during the design of the other sludge-handling facilities, and some during the design phase of the sludge disposal system. Adherence to a carefully planned sequence of activities to develop the design of a sludge disposal facility minimizes project delays and expenditures. A checklist of design activities is presented below. These activities are listed in their general order of performance; however, in many cases separate tasks can and should be performed concurrently or even out of the order given below:[1]

1. Determine sludge volumes and characteristics for the initial and design years.
2. Determine the quantities and characteristics of other residues that must also be disposed of. Some of these residues are screenings, grits, skimmings, and ash.

3. Compile existing and generate new site information such as area, property boundaries, topography and slopes, surface water, utilities, roads, structure, and land use.
4. Compile hydrogeological information and prepare location map. The general information includes (a) groundwater hydrology (average depth and seasonal fluctuations, hydraulic grade, direction and rate of flow, water quality, and uses) and (b) soil data (depth, texture, structure, porosity, permeability, pH, cation exchange capacity, ease of excavation, and stability).
5. Compile climatological data (precipitation, evaporation, temperature, wind direction, number of freezing days, etc.).
6. Select land disposal options for ultimate disposal of sludge and other residues.
7. Identify federal, state, and local regulations and design standards. Most of these regulations cover the following items: requirements for sludge stabilization; sludge-loading rates; maximum allowable contaminant buildup on disposal site; surface runoff and leachate control; distance to residences, roads, surface, and groundwater; monitoring requirements; building codes; and so on.
8. Select ultimate disposal methods for sludge and other residues (screenings, grit, and scum). Obtain the values for the design parameters and operational features.
9. The complete design of the facility should include disposal or utilization plans; odors, leachate, surface runoff, and gas control; erosion control; access roads; special working areas, structures, and utility lines; lighting; wash racks; fencing; landscaping; monitoring wells; equipment; operation plans; and cost estimates.

19-7 DESIGN EXAMPLE

19-7-1 Design Criteria Used

The selected method for the ultimate disposal of sludge and other residues are (a) land application of digested and dewatered sludge cake (biosolids) over farmland and (b) land filling of residues such as screening, grits, and skimmings, and leftover or unused biosolids.

Step A: Land Application. The following design criteria are used for sludge application:

1. Determine sludge characteristics.

   The digested and dewatered sludge will be applied over farmland for crop production. The average quantity and quality of digested and dewatered sludge for this design were established based on an extensive sampling program. These results are summarized below. The combined sludge thickened, anaerobically digested, and dewatered has a musty odor that is not offensive. It must also meet the EPA class B pathogens requirements.

   Values obtained from Chapters 12, 13, 16–18:

   Sp. gr. 1.06, sludge solids 25% (see Chapters 12, 13, 16, 17, and 18)
   TSS and volume = 4778 kg/d and 18 m^3/d (based on average quantity)
   TVSS = 55 percent
   BOD_5 = 1516 kg/d or 32 percent

$$\begin{aligned} \text{Org.-N} &= 304 \text{ kg/d or } 6.4 \text{ percent} \\ NH_4^+\text{-N} &= 8.1 \text{ kg/d or } 0.2 \text{ percent} \\ NO_3^-\text{-N} &= 0 \\ P &= 190 \text{ kg/d or } 4.0 \text{ percent} \end{aligned}$$

Values obtained from extensive sampling and testing program:

K = 24 kg/d or 0.5 percent

Arsenic = 41 mg/kg   Cadmium = 35 mg/kg   Chromium = 1180 mg/kg
Copper = 1480 mg/kg   Lead = 218 mg/kg   Mercury = 14 mg/kg
Nickel = 380 mg/kg   Selenium = 35 mg/kg   Zinc = 1800 mg/kg

Approximately 21–24 percent of the digested sludge cake will be used on the city property, park areas, and highway medians. The remaining quantity will be used for land application. The expected average quantity of sludge for land application = 3700 kg/d dry solids, or 1350 mt[c]/yr.

2. Determine the soil characteristics.
   The soils in the farmland are generally sandy loam. The representative soil analyses are as follows:

$$\begin{aligned} \text{CEC} &= 18 \text{ meq/100g} \\ \text{Soil pH (in water)} &= 6.0 \\ \text{Available P} &= 17 \text{ kg/ha (15 lb/acre)} \\ \text{Available K} &= 84 \text{ kg/ha (75 lb/acre)} \\ \text{Lime needed to raise the pH to 6.5} &= 4.6 \text{ mt/ha (2.4 T/acre)} \end{aligned}$$

3. Select crops for production.
   The crops grown in the area include corn, soybeans, oats, wheat, barley, and forages for hay and pasture. The crop yield (mt/ha) and fertilizer requirements (N, P, and K) for each crop are different. This makes the calculations repetitive and complex. To simplify the Design Example, two crops with the same cropping pattern are considered. In a real-world situation, several crops are grown in different land areas and in different seasons.

Crop selected	Corn and grain sorghum (already grown in the area) have the same nutrient requirements and cropping patterns. Use of two crops will extend the cropping period, and calculations can be combined as one crop.
Crop yield	8.4–10.1 mt/ha
N requirement	190 kg/ha·yr
P requirement	45 kg/ha·yr
K requirement	140 kg/ha·yr

P and K recommendations are based on soil test data and cropping needs.

[c]mt = metric ton = 1000 kg; mt/ha × 0.445 = T/acre.

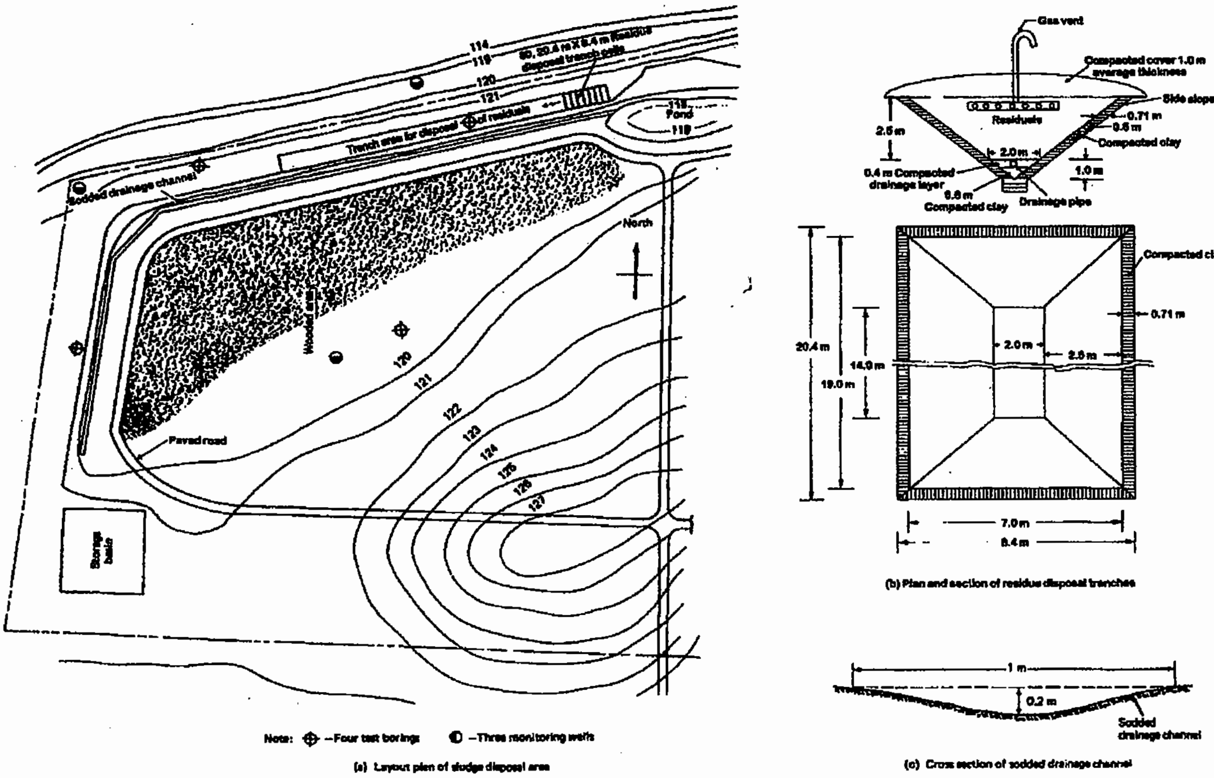

Figure 19-8 Layout Plan and Design Details of Trenches for Sludge Disposal.

4. Climate

   The climate of the area was briefly described in Sec. 6-5-2. Important climatological factors are listed below:

Average annual precipitation	= 89 cm (35 in.)
Average annual evaporation	= 76 cm (30 in.)
Number of days minimum temperature 0°C (32°F) and below	= 12 d/year

Step B: Landfilling. The following design criteria are used for the design of a landfill for disposal of residues:

1. Site location

   The sludge-landfilling site is a wooded area located on the northeastern side of the proposed site of the wastewater treatment plant. The sludge-landfilling area is flat. On the northern boundary is the flood protection levee enclosing the proposed wastewater treatment plant site. A plan view of the entire plant site is presented later in Figure 20-1. The proposed sludge landfill site is shown in Figure 19-8.
2. Soil conditions and hydrology

   Four test borings were performed to determine the soil condition and hydrology. The boring locations are shown in Figure 19-8, and the boring results are summarized in Table 19-9.

TABLE 19-9 Average Soil Condition and Hydrology

Condition	Description
Boring depth 0–6 m (0–19.5 ft)	Silt loam
6–9 m (19.5–29.5 ft)	Saturated silt loam
>9 m (29.5 ft)	Fractured crystalline rock
Depth of groundwater	6 m (19.5 ft)
Direction of flow	North
Analysis of silt loam provided the following data	
Texture	Medium
Permeability	2×10^{-4} cm/s (moderately slow)
pH	6.5
Cation exchange capacity (CEC)	18 meq/100 g

3. Type of wastes landfilled
   Disposal of various treatment plant residues will be achieved at the landfill. These residues are (a) screenings, (b) grits, (c) skimming, and (d) any leftover or unused anaerobically digested and dewatered sludge. The sludge will be placed in the fill, segregated from other residues. This will allow the sludge to be excavated in the future and used as topsoil, a soil conditioner, or biosolids.

TABLE 19-10 Characteristics of Wastewater Treatment Plant Residues Landfilled

Residues	Originating Processes	Conditioning Processes Used	Quality Description	Refs.
Screenings	Bar screens	None	Odorous and attracts flies. Moisture contents 80 percent. Unit weight 960 kg/m^3 (60 lb/ft^3)	Chapters 4 and 8
Grits	Aerated grit chamber	None	Odorous and attracts flies. Organic content 3–4 percent. Moisture content 20–30 percent. Unit weight 1600 kg/m^3 (100 lb/ft^3)	Chapters 4 and 11
Skimmings	Primary sedimentation and final clarifier	None	Highly odorous. Moisture content 40–75 percent. Specific gravity 1–0.95. Quantity of skimmings is 2–13 g/m^3 (16.8 to 110 lb/million gallon) of wastewater treated	Chapters 4 and 12

TABLE 19-11 Quantities of Wastewater Treatment Residues for Landfilling

Residue	Quantity of Residues (design year)	Ref.
Screenings, m^3/d	0.76	Chap. 8
Grit, m^3/d	1.14	Chap. 11
Skimmings, m^3/d	0.32	Chap. 12
Total, m^3/d	2.22	

The general characteristics of these residues are summarized in Table 19-10. The estimated qualities of these residues at the design year are presented in Table 19-11.

4. Landfilling method
   Select a narrow trench or cell landfilling method for segregated disposal of sludge from screenings, grit, and skimmings. This method is most appropriate because the site is flat, the groundwater table is deep, and an impervious liner is used.
5. Groundwater quality shall be monitored for possible pollution resulting from the landfill.

19-7-2 Design Calculations

Step A: Land Application. The land application of biosolids is utilized for crop production. Corn and grain sorghum are selected that have similar nutrient requirements and cropping patterns (see Design Criteria). The design calculations are combined as if only one crop is grown.

1. Calculate the quantity of sludge applied over land based on *cadmium* loading limitation.
   a. Determine the maximum amount of sludge applied over a lifetime [Eq. (19-1)].

$$S_m = \frac{L_m}{C_m \times 1000}$$

L_m = 39 kg/ha (Table 19-2)
C_m = 35 mg/kg or 0.000035 (see Design Criteria)

$$S_m = \frac{39 \text{ kg/ha}}{0.000035 \times 1000 \text{ kg/mt}} = 1114 \text{ mt/ha}$$

   b. Determine the annual limiting rate.

$$\text{Annual Cd loading rate} = \frac{1.9 \text{ kg/ha·yr (Table 19-2)}}{0.000035 \times 1000 \text{ kg/mt}}$$

$$= 54.3 \text{ mt/ha·yr}$$

2. Calculate the sludge application rate to satisfy the phosphorus requirement. If 50% P in sludge is available for crop production,

P available in sludge = fraction of P in sludge × 0.5

= 0.04 (see Design Criteria) × 0.5 × 1000 kg/mt
= 20 kg/mt

P requirement for corn = 45 kg/ha·yr

$$\text{The sludge application rate to meet P requirement} = \frac{45\ \text{kg/ha·yr}}{20\ \text{kg/mt}} = 2.3\ \text{mt/ha·yr}$$

3. Calculate the sludge application rate to satisfy the K requirement

K requirement for corn = 140 kg/ha·yr
K available in sludge = fraction of K in sludge × 1000 kg/mt
= 0.005 × 1000 kg/mt = 5 kg/mt
Sludge application rate to meet K requirement = (140 kg/ha·yr) × 1/5kg/mt

= 28 mt/ha·yr

4. Determine the sludge application rate to satisfy nitrogen requirement.
   a. Calculate the plant-available nitrogen N_p based on nitrogen values in the sludge [Eq. (19-2)].

NO_3^--N = 0, NH_4^+-N = 0.2%, Org.-N = 6.4%
For dewatered sludge $K_v = 1.0$ and mineralization factor $f = 0.2$ for first year.

$$N_p \text{ 1st yr} = 1000\ [(NO_3^-\text{-N} + K_v\ (NH_4^+\text{-N}) + f\ (\text{Org.-N})]$$
$$= 1000\ [(0 + 1.0(0.002) + 0.2\ (\text{Table 19-4}) \times 0.064]$$
$$= 14.8\ \text{kg N/mt}$$

   b. Calculate the fraction of organic nitrogen mineralized in subsequent years and plant-available nitrogen. Use Eq. (19-3). These values are summarized in Table 19-12.
   It is assumed that the steady-state value of plant-available nitrogen from the sludge is 30.8 kg N/mt dry solids.
   c. Calculate the nitrogen-limited sludge application rate. The nitrogen requirement for corn is 190 kg/ha·yr. Based on the steady-state available nitrogen from the sludge, the nitrogen-limited application rate is calculated.

$$\text{Sludge application rate} = \frac{190\ \text{kg N/ha·yr}}{30.8\ \text{kg N/mt dry solids}}$$
$$= 6.2\ \text{mt dry solids/ha·yr}$$

TABLE 19-12 Org.-N Mineralization and Plant-Available N

Year	K_t (kg/mt)	Org.-N Mineralized (kg/mt)	Plant-Available N (kg/mt)
0–1			14.8
1–2	0.8	5.1[a]	19.9[b]
2–3	0.36	2.3	22.2
3–4	0.21	1.4	23.6
4–5	0.20	1.3	24.9
5–6	0.19	1.2	26.1
6–7	0.19	1.2	27.3
7–8	0.18	1.2	28.5
8–9	0.18	1.2	29.7
9–10	0.17	1.1	30.8

[a]0.8 kg/mt × 6.4 = 5.1.
[b]14.8 kg/mt + 5.1 kg/mt = 19.9 kg/mt.

5. Select the sludge application rate.
   The sludge application rates for different constituents have been calculated above. These values are listed below.
   a. Limiting loading rate based on Cd = 54.3 mt dry solids/ha·yr
   b. The sludge application rate to satisfy the P requirement = 2.3 mt dry solids/ha·yr
   c. The sludge application rate to satisfy the K requirement = 28.0 mt dry solids/ha·yr
   d. Sludge application rate to satisfy the N requirement = 6.2 mt dry solids/ha·yr

   A comparison of different sludge loading rates with nitrogen value indicates the following:
   - A sludge application rate based on nitrogen will keep Cd loading well below the permissible limit.
   - P loading will exceed the phosphorus requirements for the corn and grain sorghum; as a result, excess P over the crop requirements would accumulate in the soil. The only adverse effects of excess P is the possible release into surface runoff.
   - K needs for the crop will not be met if the agronomic rate of nitrogen is applied. Supplemental chemical fertilizer would be needed to satisfy the K need for the crop.

   In most cases the design biosolids loading rate is generally based on the nitrogen requirement because excess nitrogen may cause migration of nitrate into the groundwater aquifer. Therefore, a biosolids application rate of 6.2 mt/ha·yr is selected for this situation.
6. Calculate the chemical fertilizer need for N, P, and K.
   A sludge application rate of 6.2 mt/ha·yr is based on N loading. The P loading will be in excess, while K loading will be less than the agronomic rates. Chemical fer-

tilizers will be needed to supplement the K deficiency. These values are summarized in Table 19-13.

7. Calculate the land area needed for cultivation.
The amount of sludge available for land application (design year) = 1350 mt dry solids/yr (Step A, 3)

$$\text{Land area needed} = \frac{1350 \text{ mt dry solids/yr}}{6.2 \text{ mt dry solids/ha}\cdot\text{yr}}$$

$$= 218 \text{ ha or } 220 \text{ ha } (544 \text{ acres})^{d}$$

Additional buffer land area will be needed.

8. Calculate the lime application rate.
The lime application rate determined by soil testing is 4.6 mt/ha as Ca $(OH)_2$ to maintain a pH of 6.5. This pH is necessary to reduce the uptake of heavy metals, particularly Cd, into the corn crop and possible entry into the food chain. Lime quantity will depend upon the land area used for cultivation.

$$\text{Quantity of lime } (Ca(OH)_2) \text{ needed} = 4.6 \text{ mt/ha} \times 220 \text{ ha} = 1012 \text{ mt}$$

9. Determine the sludge storage need.
Sludge storage is necessary to accommodate fluctuations in the sludge production rate, breakdowns in equipment, agriculture cropping patterns, and adverse weather conditions. Storage may be provided at the treatment plant, land application sites, or both. The storage volume also depends on the characteristics of the sludge.
In this design, provide 5-day storage at the treatment plant and 23-day storage at 5 distribution centers. The storage volume calculations are based on sludge volume. On average, the density of sludge is 1060 kg/m^3, and the quantity of sludge is 14 m^3/d.[e]

10. Determine sludge application scheduling.
The timing of sludge application must correspond to farming operations and is influenced by crop climate and soil properties. Sludge cannot be applied to soils that are frozen or covered with snow. After rains, muddy soils also make vehicle operation difficult, and surface runoff may occur. Applications must be scheduled around the tillage, planting, and harvesting operations for the crops grown.
Corn tillage, planting, growing, and harvesting periods in the general area of consideration are June through September. This is the period that sludge will not be applied over land. Individual states and local extension personnel can provide specific information on this subject.

11. Determine sludge transport equipment.
Land application of sludge involves transport, storage, application, and supporting facilities. Potential modes of sludge transportation include truck, pipeline, railroad, barge, or various combinations. Trucks are widely used for transporting both liquid

[d]ha × 2.4711 = acre.

[e]3700 kg/d (Step A, 1) $\times \frac{1}{0.25} \times \frac{1}{1060}$ kg/m^3 = 14 m^3/d.

TABLE 19-13 *The Sludge Application Rate and Nutrient Balance in the Crop Land*

Years Since Application Began	Biosolids Application Rate (mt/ha·yr)	Application Rate of N: Crop Need (kg/ha·yr)	Application Rate of N: Plant Available N (kg/mt)	Application Rate of N: N Provided by Biosolids Application (kg/ha·yr)	Application Rate of N: N Supplement from Chemical Fertilizer (kg/ha·yr)	Application Rate of P: Crop Need (kg/ha·yr)	Application Rate of P: Plant Available P (kg/ha·yr)	Application Rate of P: P Remaining in Crop (kg/ha·yr)	Application Rate of K: Crop Need (kg/ha·yr)	Application Rate of K: Plant Available K (kg/ha·yr)	Application Rate of K: K Supplement from Chemical Fertilizer (kg/ha·yr)
0–1	6.2[a]	190	14.8	91.8[b]	98.2[c]	45	124[d]	−79[e]	140	31[f]	109[g]
1–2	6.2	190	19.9	123.4	66.6	45	124	−79	140	31	109
2–3	6.2	190	22.2	137.6	52.4	45	124	−79	140	31	109
3–4	6.2	190	23.6	146.3	43.7	45	124	−79	140	31	109
4–5	6.2	190	24.9	154.4	35.6	45	124	−79	140	31	109
5–6	6.2	190	26.1	161.8	28.2	45	124	−79	140	31	109
6–7	6.2	190	27.3	169.3	20.7	45	124	−79	140	31	109
7–8	6.2	190	28.5	176.7	13.3	45	124	−79	140	31	109
8–9	6.2	190	29.7	184.1	5.9	45	124	−79	140	31	109
9–10	6.2	190	30.8	191.0	−1.0	45	124	−79	140	31	109

[a]Calculated in Step 5, d based on steady-state nitrogen availability to plant.
[b]14.8 kg N/mt dry solids × 6.2 mt dry solids/ha·yr = 91.8 kg/N/ha·yr.
[c]190 kg/ha·yr − 91.8 kg/ha·yr = 98.2 kg/ha·yr.
[d]20 kg/mt dry solids (Step A, 2) × 6.2 mt dry solids/ha·yr = 124 kg P/ha·yr.
[e]45 kg P/ha·yr − 124 kg P/ha·yr = −79 kg/ha·yr. The negative value indicates excess P over agronomic rates.
[f]6.2 mt dry solids/ha·yr × 5 kg K/mt (Step A, 3) = 31 kg K/ha·yr.
[g]140 kg K/ha·yr − 31 kg K/ha·yr = 109 kg/ha·yr.

and dewatered sludge, and this is the most flexible means of transportation. Liquid sludge is hauled in tractors and tank wagons. Dewatered sludge is hauled in dump trucks or hopper trucks. It is estimated that one 11-m^3 truck will be necessary to transport the required amount of dewatered sludge operating 8 h/d with a one-way haul distance of 24 km. Readers should refer to Ref. 12 for estimating the requirements and cost of sludge transport equipment.

12. Determine sludge application equipment.
    The sludge application equipment is different for liquid and dewatered sludges. Liquid sludge is surface spread with application vehicles equipped with splash poles, spray bars, or nozzles. Spreading of dewatered sludge is similar to the surface application of solid or semisolid fertilizers, lime, or animal manure. Spreading is done by box spreaders, bulldozers, loaders, or graders and then plowed or disked into the soil. The truck-mounted or tractor-powered box spreads are most commonly used.
13. Determine the monitoring requirements.
    Most regulatory agencies require monitoring of surface and groundwater. This topic is covered in Sec. 19-4-7.
14. Additional cropping patterns
    To simplify the design example, two crops of same cropping pattern are considered. In most situations, sludge is applied to more than one crop. It is suggested that application rate calculations be made for several crops grown when a detailed work plan is developed. For this Design Example, additional crops could be wheat, oats, barley, soybean, cotton, small grains, legumes, forages (alfalfa, clover, trefoil, etc.), grass (orchard grass, timothy, brome, reed canary grass, etc.), or a legume and grass mixture. Crop rotations are commonly used in many areas (e.g., corn–soybeans, soybeans–winter wheat, forage–corn and oats–forage).

Step B: Landfilling. The residues landfilled are screenings, grit, and skimmings. Leftover or unused sludge from land application is also landfilled.

1. Site development
   The landfill site is developed in accordance with the plan shown in Figure 19-8. Features of this plan include the following:
   a. Leave the maximum possible width of wooded buffer zone between the landfill and the treatment plant.
   b. Provide the sludge trenches parallel to the flood protection levee on the northern end of the site.
   c. The excavation depth of the trench or cells is determined initially by the groundwater table. If 3.5-m (11.5-ft) deep cells are excavated, a separation depth of the soil from the water table shall be 2.5 m (8.2 ft). At CEC of 18 meq/100 g and permeability classified as "moderately slow," the containment and attenuation of contaminants is good, but the requirement of the regulatory agency is to provide at least 0.5 m compacted clay on the bottom and sides, a minimum of 0.4-m drainage layer at the base, and a central pipe for leachate collection. Therefore a design similar to that shown in Figure 19-8(b) shall be utilized.

d. Provide narrow trenches or cells. Each cell shall have top dimensions of 20.4 m (67 ft) long and 8.4 m (28 ft) wide. The bottom dimensions shall be 14 m × 2 m (46 ft × 6 ft). The side slope of the trench cell shall be 1:1. The cell length shall be constructed perpendicular to the flood protection levee, with a clear spacing of 3 m between each cell. The total number of cells developed is 80. The cell arrangement is shown in Figure 19-8(a).
e. In each cell, the screenings, grit, and skimmings shall be placed on the north side. The sludge, if any, shall be compacted on the south side of the cell until the existing ground level is reached and the cell is full. A soil cover compacted to a 1-m depth shall be placed over the top of the residues, thus bringing the finished grade equal to that of the flood protection levee.
f. Provide a sodded surface water drain along the side of the cells [Figure 19-8(c)]. This drain shall carry the surface runoff to a sedimentation pond. The ground surface profile and the drainage pattern is shown in Figure 19-8(a).
g. In accordance with state regulations, one groundwater-monitoring well shall be located up gradient from the fill area. The monitoring wells are shown in Figure 19-8(a).
h. The access roads to the sludge landfill shall be paved with asphalt. Site arrangement is shown in Figure 19-8(a).

2. Life of the fill
   a. Compute the total volume available for residue disposal.
   Cell dimensions

Top length	= 20.4 m
Top width	= 8.4 m
Average depth	= 3.5 m
Side slope	= 1:1
Cross section of the cell	= triangular
Compacted clay layer at the base	= 0.6 m
Compacted drainage layer	= 0.4 m

The horizontal projection of compacted clay on the slope 0.71 m.

$$\text{Total available volume for each cell} = \frac{1}{3} \times \text{depth} \times [A_1 + A_2 + \sqrt{A_1 \times A_2}]$$

where A_1 and A_2 are bottom and top areas of the trench available for residue disposal.

$$\begin{aligned}\text{Total available volume (Figure 19-8)} &= \frac{1}{3} \times 2.5\text{ m}(2\text{ m} \times 14\text{ m} + 7\text{ m} \times 19\text{ m} \\ &\quad + \sqrt{(2\text{m} \times 14\text{m})(7\text{m} \times 19\text{m})} \\ &= \frac{2.5\text{m}}{3}(28\text{ m}^2 + 133\text{ m}^2 + \sqrt{28 \times 133\text{ m}^4} \\ &= 185\text{ m}^3\end{aligned}$$

If 20 percent volume reduction in the cell is assumed because of intermediate cover and bulking material, the total available volume of each cell = 148 m^3.
Total number of cells that will be developed over the property = 80
Total volume available for residue disposal = 80 × 148 m^3 = 11840 m^3

b. Volume occupied by residues (Table 19-11) = 2.22 $m^3/d \times 365$ d/yr = 810.3 m^3/yr
c. Total life of the fill = 11840 m^3/810.3 m^3/yr = 14.6 years
The 14.6 years of life is based on the quantity of residues produced at the design year. The unused biosolids disposal is not included. Initially, the quantity of residues produced will be less than that at the design years and will gradually increase over time. The difference in capacity will be sufficient to dispose of unused sludge. There is sufficient land available along the flood protection levees for additional land filling in the future.

3. Surface water drain
A sodded collection ditch shall be provided along the length of the trenches to intercept the uphill drainage. The drain shall be sloped toward the sedimentation pond. The cross section and alignment of the drainage ditch are shown in Figure 19-8.
4. Equipment requirements
The equipment need is determined from Table 19-8.
a. Estimate the quantity of residues landfilled per day= 2.22 m^3/d

Using a specific gravity of 1.20, total quantity per day[f]

$$= 2.22 \text{ m}^3\text{/d} \times 1.20 \times 1 \text{ mt/m}^3$$

$$= 2.66 \text{ mt/d}$$

Since the sludge disposal operation will be 5 days per week,

$$\text{Quantity handled per working day} = \frac{2.66 \text{ mt/d} \times 7 \text{ d/week}}{5 \text{ working d/week}}$$

$$= 3.7 \text{ mt/working d}$$

b. Determine the equipment need from Table 19-8.
Provide one backhoe with loader. Because of small quantities of residues that will be handled each day, this equipment will not be fully utilized. This equipment will be utilized to provide other support services such as grading and maintenance of embankment, landscaping, sedimentation pond, surface drain, landfill cover, and others.

5. Sedimentation pond and effluent structure
Provide a 1.2-ha (2.9-acre) sedimentation pond on the southeastern side of the sludge landfill area. This is the lowest area, and therefore, the surface runoff from a large portion of the site will drain to the pond. The depth of the pond is 2.0 m below the ground surface. The area surrounding the pond shall be graded to permit natural drainage to the pond. The excavated soil will be utilized in the construction of the flood protection levee.

[f]Combined sp. gr. of grit, screenings, and scum.

The stormwater structure at the pond consists of an effluent box, effluent baffles, and an outlet pipe to drain the effluent into a sump. From the sump, the effluent will be discharged by gravity into the river under low flow situations. A flood gate in the sump and stormwater pumps are provided to pump the surface runoff under flood conditions.

19-8 OPERATION AND MAINTENANCE AND SPECIFICATIONS

19-8-1 Land Application

Many state regulatory agencies require that the responsibilities of all parties for implementation of all essential components of operation, management, and maintenance program for land application systems must be clearly defined. Essential elements are briefly presented below:

1. Operations at the POTW must ensure that the treated sludge is adequately stabilized and monitored to meet the requirements for land application.
2. Flexible scheduling of sludge transport, storage, and application activities to allow for both the need of the POTW to remove sludge and the ability to apply the sludge to the land application site must be maintained.
3. Design, operation, management, and maintenance of the sludge transport system to minimize potential nuisance and health problems should be monitored. Included should be an in-place procedure for rapid response to accidents, spills, and other emergency conditions arising during routine sludge transport operations.
4. Design, operation, management, and maintenance of the sludge application site(s) and equipment to minimize potential nuisance and health problems should be closely coordinated.
5. Monitoring reporting and recordkeeping of sludge generation and analyses of sludge, soil, plant, surface water, and groundwater as needed for compliance must be conducted and maintained. It must meet the EPA Class B pathogens requirements.
6. Continuous efforts should be made to avoid or reduce nuisance problems associated with sludge hauling, application, and related operations. Potential nuisances of concern include noise, odor, spillage, mud, and dust.

19-8-2 Landfilling

The landfill of screenings, grit, and skimmings and any leftover sludge shall be performed at the plant property. The essential elements are as follows:

1. An asphalt-paved road shall be constructed to provide access to the entire sludge-landfilling area. Runoff, erosion, and sediment control, as well as monitoring wells, shall be installed. Initially, the area for cells nearest to the sedimentation pond shall be cleared and grubbed. The excavation for only one or two cells at a time shall begin and proceed generally toward the farthest end in accordance with the design dimensions. These cells shall be used for phase filling and covering. Each cell shall be ex-

cavated completely and the soil stockpiled for the filling operation. The clay for an impervious layer shall also be stockpiled.

2. Landfilling shall be performed by equipment operating outside the cell. Because the grit, screenings, and skimmings are highly odorous, a daily covering of such material is considered essential. After the placement of the impervious layer, drainage pipe, and drainage layer, the sludge shall be unloaded directly into the cell and spread out evenly. After the residue and sludge-filling areas reach the ground level, approximately 1 m of final cover shall be applied over the entire cell area. At least 1 week in advance of completion of each cell, a new cell shall be made ready for filling.

   Operation shall be conducted at the site 8 h/d and 5 d/week to coincide with residue deliveries and to avoid odors generally encountered with storage of sludge and other residues. The equipment shall be operated 7 h/d plus 1 h/d downtime for routine maintenance and cleanup.

3. Approximately 1 month after completion of each cell, the surface shall be graded to the smooth ground surface at an elevation of the flood protection levee. The surface shall be sloped inward to drain into the surface water drains. Immediately thereafter, the site shall be hydroseeded if weather conditions permit. Leftover soil shall be utilized in surface grading, as well as the construction of a levee and other embankments.
4. All surface water drains shall be sodded and kept clean from debris. All on-site drainage shall be channeled to the sedimentation pond.
5. During inclement weather, soil shall be stockpiled and covered with plastic sheets to keep dry and workable. The mud from the haul vehicles and equipment shall be washed in the wash rack area.
6. Background samples shall be collected from inspection wells as well as from existing wells and analyzed prior to initiating landfilling operation. This information shall be used to establish baseline data. Subsequently, samples from each well shall be collected at 6-month intervals and analyzed for the constituents discussed in Sec. 19-4-7.

19-9 PROBLEMS AND DISCUSSION TOPICS

19-1 Discuss the technical considerations that are involved in site evaluation and selection of agricultural land application of biosolids.

19-2 The cumulative Cd loading rate for agricultural land is 39 kg/ha. The average concentration of Cd in biosolids is 80 mg/kg. Calculate the maximum amount of dry biosolids that can be applied over the useful life of the site (mt/ha). If the biosolids application rate is 10 mt/dry solids/ha·yr and if there is no loss of Cd in the crop, how many years will it take to reach the limiting value?

19-3 Anaerobically digested and dewatered biosolids containing NH_4^+-N = 0.5%, NO_3^--N = 0.2%, and Org.-N = 4.8% is applied over a farm land. The solids application rate is 9 mt/ha·yr. Calculate the plant-available nitrogen in the fifth and sixth years. How much increase in plant-available N will occur between the fifth and sixth years?

19-4 A biosolids disposal project is using digested dewatered solids with a nutrient supplement. The nutrient contents in biosolids are NH_4^+-N = 0.8%, NO_3-N = 0.1%, Org.-N = 3.8%, P = 4.5%, and K = 0.4%. The nutrient requirements for the crop are N = 300 kg/ha·yr, P = 40 kg/ha·yr, and K = 180 kg/ha·yr. Estimate the quantities of nutrient supplements if the biosolids application rate is 10 mt/ha·yr.

19-5 Discuss the technical considerations that are involved in site selection of a sludge disposal landfill. How do they differ from those for site selection of a wastewater treatment plant, which is presented in Chapter 2?

19-6 Compare the trenching and area landfilling methods of sludge disposal. Describe the site conditions for which each method is most suitable.

19-7 Calculate the useful life of a site in which sludge will be landfilled by an area fill mound operation. The site is approximately 3 ha, and after removing the cover and bedding material, the average depth of the disposal site is 3.5 m. The finished grade must be kept in line with the ground surface above the depression. Approximately 27 mt of residues will be landfilled each day. The bedding material and intermediate covers constitute 18 percent volume. The top cover is 1.3 m. Assume that the specific gravity of the mixed residue is 1.2. Also determine the equipment needed.

19-8 List different methods of leachate and gas control in a sludge landfill.

19-9 Calculate the average quantity of digested sludge handled over 15 years in m-tons per working day. Use the solids data given in Sec. 19-7-1, Step A and flow data in Table 6-9 to compute the change in the quantity of sludge. The unit weight of sludge is 1.06 m-ton per m^3. Assume the operation of sludge disposal is 5 days per week.

19-10 Calculate the life of a landfill site in which sludge and screenings will be landfilled by a dike containment method. The total volume of the sludge and other residues is 3942 m^3/y. The dimensions of the dike containment area are length = 200 m; width = 60 m; maximum depth = 4.6 m; and ground after scraping of cover material is flat. An intermediate cover of 0.3 m is applied when a 1.5-m depth of sludge is reached. Final cover is 1 m. Volume occupied by bulking material is 12 percent. Draw the cross-sectional view of the completed fill.

19-11 Study Ref. 1 and discuss advantages and disadvantages of co-disposal of sludge and municipal solid wastes in sanitary landfills.

19-12 What are sludge conversion processes? Compare and list the advantages and disadvantages of each method.

REFERENCES

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20

Plant Layout

20-1 INTRODUCTION

Plant layout is the physical arrangement of designed treatment units on the selected site. Careful consideration must be given to properly locating the treatment units, connecting conduits, roads and parking facilities, administration building, and maintenance shops. The design engineer must integrate the functions of all components. However, it should be remembered that the factors influencing site design are both natural and social. Therefore, efforts should be made to become familiar with the proposed site and its neighborhood. Experience has shown that proper plant layout can (1) enhance the attractiveness of the plant site, (2) fit the operational needs of the processes, (3) suit the maintenance needs of the plant personnel, (4) minimize construction and operational costs, (5) offer flexibility in future process modifications and plant expansion, and (6) maintain the landscaping and plant structures in perfect harmony with the environment.[1-3]

In this chapter the basic considerations for plant layout are discussed. Also, the specific site plan for the plant in the Design Example has been developed.

20-2 FACTORS AFFECTING PLANT LAYOUT AND SITE DEVELOPMENT

A variety of factors should be considered as general design guidelines for plant layout on the selected site. Some of these factors are discussed below. Typical plant layout plans for medium and large wastewater treatment facilities are shown in Figure 20-1.

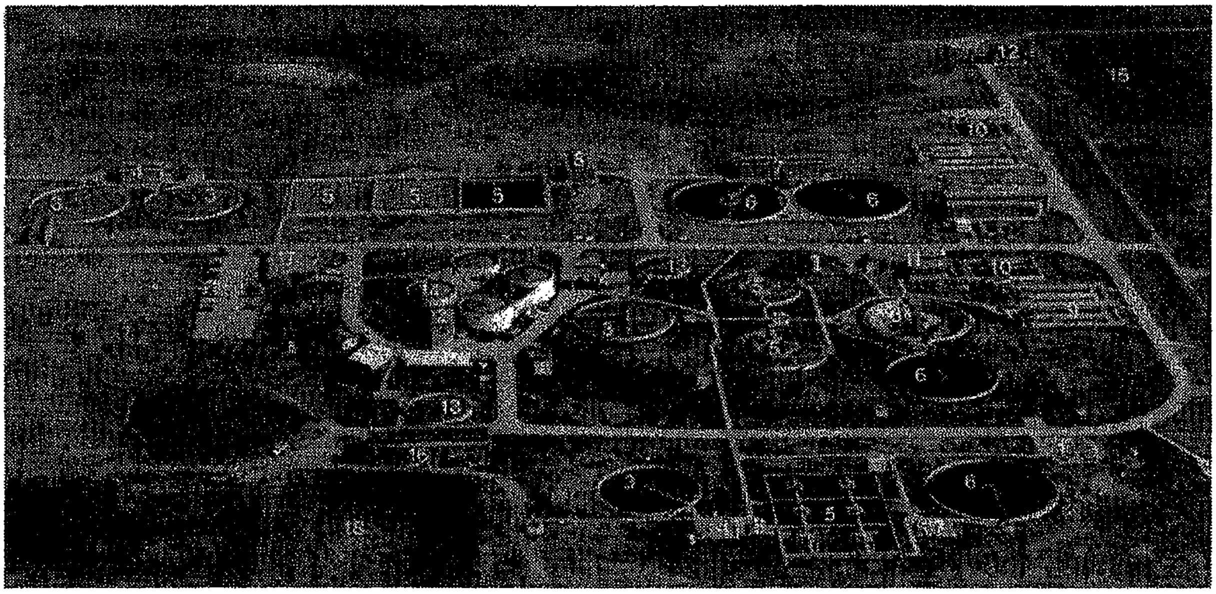
20-2-1 Unit Construction Considerations

Site Topography and Geology. Consideration of site topography and geology is important because plant layout should respect such existing site features as character, topography, and shoreline. Site development should take the advantage of the existing site topography to either emphasize or diminish the visual impact of the facility, depending on the design goals. The following principles are important to consider:

1. A site on a side-hill slope can facilitate gravity flow that will reduce pumping requirements and locate normal sequence of units without excessive excavation and fill.


(a)

Figure 20-1 Layout of Three Wastewater Treatment Plants: (a) Pecan Creek Wastewater Treatment Plant, Denton, Texas. Average design flow 0.53 m^3/s: bar screen/pump station, (2) flow meter, (3) grit chamber, (4) primary clarifier, (5) primary sludge and grit building, (6) aeration basins, (7) final clarifier, (8) blower building, (9) return sludge pump, (10) effluent filters, (11) chlorine contact basins, (12) chlorine storage, (13) effluent pumps, (14) gravity thickener, (15) dissolved air flotation, (16) sludge digester, (17) sludge-drying beds, (18) administration building (courtesy Freese and Nicohols, Inc.); (b) Ten Miles Creek Regional Wastewater Treatment Plant, (Trinity River Authority of Texas). Average design capacity 0.93 m^3/s: (1) pump station/bar screen, (2) grit removal building, (3) primary clarifier, (4) primary sludge pump station, (5) aeration basin, (6) final clarifier, (7) return sludge pump station, (8) blower building, (9) effluent filters, (10) chlorine contract basins, (11) chlorine building, (12) dechlorination building, (13) gravity thickener, (14) anaerobic sludge digesters, (15) sludge-drying lagoons, (16) maintenance building, (17) administration building, (18) emergency storage basin (courtesy Chiang, Patel and Yerby, Inc.); (c) Red Oak Creek Regional Wastewater Treatment Plant (Trinity River Authority of Texas). A compact plan design. Average design flow 0.15 m^3/s: (1) lift station, valves, and motor control center, (2) grinder, scum box, and aerated grit chamber, (3) under the roof, blowers, primary sludge, scum and grit pumps, (4) primary clarifier, (5) division box, (6) aeration basin, (7) under the roof, RAS, WAS, scum, and sludge pumps, (8) final clarifier, (9) chlorine contact basins, (10) chlorine building, (11) aerobic sludge digester, (12) sludge-drying beds, (13) outfall structure, (14) administration building (courtesy Chiang, Patel and Yerby, Inc.).



(b)

(c)

Figure 20-1—cont'd

2. A side-hill location can be used to advantage because the sludge-handling building can provide direct access to trucks from the ground and upper levels. Also, a side-hill location can be used to advantage because the chemical storage silos can have direct access from the ground and upper levels.
3. When landscaping is utilized, it should reflect the character of the surrounding area. Site development should alter existing, naturally stabilized site contours and drainage patterns as little as possible. Consideration to limit erosion and siltation must also be given.
4. The developed site should be compatible with the existing land uses and the comprehensive development plan. In some instances, it may be desirable to design the various buildings at a treatment plant to blend the treatment facility with the surrounding developments.

Foundation Considerations. The results of soil investigation should be utilized in locating the treatment units, buildings, and heavy equipment. Consideration should be given to load-bearing capacities, water table and flotation effects, and piling. Separation of massive concrete structures (such as basins, filters, and operations buildings) from other facilities is necessary to minimize uneven subsidence of the structure.

Access Roads. Access roads should be included in plant layout. Access roads should be provided to serve equipment in process areas. Employee parking at the rear and visitors parking in the front of the administration building should be provided. Trucks and service traffic should be separated from visitors and employee traffic early upon site entry. Signs for visitor parking and appropriate building entry should be posted.

Odor and Aerosol Sources. Processes causing potential odors or aerosol should be located downwind from public spaces or developments near the site. Such units should be located near each other and should be covered if necessary. Protective barriers, such as heavy plantation of large trees and buffer land, should also be considered.

Noise Sources. Noise should be controlled to prevent discomfort to plant personnel and neighbors. Equipment such as pumps, ejectors, generators, blowers, and compressors can produce disturbing sound levels and therefore should be isolated. Where necessary, noise sources should be provided with sound barriers, absorptive enclosures, etc. Vehicular noise should also be considered in plant layout.

20-2-2 Compact and Modular Site Development

A relatively compact site plan can minimize piping requirements. Centralization of similar process units, process equipment, personnel, and facilities may reduce total staff size, as well as optimize plant supervision and operation features. The traditional linear plant layout is not ideal for chemical feed or operation because of long distances. A compact plant layout clusters chemical and mechanical equipment. This concept offers many advantages over traditional design using linear layout. The advantages and basic design considerations of a compact layout are summarized in Table 20-1. Many multistory, underground, and totally covered treatment facilities have been designed and built in the past. Many of these facilities are in Japan, Sweden, the United States, and other countries. Readers may find information on such compact designs in Refs. 4–14. It is expected that in the future many more multistory, underground, and compact wastewater treatment facilities will be designed and built.

Many important design considerations include the following:

- Modular expansion of a compact plant layout is more suitable. The designer should make provisions not only for the expected future capacity, but should also allow for additional treatment units that may be required to meet more stringent future regulations.
- Equipment access in a compact design may be restricted. Trucks may not be able to get as close to the process equipment as they might with a dispersed plant layout. Provisions for monorails, bridge crane, access openings, and other appropriate measures should be provided to allow for removal and replacement of equipment. Adequate clearance should be provided around all items of mechanical equipment for maintenance and disassembly.
- In compact design and layout, the elevations of most treatment units, chemical storage and feeders, and floors of buildings become interrelated. Raising or lowering the elevation of one basin may affect the elevations of other units and building floor. Careful consideration should be given to such details at an early stage of the design.
- Safety aspects need more attention with the compact design because of the proximity of plant staff to storage, handling, and delivery of hazardous chemicals. Appropriate articles of the Uniform Fire Code (UFC)[14] dealing with hazardous materials used in wastewater treatment plants should be studied. Minimum distances from storage tank to building, openings in buildings, and air intakes are important design considerations. Other relevant authorities about the governing codes and local regulations should be checked at an early stage of the design. Related safety issues and compliance with the relevant building codes for fire ratings, access-egress, enclosures of stairways, and the like must be met. Combining administration and operation into a single structure results in a building with mixed occupancy classifications. Under such conditions, it can be difficult to meet all the requirements of the building code.[15]
- The electrical needs require special attention in a compact layout. Electrical conduits, chemical pipes, heating-ventilating-air conditioning (HVAC), and plumbing will be concentrated into a smaller area and at two to three floors. Careful planning is needed to avoid conflicts between disciplines in both vertical and horizontal directions, room layout, floor openings, and slab penetrations.
- Compact design uses common-wall construction. Provisions for structural movement caused by concrete shrinkage, expansion, soil movement, and unequal settling must be made. The location of isolation-expansion joints should be coordinated with all disciplines so that appropriate measures can be taken to provide for potential movement.

TABLE 20-1 *Advantages and Design Considerations of Compact Layout*

Parameter	Advantages	Design Considerations
Mechanical	• The major items of mechanical equipment are consolidated into one central area near the support facilities. • Chemical feed lines are relatively short because the chemical application points are all located close to the operations area. • Because there are fewer separate buildings, fewer HVAC systems are required.	• Removal of equipment for off-site maintenance may be more difficult because of restricted access.
Operations	• Short walking distance to all equipment • Mechanical equipment is close to maintenance facilities in operations building. • Covered access by means of galleries and tunnels • Number of plant staff may be reduced because the equipment needing regular attention is located close to the operations building.	• Less room for maintenance activities
Structural	• Reduced excavation volumes • Reduced concrete quantities because of common wall construction • Less total floor area required	• Because electrical and instrumentation cables are concentrated near the operations area, numerous floor openings and slab penetrations are required. • Need to make extra provisions for concrete shrinkage and expansion and for soil movement

Electrical	• Lengths of electrical and instrumentation conduits and cables are reduced. • Cable size (determined by voltage drop) may be reduced because of shorter cable lengths. • Because cables can be routed indoors instead of in buried conduits between structures, cost-efficient cable trays can be used. • Cables routed indoors are more accessible.	
Hydraulic	• Deleting the interconnecting piping associated with separate basins reduces hydraulic losses.	• Long channel or pipe may be required to route settled water to filters.
Architectural	• It is easier to develop an architectural theme for a single mass than for several separate structures.	• Mixed use of building (operations plus administration) complicates building architectural design.
General	• Less land is required. • With fewer structures, the number of internal access roads and the amount of paving on-site is reduced. • Less underground large piping • Because of a compact design, the odors and environmental control facilities are more effectively and economically installed and operated.	• Having one large building containing all facilities makes it more difficult to comply with fire code requirements for access-egress because of the increased use of hallways, stairways, and so forth. • If a uniform top-of-wall elevation is used for all process basins (preliminary, primary, secondary and filters), then freeboard in some basins may be excessive. • Alternatively, if the top-of-wall is varied to achieve a uniform freeboard, this will result in steps in the walkways along the top of the basins. • Increased need for safety precautions arising from proximity of bulk chemical storage area to administration building

Source: Ref. 15

20-2-3 Buildings

Buildings are needed for plant personnel, process equipment, and visitors. The following considerations should be given to building design and location:

1. Location of equipment at the point of maximum usage will be helpful. However, to avoid a cluttered appearance, buildings in groups should be provided with consideration of minimal loss in efficiency.
2. Buildings may be located as barriers to undesired views of the facility.
3. The climate of the area should be considered in building orientation and design to minimize heating, ventilation, lighting, and air-conditioning costs. In cold areas, buildings should not shade trucking and parking areas, thus reducing snow and ice clearance problems.
4. Area and space requirements should be based on number of people served and current and future functions.
5. The administration building should be located near the entrance and should generally be in public view. The administration building should contain offices, laboratory, instrumentation and control room, shower, lavatories, and locker rooms. In addition, the visitors' lobby should have educational displays and tour information.
6. Other building requirements include machine shops with tools and storerooms, garages, and equipment buildings, e.g., pump, compressors, chemicals, and instrumentation.

20-2-4 Shoreline Planning

The waterfront is a prime recreational zone. Erosion of banks, siltation of waterways, and destruction of valuable ecological niches should be protected from damage. Plant structures should be located as far back from the water edge as possible to permit other types of compatible land uses, particularly recreation and open space. Also, the long axis of the plant structures should be perpendicular to the natural water bodies in order to avoid blocking views and accesses.[4] An example of shoreline planning is illustrated in Figure 20-2.

20-2-5 Flood Plain Avoidance

Plant siting in flood plain should be avoided. If necessary, raising of the facilities and buildings above flood levels and construction of flood protection levees should be considered. Provision must be made for collection of surface runoff at a central location. A stormwater management system with outlet structure, flood gate, stormwater pumps, and outfall pipe must be designed for removal of stormwater under critical conditions when the highest flood stage is reached in the receiving water.

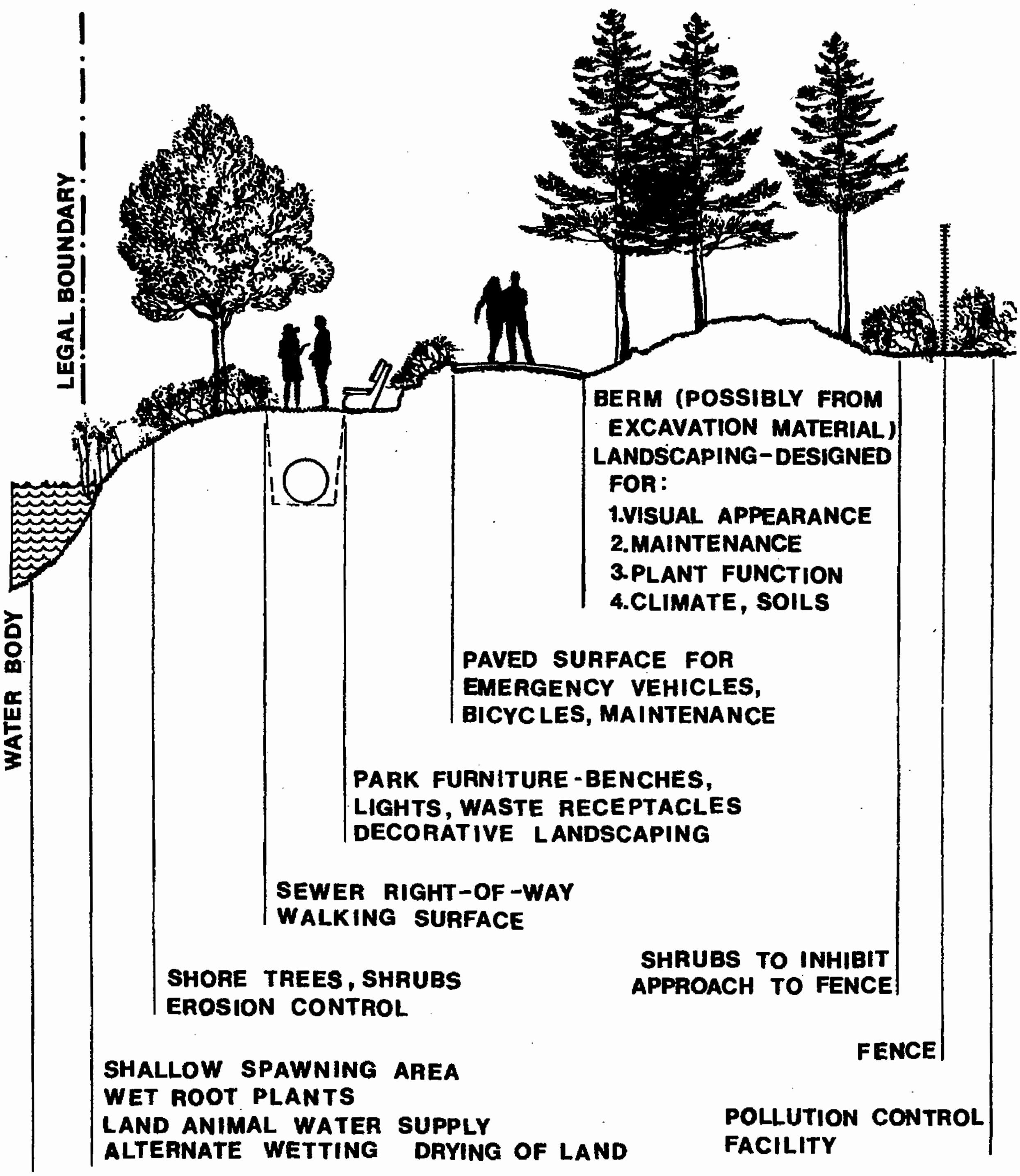


Figure 20-2 Wastewater Treatment Plant Site, Shoreline Planning (from Ref. 2).

20-2-6 Landscaping

Existing site vegetation, trees, and shrubs should be assessed and utilized. Planting should be considered for control of slope erosion, surface runoff, and enhanced attractiveness and to provide sound and odor control barriers. Local soils and climatological and biological conditions should be carefully investigated by a competent landscape architect.

20-2-7 Lighting

Proper lighting at treatment facilities promotes safer operation, efficiency, and security. Considerations should be given to interior, exterior, safety, and security lighting. Illumination to highlight structural and landscape features should also be provided.

20-2-8 Plant Utilities

The utilities at the treatment plant include electrical power, natural gas, water lines, effluent lines, telephone lines, and an intercommunication system. Power generation from wastewater gas is usually not economical, except for larger plants. The cost of power generation is generally determined from careful consideration of many economic factors. The wastewater gas is used to heat the buildings and digesters and to produce hot water and steam for other uses.

The design of the utility system should conform to the applicable codes and regulations of the municipality concerned and to the operating rules of the concerned utilities. All utility lines should be properly shown on the layout plans, marked on the site (if exposed), and grouped properly to facilitate repairs, modifications, and expansion. Additional discussion of utilities may be found in Refs. 1, 14, and 17.

20-2-9 Occupational Health and Safety

A wastewater treatment facility offers many types of occupational health hazards that must be considered as part of plant design and layout. Important factors for which proper safeguards must be provided include chemicals and chemical handling, biological vectors, toxic gases, fire protection, explosions, burns, electric shocks, rotating machinery parts, material and equipment handling, falls and drowning, and the like. Excellent discussions of the subject can be found in Refs. 1, 14, 15, and 17.

20-2-10 Security

All accesses to the treatment plant should be controlled. Fences and other barriers should be provided to enclose the facility. Proper signs should be displayed at all accesses, indicating the name and owner of the facility.

20-2-11 Future Expansion

Provisions for future plant expansion must be made. The provisions should include (1) future space requirements, (2) plant expansion with least interruption to plant operation, and (3) process modifications with little interruption to the existing plant operation.

20-3 DESIGN EXAMPLE

The site development plan of the Design Example is shown in Figure 20-3. Various treatment units, pipings, buildings, access roads and parkings, etc., are properly laid, uti-

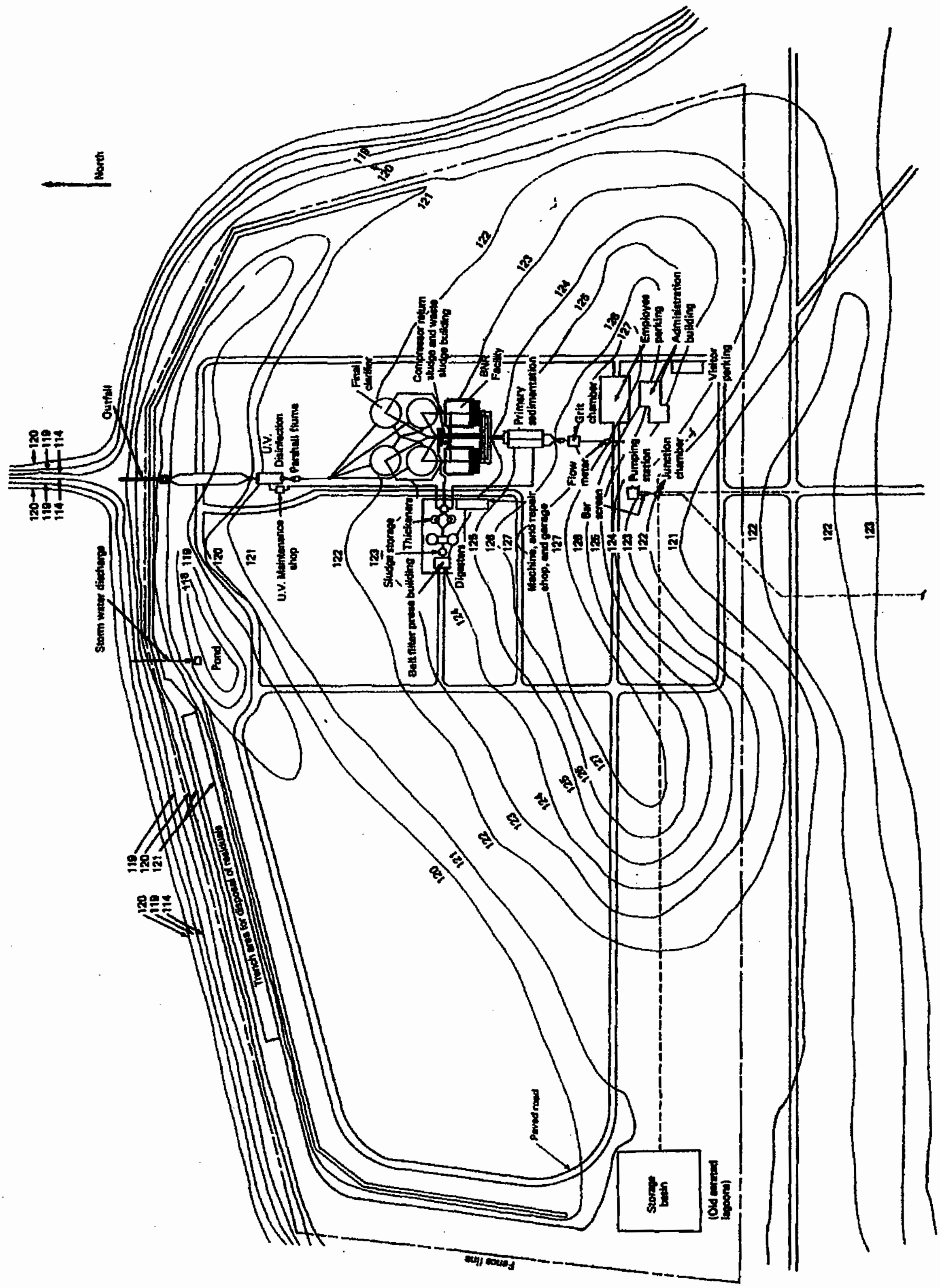


Figure 20-3 Layout Plan and Final Ground Surface Grade of the Treatment Facility in the Design Example.

lizing the many principles and factors discussed earlier. Special considerations are given to social and delicate environmental factors.

20-3-1 Description of the Site

The proposed site is classified as a socially and ecologically sensitive suburban site. An inventory of the existing conditions indicates that the site is a 125-ha, undeveloped tract bounded by two small streams that join immediately north of the site. Suburban developments are on the south, characterized by industrial park and some single- and multiple-family residences (Figure 6-2). The prevailing winds are northernly, and the site access and influent lines are from the south. The existing aerated lagoon is located on the western side of the site (Figure 20-3).

The principal constraints to the site are waterfronts on three sides somewhat haphazardly bounded by the streams. Most of the waterfronts have severe slopes, erodible soils, flood plane, and drainage pattern encroachment. Site inventory, however, shows no valuable vegetation or special wildlife habitats.

20-3-2 Description of Plant Layout and Site Development

After careful consideration of the site development factors, the site plan was prepared, and treatment units and buildings were located. The following points may be noted in Figure 20-3:

1. The physical arrangement of the pumping and treatment units is fairly spread out to utilize the existing features, site topography, and foundation requirements.
2. The pump station is located on the western side, and the wastewater is pumped to a high location from which flow by gravity is achieved in normal sequence of treatment units.
3. The existing aerated lagoon is utilized for diverting the flow from the collection box in case a power failure occurs (see Sec. 6-10-2).
4. The facility has been located away from the developed area, and existing trees and new landscaping have been used as buffers.
5. The administration building is located in the front, with all treatment units far behind. Visitor parking is provided in the front and employees parking in the back. Each equipment building contains a small work area and necessary tools. A separate machine shop and repairs building is provided at a central location for general use.
6. The sludge-processing area is kept on the west side and perpendicular to the process train.
7. Aeration basins are located with landscaping and vegetation around the facility to provide a buffer against aerosols.
8. Sludge return and waste activated sludge pumps and compressors are located in the adjacent buildings to facilitate operation and maintenance.
9. Ample space has been provided for each treatment unit for future expansion. Such expansion would maintain modular design and centralization of similar processing units.

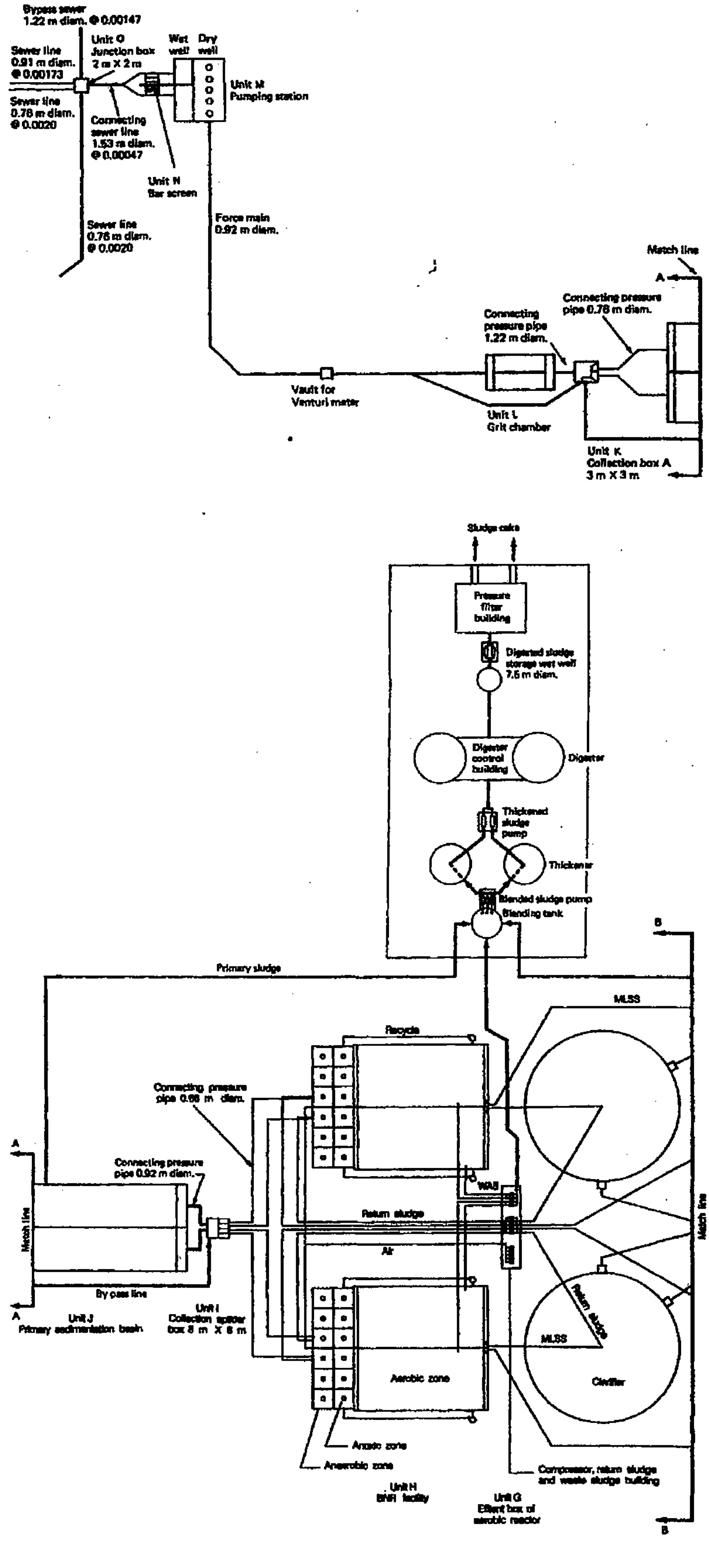


Figure 20-4 Plant Layout and Connecting Yard Pipings (four sections of the plant layout are connected by match lines AA, BB, and CC).

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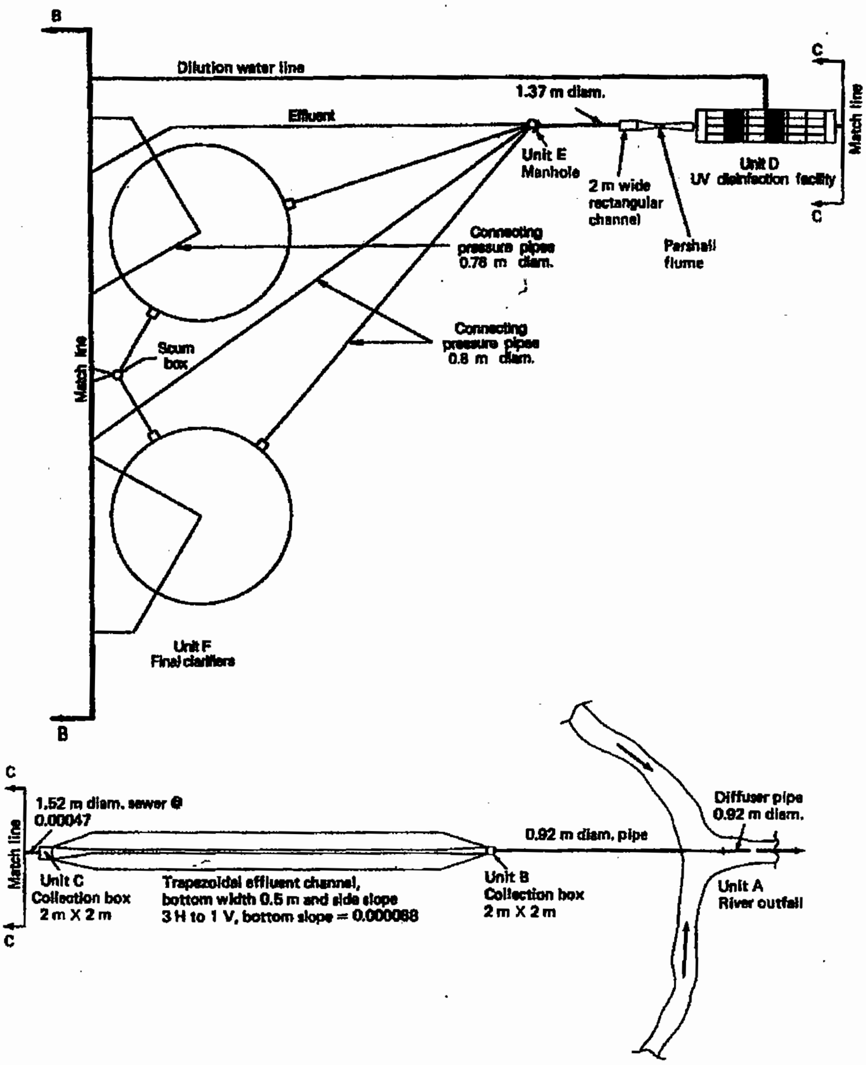

Figure 20-4—cont'd

10. The surface is drained away from the main structures, and the entire site is sloped towards the sedimentation pond at the northern edge of the site plan. The pond is located at the lowest location to permit natural drainage.
11. As much as possible, the developed site does not alter the existing naturally stabilized site contour and drainage pattern. The landscaping reflects the rolling character of the surrounding area. The entire area is sodded to reduce erosion.
12. The stormwater structure at the sedimentation pond utilizes flood control gates and stormwater pumps to remove surface runoff under the highest flood level on the other side of the levees.
13. The layout of various units is compact, with sufficient lengths of connecting conduits for future expansion.
14. Paved service roads pass by each facility.
15. The facility has been located such that the use of the river edge as a recreation area can be accomplished by building garden walks and esplanades similar to those shown in Figure 20-2.

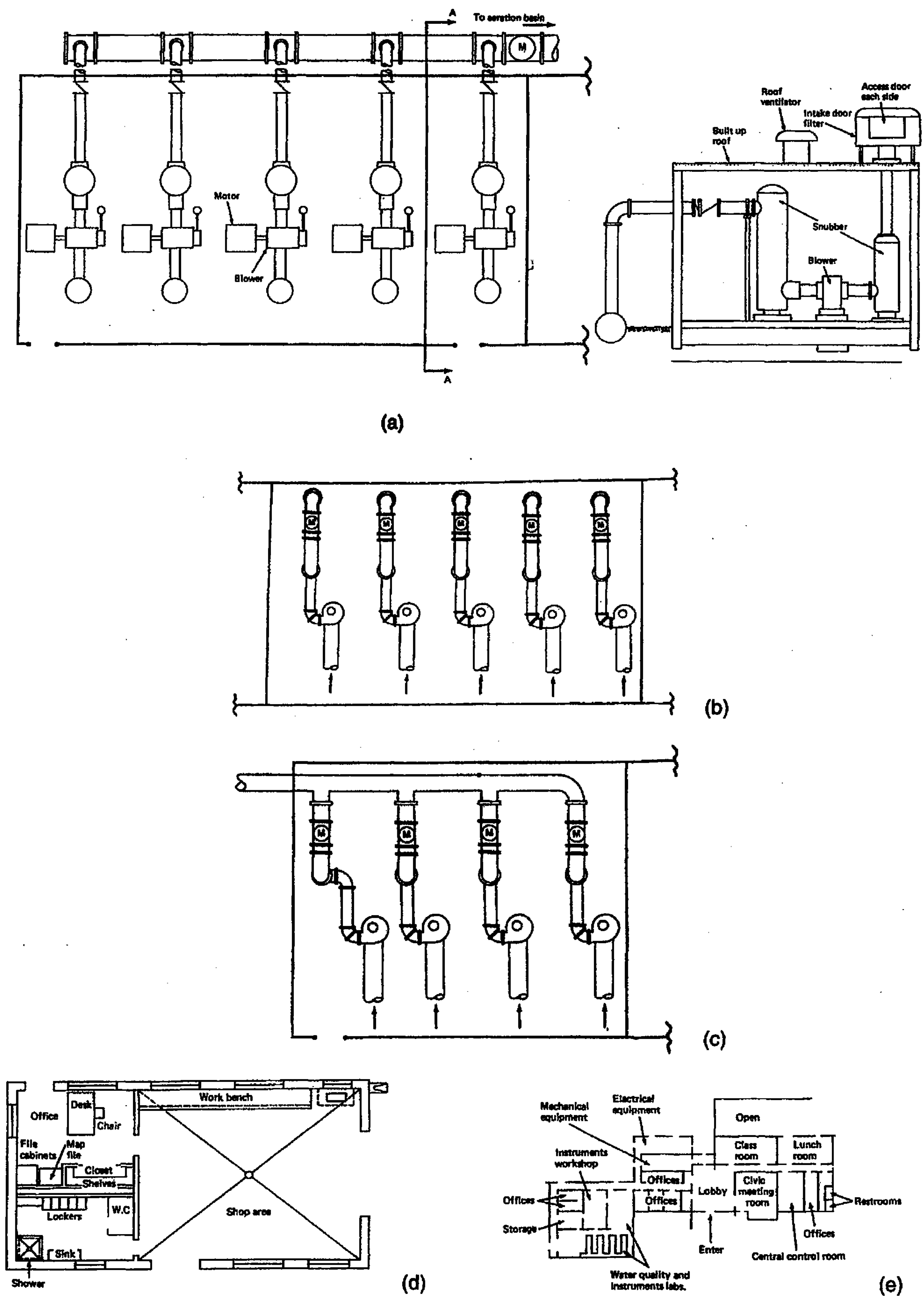

Figure 20-5 Details of Utility, Equipment, and Administration Buildings: (a) floor plan and sectional view of blower, (b) return sludge pumps, (c) waste activated sludge pumps, (d) maintenance and machine shop, and (e) administration building.

With the above design and site development goals, the plant layout has been prepared. Additional considerations were made for design of individual unit operations and processes. Such considerations may be found in Chapters 7–19. The site plan with finished contours is shown in Figure 20-3. Unit details and connecting pipings are shown in Figure 20-4. The plans of the administration and other utility buildings are given in Figure 20-5.

20-4 PROBLEMS AND DISCUSSION TOPICS

20-1 As a design engineer, you have the responsibility to develop the plant layout, site plan, landscaping, and finished contours of a wastewater treatment facility. List the basic rules that you must consider in developing such a plan.

20-2 Study the site plan given in Figure 20-3. Identify those features that do not conform to the basic rules of the site plans that you have listed in Problem 20-1.

20-3 In Figure 20-3, a total area of 80 ha drains toward the storage pond. Assuming a rainfall intensity of 0.89 cm/h over the time of concentration of 2 h and the coefficient of runoff of 0.13, design a suitable stormwater outfall system at the storage pond. The outfall system should include an outlet structure with flood gate, outfall pipe, and stormwater pumps for emergency conditions. Use rational formula for calculation of stormwater runoff (see Sec. 26-3-2).

20-4 Review the layout plan, finished contours, and landscaping features of the wastewater treatment plant in your community. Compare the important features of this plant layout with that of the Design Example given in Figure 20-3.

20-5 Design a stormwater pumping station for the flow calculated in Problem 20-3. Describe the type of pumps and reasons for their selection. Draw the details of the pumping station. The static head is 7 m and force main is 50 m.

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21

Yard Piping and Hydraulic Profile

21-1 INTRODUCTION

Yard piping includes connecting conduits, collection and division boxes, valves and gates, and manholes and appurtenances between various treatment units. After the treatment units and the connecting pipes and appurtenances are marked on the layout plan, the head losses through pipings and treatment units are calculated. Hydraulic profile is the graphical representation of the hydraulic grade line through the plant. The elevations of treatment units and pipings are adjusted to give adequate hydraulic gradient to ensure gravity flow.

Many treatment plants have encountered serious operational problems (even flooding under peak flow) because of an inadequate hydraulic gradient through the various units. It is therefore important that the design engineer take proper care in developing the hydraulic profile. In this chapter the basic principles of designing yard piping and preparation of hydraulic profile are presented. Furthermore, the calculation procedure and graphical representation of the hydraulic profile for the Design Example are shown.

21-2 YARD PIPING

The arrangement of channels, pipelines, and appurtenances is important to transmit flows from one treatment unit to the other. Often, flows are collected from several treatment units operating in parallel or divided into several units. Hydraulically similar inlet pipings and channels, as well as splitting boxes, are normally used for flow splitting and solids distribution. There are three basic considerations in preparing the piping layout: (1) convenience of construction and operation, (2) accessibility for maintenance, and (3) ease with which future connections can be made or more lines added.

Small distances between the adjacent treatment units can provide compact plant layout and minimize the costs of connecting pipings. However, it should not be forgotten that the connecting pipings and space between the treatment units are the primary means to expand a plant expeditiously. Many unexperienced designers overlook this fact. Plants with tight pipings and hydraulics have resulted in serious expansion problems; often, pumping was needed to route the flows through the new treatment units.

The connecting channels and pipings may be above or below ground level. However, all pipings must be clearly marked with reference to the treatment units on the layout plans. Provisions for future tie-ins should be made and properly indicated on the plans. Valves or gates should be provided in the connecting piping to isolate or bypass units and modules for routine servicing and maintenance. Clearly marked schematic drawings with treatment units, connecting piping, and valves will assist the designers and operator to understand the flow routing and unit isolation and, in general, to understand the operational capability of a well-designed plant.

In many plants, underground tunnels (called pipe galleries or operating galleries) are constructed to locate pipings and necessary controls. Although such galleries provide access to pipings and controls and passage between the buildings, such expenditures can be justified only for large plants. In small- and medium-sized plants, pipes are commonly located above or below the ground. Plant construction photographs and videos showing yard pipings are very helpful in reaching proper pipes and controls for operation and maintenance needs.

21-3 PLANT HYDRAULICS

A hydraulic profile is the graphical representation of the hydraulic grade line through the treatment plant. If the high-water level in the receiving water is known, this level is used as a control point, and the head loss computations are started backward through the plant. Sometimes the computations are started in the direction of the flow from the interceptor, using the water surface as the reference level. In some plants, the hydraulic calculations are started somewhere in the middle using an arbitrary elevation. At the end, the elevations of water surface are adjusted in both directions.

The total available head at a plant is the difference of water surface elevation in the interceptor and the water surface elevations in the receiving water at high flood level. If the total available head is less than the head loss through the plant, flow by gravity cannot be achieved. In such cases pumping is needed to raise the head so that flow by gravity can occur.

There are many basic principles that must be considered when preparing the hydraulic profile through a plant. Some of these principles are listed below:

1. The hydraulic profiles are prepared at peak and average design flows and at minimum initial flow.
2. The hydraulic profile is generally prepared for all main paths of flow through the plant.
3. The total head loss through a treatment plant is the sum of head losses in the treatment units and in the connecting piping and appurtenances.
4. The head losses through the treatment unit include the following:
   a. Head losses at the influent structure
   b. Head losses at the effluent structure
   c. Head losses through the unit
   d. Miscellaneous and free-fall surface allowances

The largest head loss through a treatment unit may occur at peak design flow plus recirculation when the largest unit is out of service.

The approximate head losses across different treatment units may be as follows:

Bar screen	0.02–0.3m
Grit removal	
Aerated grit channels	0.5–1.2m
Velocity-controlled grit channel	1.0–2.5m
Primary sedimentation	0.5–1.0m
Aeration basin	0.3–0.8m
Trickling filter	
Low rate with dosing tank	3.0–6.0m
High rate, single stage	2.0–5.0m
Secondary clarifier	0.5–1.0m
Disinfection facility	0.2–2.5m

5. The total loss through the connecting pipings, channels, and appurtenances is the sum of the following:
   a. Head loss because of entrance
   b. Head loss because of exit
   c. Head loss caused by contraction and enlargement
   d. Head loss caused by friction
   e. Head loss caused by bends, fittings, gates, valves, and meters
   f. Head required over weir and other hydraulic controls
   g. Free-fall surface allowance
   h. Head allowance for future expansions of the treatment facility
6. The velocity in the connecting pipings and conduits is kept large enough to keep the solids in suspension. A minimum velocity of 0.6 m/s at peak design flow is considered adequate. At minimum initial flow, a velocity of 0.3 m/s is considered necessary to transport the organic solids. Often, the ratio of maximum and minimum flows is so large that self-cleaning velocity cannot be maintained under the initial flow conditions. In such cases the flushing actions can only be achieved under higher flows. Frequency of occurrence of flushing actions must be considered in such designs. In many cases aeration is provided to keep solids in suspension. It may even be desirable to provide separate lines from multiple units so that the line could be cleaned when a unit is out of service.
7. The minor head losses in open channels and conduits are calculated in terms of the velocity head. Detailed discussions may be found in Chapters 7 and 9.
8. Friction losses in pressure conduits are obtained using the Hazen-Williams formula (see Chapter 9).
9. In channels, the depth of flow varies, depending on the flow conditions. Therefore, the depth and grade of an open channel is kept in such a way that the water surface at the design flow corresponds to the hydraulic profile (see Chapter 7).

   In open channels, the flow may be either uniform or nonuniform. Uniform flow occurs in channels that have constant cross section, flow, and velocity. Manning's equation is generally used to calculate the grade of the water surface.

In design of channels, it is generally assumed that the flow is uniform at peak design flow.

Nonuniform flow exists in channels when cross section changes or the volume of wastewater entering the channel is not constant. Nonuniform flow generally occurs in channels that have free-fall, effluent flumes or launders, or channel junctions with surcharge. Friction formula does not apply for nonuniform flows. Backwater or drawdown analysis is necessary. Computational techniques for nonuniform flows are covered in Chapters 11–15. Readers are referred to some excellent textbooks on hydraulics of open channel for more detailed discussion on this subject.[1-4]

In wastewater treatment plants sufficient allowances are made for the transitions and nonuniform flows by providing invert drops. Head loss through the transitions is generally calculated by using the energy equation.

10. Most of the flow-measuring devices used in wastewater treatment plants operate with head loss. Proper head loss calculations should be made for flow-measuring devices (Venturi tube, Parshall flume, orifice plate, weir, etc.) and included in the hydraulic profile.
11. In preparation for the hydraulic profile, the vertical scale is intentionally distorted to show the treatment facilities and the elevation of the water surface. Ground surface is also indicated to establish the optimum elevation of the plant structures and the hydraulic controls.

21-4 DESIGN EXAMPLE

The procedure for preparing the hydraulic profile through the treatment plant designed in Chapters 7–15 is presented in this section. The procedure involves determination of head loss as the wastewater flows through various treatment units, the connecting conduits, and appurtenances.

21-4-1 Head Losses across Treatment Units

The head loss calculations and hydraulic profiles across various treatment units have been developed in other chapters. The head loss calculations were made under peak design flow when the largest component was out of service. The hydraulic capacity and the head loss across each treatment unit are summarized in Table 21-1. The head loss across a treatment unit is the difference of water surface elevations in the influent and effluent structures.

21-4-2 Head Losses in Connecting Pipings

The total head loss in the connecting piping is the sum of all head losses encountered in the pipings, channels, collection and division boxes; head over weir; and allowance for free-fall and future expansions. The largest head loss generally occurs at the control points such as weir, flume, drop manhole, etc. The head losses caused by friction, bends, entrances, and exits may also be significant. A detailed plant layout with piping schedule

TABLE 21-1 Hydraulic Capacity and Head Loss through Individual Treatment Units

Treatment Unit	Total Number of Components	Flow	Hydraulic Capacity	Head Loss across the Unit (m)	Ref.
Bar screen	2	Peak design flow	Flow through one unit (clean rack)	0.03	Figure 8-6
Pump station	5	Peak design flow	Four units	12.70 (operating head at max. station capacity)	Table 9-9
Venturi meter	1	Peak design flow	Flow through one unit	0.45	Sec. 10-7-2, Step C
Grit chamber	2	Peak design flow	Flow through one unit	1.07	Figure 11-7
Primary sedimentation[a]	2	Peak design flow	Flow through two units	0.98	Figure 12-19
BNR facility	4	Peak design flow plus recirculation	Flow through four units	1.07	Figure 13-26
Final clarifier	4	Peak design flow plus recirculation	Flow through four units	0.55	Figure 13-29
Parshall flume and disinfection facility (UV system)	4	Peak design flow	Flow through four units	1.64	Figure 14-23
Outfall	1	Peak design flow	Flow through one unit	1.66	Figure 15-1

[a]When one unit is out of service, the flow is bypassed to the aeration basin.

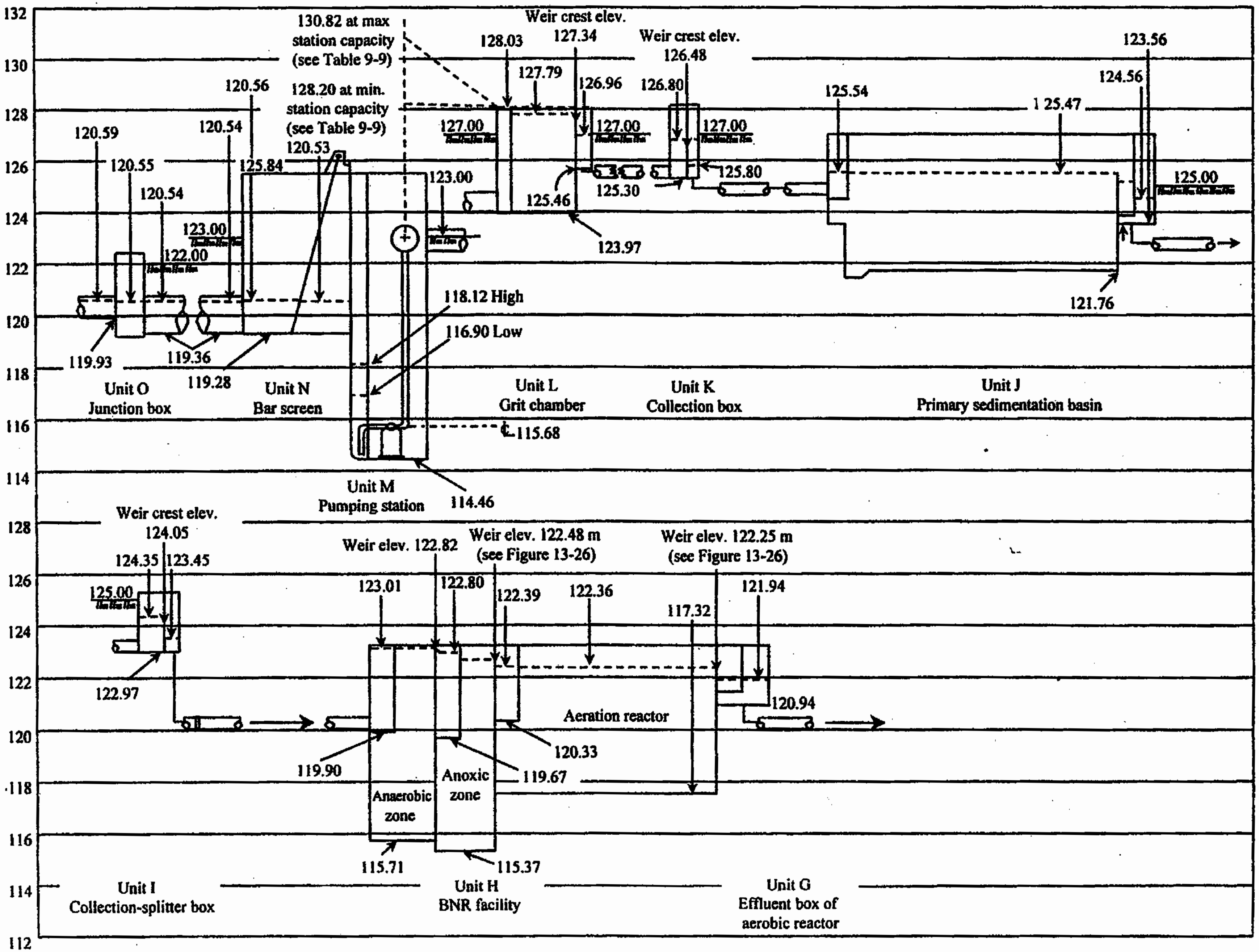

Figure 21-1 Hydraulic Profile through the Treatment Facility in the Design Example.

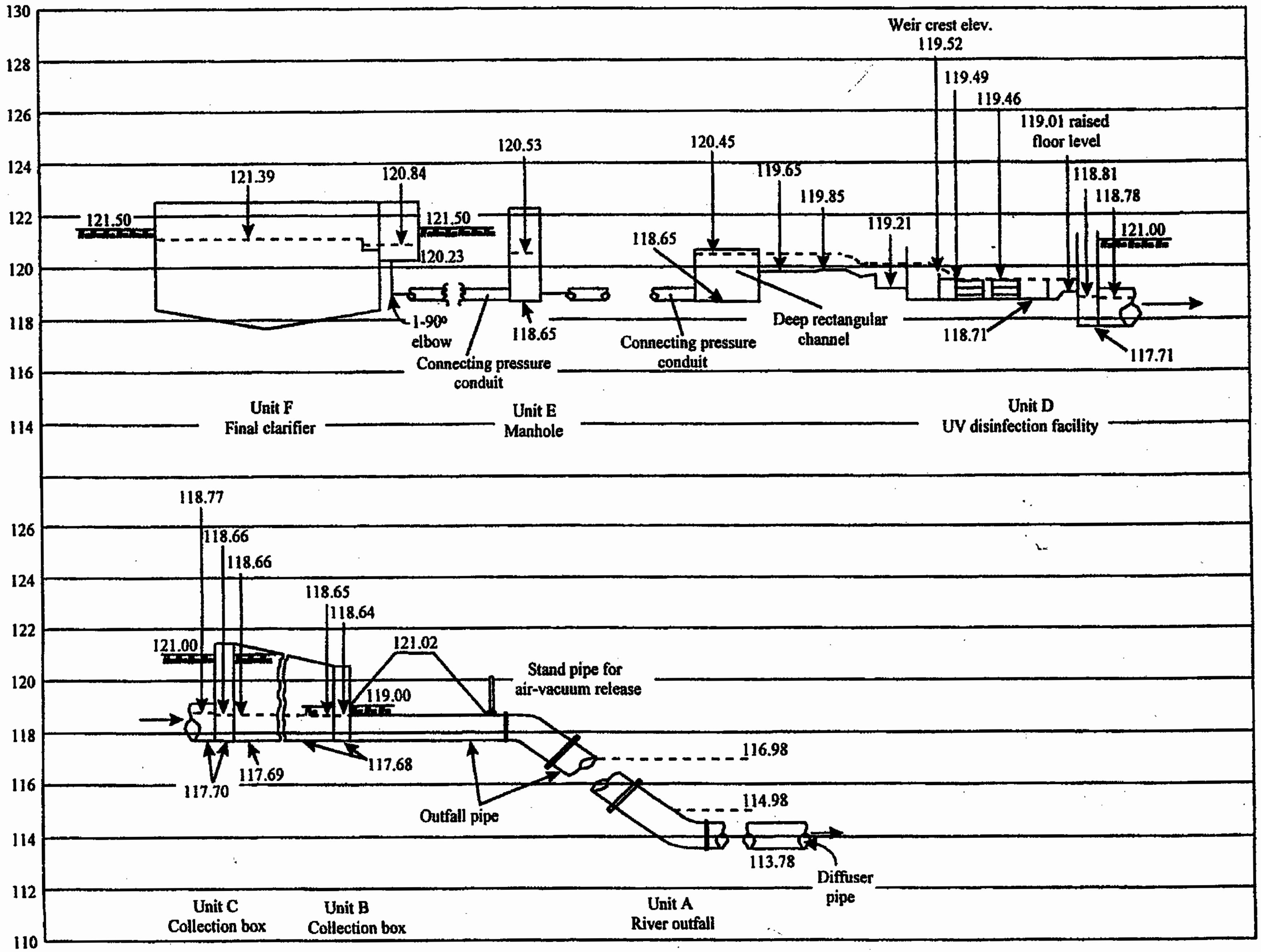

Figure 21-1—cont'd

showing straight lengths, as well as vertical and horizontal bends, should be prepared. Then the head loss calculations are started. These calculations are tedious and time-consuming. It is suggested that the design engineer should use a format that is systematic, concise, and easier to perform and check calculations.

Preparation of plant layout has been discussed in Chapter 20. Proposed plant layout and the connecting pipings of the Design Example are illustrated in Figures 20-3 and 20-4. Head loss calculations are started from the high-water elevation in the receiving water and carried backward. The hydraulic profile is shown in Figure 21-1. The head loss calculations are summarized in the following sections. Other examples of head loss calculations through the treatment units and connecting conduits and appurtenances may be found in Refs. 4–7.

21-4-3 Head Loss Calculations for Connecting Conduits

Step A: Outfall Conditions (Unit A). High-water surface elevation in the receiving stream (Figure 21-1) = 116.98 m. Stream bottom elevation = 113.78 m.

Step B: Water Surface and Invert Elevations in Collection Box (Unit B). The effluent structure consists of effluent pipe and diffusers.

1. Head loss in the diffuser and outfall pipe (entrance, bends, and friction, see Sec. 15-4-2, Step E) = 1.66 m
2. Water surface elevation in unit B = 116.98 m + 1.66 m = 118.64 m
3. Invert elevation if pipe submergence is 0.04 m = 118.64 m − 0.92 m − 0.04 m = 117.68 m

Step C: Water Surface and Invert Elevations in Collection Box (Unit C). The connecting conduit is a trapezoidal channel. The channel has a bottom width of 0.5 m and a side slope of 3 horizontal to 1 vertical. The ends of the channel have smooth transition to connect into a 2-m-wide rectangular opening in the box.[a]

1. Compute the depth of flow and velocity in the rectangular portion of the channel.

Peak design flow = 1.321 m^3/s

The depth of flow in the rectangular portion of the channel = 0.96 m

$$\text{Velocity at the rectangular portion of the channel} = \frac{1.321\ \text{m}^3/\text{s}}{0.96\ \text{m} \times 2\ \text{m}} = 0.69\ \text{m/s}$$

[a]The purpose of the trapezoidal channel is to provide postreaeration to increase the dissolved oxygen in the effluent. Because of a short detention time in the channel, photoreactivation of fecal coliform is not expected to occur in the channel.

2. Compute the velocity and depth of flow in the trapezoidal channel.
Apply the energy equation [Eq. (8-4)] at trapezoidal and rectangular sections [see Figure 15-1(c)]. This is a smooth contraction. For smooth transition of sides and bed slopes, $K_c = 0.2$ and $(Z_1 - Z_2) = 0$.

$$y_1 + \frac{(q/a)^2}{2g} = 0.96 \text{ m} + \frac{(0.69 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2} + 0.2\left[\frac{-(q/a)^2}{2g} + \frac{(0.69 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2}\right]$$

$$q = 1.321 \text{ m}^3\text{/s}, \ a = \frac{0.5 \text{ m} + (6\,y_1 + 0.5) \text{ m}}{2} \times y_1 \text{ (m)}$$

The solution of the above equation is reached by a trial-and-error procedure. Solving the above equation,

$$y_1 = 0.97 \text{ m}$$

$$a = \frac{0.5 \text{ m} + [(2 \times 3 \times 0.97 \text{ m}) + 0.5 \text{ m}]}{2} \times 0.97 \text{ m}$$

$$= 3.31 \text{ m}^2$$

$$\text{Velocity} = \frac{q}{a} = \frac{1.321 \text{ m}^3\text{/s}}{3.31 \text{ m}^2} = 0.40 \text{ m/s}$$

3. Compute the slope of the invert and water surface elevations at the trapezoidal channel near the collection box (unit B).

Invert elevation (same for unit B) = 117.68 m
Water surface elevation = 117.68 m + 0.97 m = 118.65 m

4. Compute the slope of the trapezoidal channel from the Manning equation [Eq. (7-1)]. For a trapezoidal channel at depth = 0.97 m,

$$a = 3.31 \text{ m}^2, \ n = 0.013$$

$$\text{Wetted parameter} = 0.5 \text{ m} + 2\sqrt{(0.97\text{m})^2 + (3 \times 0.97 \text{ m})^2}$$

$$= 0.5 \text{ m} + 6.13 \text{ m} = 6.63 \text{ m}$$

$$r = \frac{a}{\text{wetted perimeter}} = \frac{3.31 \text{ m}^2}{6.63 \text{ m}} = 0.50 \text{ m}$$

$$s = \left[\frac{0.40 \text{ m/s} \times 0.013}{(0.50)^{2/3}}\right]^2 = 0.000068$$

5. Compute the invert and water surface elevations in the trapezoidal channel near unit C.

Differential elevation of the channel bottom because of slope	= channel length $\times$ slope
	= 95 m $\times$ 0.000068
	= 0.0065m $\approx$ 0.01 m
Invert elevation	= 117.68 m + 0.01 m = 117.69 m
Water surface elevation	= 117.69 m + 0.97 m = 118.66 m

6. Compute depth of flow and velocity in unit C.
   Use the energy equation as in Step C, 2. Raise the invert elevation of unit C by 0.01 m and solve for depth of flow and velocity by trial and error.

   Depth of flow = 0.96 m
   Velocity = 0.41 m/s

7. Compute the water surface elevation in unit C.

   The water surface elevation in unit C = 117.69 m + 0.01 m + 0.96 m
   = 118.66 m

Step D: Water Surface and Other Elevations in the UV Disinfection Facility (Unit D). A 1.52-m sewer line connects unit C with the effluent channel of the UV disinfection facility.

1. Compute the depth of flow in the connecting sewer line at peak design flow.
   Diameter and slope of connecting sewer are 1.52 m and 0.00047, respectively. The sewer will be flowing partially full. From the Manning equation (n = 0.013), the depth of flow and velocity at peak design flow of 1.321 m^3/s are 1.07 m and 0.97 m/s (see Chapter 7 for calculation procedure).
2. Compute the invert and water surface elevations in the connecting sewer line.
   Set the invert of the connecting sewer the same as that of unit C. Using a sudden expansion outlet with a coefficient of 0.75, the upstream depth of 0.95 m is obtained from the energy equation. This gives subcritical flow in the sewer, and an M2 profile is indicated. The maximum depth of water cannot exceed the uniform depth of 1.07 m. Therefore, an upstream depth of 1.07 m in the sewer will give a conservative design. Water surface elevation in sewer is 118.77 m and invert elevation is 117.70 m.
3. Compute the invert and water surface elevations in the connecting chamber to the effluent channel of the UV disinfection facility.

Differential invert elevation of the connecting sewer line	= slope $\times$ length
	= 0.00047 $\times$ 24 m = 0.01
Invert elevations of the sewer and that of the connecting chamber to the UV effluent channel	= 117.71 m

Water surface elevation in the sewer at the effluent channel	$= 117.7\text{ m} + 1.07\text{ m}$ $= 118.78\text{ m}$
Entrance loss ($K = 0.15$)	$= \dfrac{0.15 \times (0.97\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = 0.01\text{ m}$
Miscellaneous losses caused by change in direction and turbulence	$= 0.02\text{ m}$
Water surface elevation in the connecting chamber to the UV effluent channel	$= 118.78\text{ m} + 0.01\text{ m} + 0.02\text{ m}$ $= 118.81\text{ m}$

4. Compute various elevations in the UV disinfection facility. The hydraulic profile through the facility was prepared in Chapter 14. Several key water surface and invert elevations in the UV disinfection channel are shown in Figure 21-1. Total head loss through the facility including Parshall flume = 1.64 m.

The water surface elevation in the rectangular channel upstream of the Parshall flume	$= 118.81\text{ m} + 1.64\text{ m} = 120.45\text{ m}$
The invert elevation of the rectangular channel of the disinfection facility	$= 120.45\text{ m} - 0.8\text{ m}$ (depth of flow) $= 119.65\text{ m}$

Step E: Water Surface and Invert Elevations in Collection Box (Unit E). The connecting conduits between the Parshall flume and unit E are a deep rectangular channel and a pressure pipe. The rectangular channel is 2 m wide and 4 m long (Figure 20-4) and 1.8 m deep. The water surface and invert elevations in the rectangular channel are 120.45 and 118.65 m, respectively.

1. Compute the head loss in the connecting pressure pipe.

$$\text{Diameter of the pipe} = 1.37\text{ m}$$
$$Q = 1.321\text{ m}^3/\text{s}$$
$$V = \frac{1.321\text{ m}^3/\text{s}}{(\pi/4)\,(1.37\text{ m})^2}$$
$$= 0.90\text{ m/s}$$
$$L = 40\text{ m}$$

Friction loss is calculated from the Hazen-Williams equation [Eq. (9-3)].

$$C = 100$$

$$h_f = 6.82\left(\frac{0.90}{100}\right)^{1.85} \times \frac{40\text{ m}}{(1.37)^{1.167}}$$

$$\text{Exit loss at the rectangular channel upstream of Parshall flume } (K = 0.5) = \frac{0.50 \times (0.90\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2}$$

$$= 0.02\text{ m}$$

$$\text{Miscellaneous losses} = 0.01\text{ m}$$

$$\text{Total losses in the connecting pipe} = 0.03\text{ m} + 0.02\text{ m} + 0.01\text{ m}$$

$$= 0.06\text{ m}$$

$$\text{Entrance loss at pressure pipe } (K = 0.35) = \frac{0.35 \times (0.95\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = 0.02\text{ m}$$

$$\text{Water surface elevation in the manhole (unit E)} = 120.45\text{ m} + 0.06\text{ m} + 0.02\text{ m}$$

$$= 120.53\text{ m}$$

Invert elevation of the manhole is kept the same as that of the rectangular channel upstream of the Parshall flume.

$$\text{Invert elevation of manhole (unit E)} = 118.65\text{ m}$$

Step F: Water Surface and Other Elevations in the Final Clarifier (Unit F). There are four pressure pipes connecting the manhole (unit E) with the effluent box of the final clarifiers. The piping is sized for emergency condition when one clarifier is out of service. The calculations for the longest path are given below:

1. Compute the head loss through the final clarifier.

$$\text{Peak design flow when three clarifiers are in operation} = \frac{1.321\text{ m}^3/\text{s}}{3}$$

$$= 0.44\text{ m}^3/\text{s}$$

$$\text{Diameter of the connecting pipe} = 0.80\text{ m}$$

$$\text{Velocity} = \frac{0.44\text{ m}^3/\text{s}}{(\pi/4)\,(0.8\text{ m})^2} = 0.9\text{ m/s}$$

$$\text{Exit loss at unit E } (K = 0.5) = \frac{0.5 \times (0.9\text{ m/s})^2}{2 \times 9.81\text{ m/s}^2} = 0.02\text{ m}$$

Minor loss (1–90° elbows, see Figure 21-1, $K = 0.3$)

$$= \frac{1 \times 0.3 \times (0.9 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2} = 0.02 \text{ m}$$

Entrance loss at unit F ($K = 1.0$) (final clarifier) $= \dfrac{1.0 \times (0.9 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2} = 0.04$ m

Friction loss from the Hazen-Williams equation [Eq. (9-3)] ($C = 100$ and $L = 140$ m) = 0.20 m

Assume miscellaneous losses = 0.03 m

Total loss = 0.02 m + 0.02 m + 0.04 m + 0.20 m + 0.03 m

= 0.31 m

2. Compute the water surface and other elevations in the final clarifiers (unit F).

Water surface elevation in the effluent box = 120.53 m + 0.31 m
= 120.84 m

Invert elevation of the effluent box of the final clarifier = 120.84 m − 0.61 m (see Chapter 13)

= 120.23 m

Maximum water surface elevation in the influent central well at peak design flow plus recirculation = 120.84 m + 0.55 m (Table 21-1)

= 121.39 m

Consult Chapter 13 for other elevations in the final clarifiers.

Step G: Water Surface Elevation in the Effluent Box of Aerobic Reactor (Unit G). The MLSS from the effluent box of the aerobic reactor is piped to the central wells of respective final clarifiers. The connecting conducts are pressure pipes. There are four connecting pipes. The piping is sized for emergency condition when one basin is out of service. The calculations for the longest path are given below:

1. Compute the head loss in the pressure pipe connecting the effluent box of the aerobic basin to the central well of the final clarifier.

Peak design flow plus recirculation when one clarifier is out of service (volume of WAS is ignored) $= \dfrac{1.321 \text{ m}^3/\text{s} + 0.292 \text{ m}^3/\text{s}}{3}$

$= 0.538 \text{ m}^3/\text{s}$

The diameter of the connecting pressure pipe $= 0.76$ m

Velocity in the pipe $= \dfrac{0.538\ \text{m}^3/\text{s}}{\pi/4\ (0.76\ \text{m})^2} = 1.19$ m/s

Exit loss at the central well ($K = 1.0$) $= \dfrac{1 \times (1.19\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.07$ m

Minor loss (1–90° elbows, see Figure 21-1, $K = 0.3$) $= \dfrac{1 \times 0.3 \times (1.19\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2}$

$= 0.02$

Minor loss (3–45° elbows, see Figure 20-4, $K = 0.2$) $= \dfrac{3 \times 0.2 \times (1.19\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2}$

$= 0.04$ m

Friction loss from the Hazen-Williams equation [Eq. (9-3)] ($C = 100$, $L = 130$ m) $= 0.35$ m

Entrance loss at the effluent box of aerobic reactor ($K = 0.5$) $= \dfrac{0.5 \times (1.19\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.04$ m

Assume miscellaneous losses $= 0.03$ m

Total head loss in connecting pipe $= 0.07\ \text{m} + 0.02\ \text{m} + 0.04\ \text{m} + 0.35\ \text{m}$

$+ 0.04\ \text{m} + 0.03\ \text{m}$

$= 0.55$ m

2. Compute the water surface elevation in the effluent box of the aerobic reactor.

$= 121.39\ \text{m} + 0.55\ \text{m}$
$= 121.94$ m

Invert elevation in the effluent box of the aerobic reactor $= 121.94\ \text{m} - 1.00\ \text{m}$
$= 120.94$ m

3. Determine weir elevation at the effluent trough (Figure 13-26) $= 120.94\ \text{m} + 0.56\ \text{m} + 0.75\ \text{m} = 122.25$ m

Step H: Water Surface Elevations in the BNR Facility (Unit H). The water surface elevations in the effluent box of the aeration basin (unit G) and the influent channel are obtained from Figure 13-26.

1. Compute water surface and other elevations in the aeration basin and the influent channel of the anaerobic reactor.
   The hydraulic profile through the BNR facility is shown in Figure 13-26.

Head loss in the aeration basin at peak design flow plus return sludge and recirculation	= 0.45 m
Water surface elevation in the influent channel of aeration basin	= 121.94 m + 0.45 m = 122.39 m
Invert elevation of the influent channel	= 122.39 m − 2.06 m (Figure 13-26) = 120.33 m
Weir elevation at the effluent structure of anoxic reactor	= 120.33 m + 2.15 m (Figure 13-26) = 122.48 m
Head loss in the anoxic zone at peak design flow plus return sludge and recycle	= 0.41 m
Water surface elevation in the influent channel of anoxic reactor	= 122.39 m + 0.41 m
	= 122.80 m
Invert elevation of influent channel of anoxic reactor	= 122.80 m − 3.13 m (Figure 13-26)
	= 119.67 m
Weir elevation of effluent structure of anaerobic reactor	= 119.67 m + 3.15 m (Figure 13-26)
	= 122.82 m
Head loss in anaerobic reactor at peak design flow plus return sludge	= 0.21 m
Water surface elevation in the influent channel of anaerobic reactor	= 122.80 m + 0.21 m = 123.01 m
Invert elevation of the influent channel of anaerobic reactor	= 123.01 m − 3.11 m (Figure 13-26)
	= 119.90 m

Other elevations are given in Figures 13-26 and 21-1.

Step I: Water Surface and Other Elevations in the Collection-Splitter Box (Unit I). The collection-splitter box (unit I) is located upstream of the BNR facility. This box receives flow from the primary sedimentation basin and side streams from the sludge-processing area. The flow is then discharged over four identical weirs for division into four pressure pipes that lead to four aeration basins. Each weir is adjustable, has stop gate, and is equipped with manual head measurement system over the weirs.

1. Compute the head loss in the pressure pipe connecting the splitter box and the influent channel of anaerobic reactor of BNR facility.

Peak flow in the pipe when one process train is out of service[b] $= \dfrac{1.321\ \text{m}^3/\text{s}}{3}$

$= 0.44\ \text{m}^3/\text{s}$

Pipe diameter $= 0.66\ \text{m}$

Velocity $= \dfrac{0.44\ \text{m}^3/\text{s}}{(\pi/4)\,(0.66\ \text{m})^2} = 1.29\ \text{m/s}$

Exit loss at the aeration basin ($K = 1.0$) $= \dfrac{1.0 \times (1.29\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.09\ \text{m}$

Minor losses (3–90° elbows, one elbow in vertical plane and two elbows in horizontal plane, $K = 0.3$)

$$= \frac{3 \times 0.3 \times (1.29\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.09\ \text{m}$$

Entrance loss at the splitter box ($K = 0.5$) $= \dfrac{0.5 \times (1.29\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2}$

$= 0.04\ \text{m}$

Friction loss ($L = 50$ m longest line, $C = 100$) $= 0.18\ \text{m}$

Miscellaneous losses $= 0.04\ \text{m}$

Total head loss $= 0.09\ \text{m} + 0.09\ \text{m} + 0.04\ \text{m} + 0.18\ \text{m} + 0.04\ \text{m}$

$= 0.44\ \text{m}$

Water surface elevation $= 123.01\ \text{m} + 0.44\ \text{m} = 123.45\ \text{m}$

Invert elevation $= 123.45 - 0.48\ \text{m}$ (water depth in the box)

$= 122.97\ \text{m}$

2. Compute the weir crest and water surface elevation in the collection box.
The length of the rectangular weir is 1.50 m. The procedure for calculation of head over weir is the same as presented in Sec. 11-10-2, Step E, 2. From Eq. (11-11), head over weir = 0.30 m. Provide an allowance of 0.60 m for free-fall.

Elevation of weir crest = 123.45 m + 0.60 m = 124.05 m

[b]The flow caused by returned side streams from the sludge-processing area is small. The return sludge goes directly to different anaerobic chambers of the BNR facility.

A free-fall allowance of 0.60 m may seem high. The reason is to build an extra safety because this collection-splitter box is the junction between the primary and secondary facilities. Such safety will be beneficial for construction of flow meter and flow collection-division box for future plant expansions.

$$\text{Water surface elevation in the collection box} = 124.05 \text{ m} + 0.30 \text{ m} = 124.35 \text{ m}$$

Step J: Water Surface and Other Elevations in the Primary Sedimentation Basin (Unit J). The pressure pipes connecting the effluent boxes of primary sedimentation basin with the collection-splitter box are 92 cm in diameter. Each pipe is designed to carry a peak design flow of 0.66 m^3/s from each sedimentation basin. In case one sedimentation unit is out of service, half of the flow is diverted to the aeration basin without primary treatment (see Sec. 12-6-2 for design criteria of primary sedimentation facility).

1. Compute head loss in the connecting pipe.

Velocity $= \dfrac{Q}{A} = \dfrac{0.66 \text{ m}^3\text{/s}}{(\pi/4)\,(0.92 \text{ m})^2}$

$= 0.99$ m/s

Exit loss ($K = 1.0$) $= \dfrac{1.0 \times (0.99 \text{ m/s})^2}{2 \times 9.81 \text{ /s}} = 0.05$ m

Minor loss (3–90° elbows, one elbow in vertical plane and two elbows in horizontal plane, $K = 0.3$) $= \dfrac{3 \times 0.3 \times (0.99 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2}$

$= 0.05$ m

Friction loss (from the Hazen-Williams equation, $C = 100$ and $L = 30$ m) $= 0.05$ m

Entrance loss at the effluent box of primary sedimentation facility ($K = 0.5$) $= \dfrac{0.5 \times (0.99 \text{ m/s})^2}{2 \times 9.81 \text{ m/s}^2}$

$= 0.02$ m

Assume miscellaneous losses $= 0.04$ m

Total head loss $= 0.05 \text{ m} + 0.05 \text{ m} + 0.05 \text{ m} + 0.02 \text{ m} + 0.04 \text{ m}$

$= 0.21$ m

2. Compute water surface and invert elevations in the effluent box of the primary sedimentation facility.

$$\text{Water surface elevation} = 124.35 \text{ m} + 0.21 \text{ m} = 124.56 \text{ m}$$
$$\text{Invert elevation} = 124.56 \text{ m} - 1.00 \text{ m} = 123.56 \text{ m}$$

3. Floor elevation of primary basin at the upstream end (Figure 12-19) = 123.56 m − 1.80 m = 121.76 m
4. Water surface elevation in primary basin at peak design flow condition = 121.76 m + 3.71 m (Figure 12-19) = 125.47 m
5. Compute water surface and other elevations in the primary sedimentation facility.

Head loss across primary sedimentation facility (Table 21-1)	= 0.98 m
Water surface elevation in the influent channel of the primary sedimentation tank at peak design flow	= 124.56 m + 0.98 m
	= 125.54 m

Step K: Water Surface and Invert Elevations in the Collection-Division Box (Unit K). The collection-division box receives flow from the grit removal facility and splits the flow equally into two pipes leading to the primary sedimentation basins. The flow is controlled by adjustable weirs. Sluice gates are provided to remove one sedimentation basin from service and divert the flow into the aeration basin through a third weir and bypass pipe (Figure 20-4).

1. Compute the head loss in the connecting pipe and water surface elevation in the outer chamber.

Diameter of the connecting pipe = 0.76 m

Peak design flow = 0.66 m^3/s

$$\text{Velocity} = \frac{Q}{A} = \frac{0.66\ \text{m}^3/\text{s}}{\pi/4 \times (0.76\ \text{m})^2}$$

$$= 1.46\ \text{m/s}$$

$$\text{Exit loss at unit J } (K = 1.00) = \frac{1.00 \times (1.46\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.11\ \text{m}$$

$$\text{Minor losses 1–90° elbow in vertical plane } (K = 0.3) = \frac{1 \times 0.3 \times (1.46\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.03\ \text{m}$$

$$\text{Minor losses 2–45° elbows in horizontal plane } (K = 0.2) = \frac{2 \times 0.2 \times (1.46\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.01\ \text{m}$$

$$\text{Entrance loss at unit L } (K = 0.3) = \frac{0.3 \times (1.46\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.03\ \text{m}$$

Friction loss (Hazen-Williams equation, $C = 100$, $L = 22$ m) = 0.08 m

Total head loss = 0.11 m + 0.03 m + 0.01 m + 0.03 m + 0.08 m = 0.26 m

Water surface elevation in the outer chamber	= 125.54 m + 0.26 m
	= 125.80 m

2. Compute the elevation of weir crest and water surface elevation in the collection-division box.

Free-fall allowance	= 0.68 m[c]
Elevation of the weir crest	= 126.30 m + 0.68 m = 126.48 m
Provide weir length	= 2 m
Head over weir from Eq. (11-3)	= 0.32 m
Water surface elevation	= 126.48 m + 0.32 m = 126.80 m
Depth of water in the box (assumed)	= 1.50 m
Invert elevation of the box	= 126.80 m − 1.50 m = 125.30 m

Step L: Water Surface and Other Elevations in Grit Chamber (Unit L). A 1.22-m-diameter pressure pipe connects the collection-division box (unit L) with the effluent box of the grit channel.

1. Compute the head loss in the connecting pipe.

Diameter = 1.22 m

Peak design flow = 1.321 m^3/s

$$\text{Velocity} = \frac{1.321\ \text{m}^3/\text{s}}{(\pi/4) \times (1.22\ \text{m})^2} = 1.13\ \text{m/s}$$

$$\text{Exit loss at unit J } (K = 1.0) = \frac{1.0 \times (1.13\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2}$$

$$= 0.07\ \text{m}$$

Minor loss (1–90° elbows in vertical plane, $K = 0.3$)
$$= \frac{1 \times 0.3 \times (1.13\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.02\ \text{m}$$

Friction loss (Hazen-Williams equation, $C = 100$, $L = 10$ m) = 0.01 m

Entrance loss at effluent box of grit chamber ($K = 0.5$)
$$= \frac{0.5 \times (1.13\ \text{m/s})^2}{2 \times 9.81\ \text{m/s}^2} = 0.03\ \text{m}$$

Miscellaneous losses = 0.03 m

Total head loss = 0.07 m + 0.02 m + 0.01 m + 0.03 m + 0.03 m
= 0.16 m

[c]A free fall of 0.68 m may seem high. The reason for providing such a high free fall is to build a sufficient safety factor because this collection-splitter box is a junction between the preliminary and primary treatment facilities. Such safety will be beneficial if a flow equilization basin or chemical precipitation facility is added in the future, or other units are added for future expansion.

2. Compute the water surface and other elevations in the grit chamber.

Surface elevation in the effluent box of grit chamber	= 126.80 m + 0.16 m
	= 126.96 m
Invert elevation of the effluent box	= 126.96 m − 1.5 m = 125.46 m
Depth of effluent trough	= 1.88 m (Figure 11-7)
Weir crest elevation	= 125.46 m + 1.88 m = 127.34 m
Head loss at peak design flow when one unit is out of service	= 0.45 m (Figure 11-7)
Water surface elevation at peak design flow when one unit is out of service	= 127.34 m + 0.45 m = 127.79 m

Water surface elevation in the influent channel of the grit removal facility at peak design flow when only one unit is in service = 126.96 m + 1.07 m = 128.03 m.
Invert elevation of grit chamber = 128.03 m − 4.06 m (Figure 11-7) = 123.97 m

Step M: Water Surface and Other Elevations in the Pumping Stations (Unit M). The pumping station includes wet well, dry well, valve fittings and specials, Venturi meter, and transmission line. The head loss calculations are performed in Chapter 9.

The following elevations are taken from Table 9-9, Figure 9-15, and Chapter 10.

Total discharge head at minimum wet well elevation	= 11.13 m
Total discharge head at maximum wet well elevation	= 9.91 m
Low-water surface elevation in wet well	= 128.03 m − 11.13 m = 116.90 m
High-water surface elevation in wet well	= 128.03 m − 9.91 m = 118.12 m
Floor elevation of wet and dry pit	= 116.90 m − 2.44 m = 114.46 m
Elevation of energy line at the pumps at maximum station capacity (Table 9-9)	= 118.12 m + 12.70 m
	= 130.82 m[d]
Elevation of energy line at the pump at minimum station capacity (Table 9-9)	= 116.90 m + 9.91 m = 128.20 m[d]

Step N: Water Surface and Other Elevations in the Bar Screen (Unit N). Hydraulic profile through the bar screen is given in Figure 8-6.

[d]A final check of the pumping head can be made at this point. At maximum and minimum station capacities, the total head losses in the force main should, respectively, be equal to or slightly less than the difference in elevation of energy line at station common header at maximum and minimum station capacities and the maximum water surface elevation in the grit chamber.

Floor elevation of the screen chamber	= 114.46 m + 4.82 m = 119.28 m
Water surface elevation below screen at peak design flow when one unit is out of service	= 119.28 m + 1.25 m = 120.53 m
Water surface elevation above screen at peak design flow when one chamber is out of service (clean screen)	= 120.53 m + 0.03 m = 120.56 m
Water surface elevation in the interceptor	= 120.54 m
Invert elevation of the interceptor	= 119.36 m

Step O: Water Surface and Other Elevations in the Junction Box (Unit O). The water surface and invert elevations in the connecting sewer lines and junction box are calculated in Chapter 7 (Figure 7-9).

Water surface elevation in the junction box (unit O)	= 120.55 m
Invert elevation in the junction box (unit O)	= 119.36 m
Invert elevation of sewer line i at the junction box	= 119.93 m
Water surface elevation in sewer line i at the junction box	= 120.59 m

All the above elevations in various units are given in Figure 21-1.

21-5 PROBLEMS AND DISCUSSION TOPICS

21-1 A clarifier is added to an aerated lagoon. The crest elevation of the effluent weir in the lagoon is 61.00 m. The water flows over the weir and drops into an outlet box. The water surface elevation in the outlet box is 0.8 m below the weir crest. A 20-cm pipe connects the outlet box and the influent well of the circular clarifier. The pipe is an inverted syphon and has two 45° and two 90° bends and a straight length of 68 m. Assume K for 45° and 90° bends; entrance and exit conditions are 0.2, 0.3, 0.15, and 1.0, respectively; $C = 110$. Determine the water surface elevation and total number of 90° V-notches in the final clarifier. Each V-notch is 20 cm deep and has a freeboard of 5 cm at peak design flow of 0.015 m^3/s.

21-2 The hydraulic profile in Figure 20-1 is prepared at peak design flow when the largest unit is out of service. Prepare a hydraulic profile at average design flow of 0.44 m^3/s when all treatment units are operating. Consult Problems 7-3, 8-1, 11-11, 12-5, 13-17, 13-18, 14-4, and 15-4 to obtain head losses through different treatment units under average flow conditions.

21-3 As a design engineer, you are required to prepare the piping layout and hydraulic profile of a wastewater treatment plant. Discuss various design considerations that are necessary in developing the piping layout and hydraulic profile through the treatment plant.

21-4 A wastewater treatment plant has the following treatment units: (a) bar screen; (b) velocity controlled grit channel; (c) primary sedimentation; (d) high-rate, single-stage trickling filter; (e) final clarifier; and (f) chlorination facility without the proportional weir. The connecting pipings between the units have head loss of approximately 20 percent of the average head loss across the upstream and downstream units. The outfall structure has the same head loss as the chlorination facility. Assume proper head losses across various treatment units, and justify your assumption with brief reasoning. Draw the hydraulic profile through the treatment plant having the pumping station after the primary treatment facility.

21-5 A wastewater treatment plant was designed to provide secondary treatment. The head losses in the bar screen, aeration basin, final clarifier, and chlorination facility at peak design flow were 0.06, 0.5, 0.8, and 0.6 m, respectively. Assume all units are directly connected by inverted syphon with 2–90° elbows in the connecting pipings between the units. If the diameter, straight length, and C of the connecting pipings are 20 cm, 50 m, and 110, respectively, draw the hydraulic profile through the plant. Assume that the outfall pipe has a submerged discharge. The high flood level in the receiving water is 100 m. The flow through all connecting piping is 0.02 m^3/s.

21-6 Obtain the hydraulic profile of the wastewater treatment plant in your community. Tabulate the following:

(a) Head losses in each treatment unit
(b) Head losses in the junction boxes
(c) Head losses in the connecting pipings

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22

Instrumentation and Controls

22-1 INTRODUCTION

Manual measurements of process control variables are often infrequent, may lack the desired precision, and often cannot be made within the short span of time necessary for proper operation of the facility. Therefore, the use of instrumentation and control in wastewater treatment facilities is increasing. This trend is not surprising because proper use of instrumentation and automatic controls can reduce labor, chemicals, and energy consumption, and improve treatment process efficiency, reliability, and data acquisition and recording.

The subject of instrumentation and controls is very complex, requiring an extensive background for those involved in design, analysis, and selection of instrumentation for wastewater treatment facilities. The purpose of this chapter is to present process control strategies, physical and chemical measuring elements and instruments, computers, and final control elements that are integral parts of the process control systems for a wastewater treatment facility. The type of instrumentation and control devices and simplified control loop diagrams selected for various treatment processes in the Design Example are also presented in this chapter.

22-2 BENEFITS AND NEED OF INSTRUMENTATION AND CONTROL SYSTEMS

The general motives for installing a sophisticated instrumentation and control system in wastewater treatment are numerous. The benefits may be in process improvement, equipment performance, and convenience to personnel. Many specific benefits are summarized in Table 22-1.[1,2]

Many factors are considered that influence the degree of process control sophistication needed at a facility.[3,4] Many of these include (a) size of the plant and future expansion needs; (b) complexity of the treatment processes and treatability aspects; (c) number of control points; (d) interdependence of process elements and controls, including process time delays; (e) number and capability of operating and maintenance personnel

TABLE 22-1 Benefits of the Instrumentation and Control Systems in Wastewater Treatment[1,2]

Purpose	Benefits
Process	• Improved process performance and better process results • Efficient use of energy • Efficient use of chemicals • Process changes detected in a timely manner • Automatic execution of corrective measures • Greater ability to control complex processes
Equipment	• Immediate alert signal of malfunction • Ability to diagnose problems in remotely located equipment before malfunction occurs • Status known at all times • Automatic execution of corrective measures and automatic response to potentially disastrous situations • Automatic shutdown to prevent major damage • Increase in running time
Personnel	• Timely and accurate process information • Safer operation • Efficient use of labor • Capability to quickly solve analytical problems • Minimize the potential for human error • Allows for an overview of plant operation • Decrease in manual paperwork • More complete records that may allow an overview of plant operation and plant behavior, and design of future expansion • Increased security

available, either owner-furnished or contracted from outside companies; and (f) useful life and process obsolescence caused by rapid technological advances and unavailability of replacement parts. Based on these factors, the selection of the appropriate method of control, data acquisition, and data display systems may be selected.

22-3 INSTRUMENTATION AND CONTROL SYSTEMS

A process control system utilizes (1) process variables and (2) associated controls.[5] The process variables are measured by the sensor of the control systems. The measured value is transmitted, displayed, or recorded to guide the operator in taking the proper corrective process adjustments. In other automatic control systems, the process variable signals

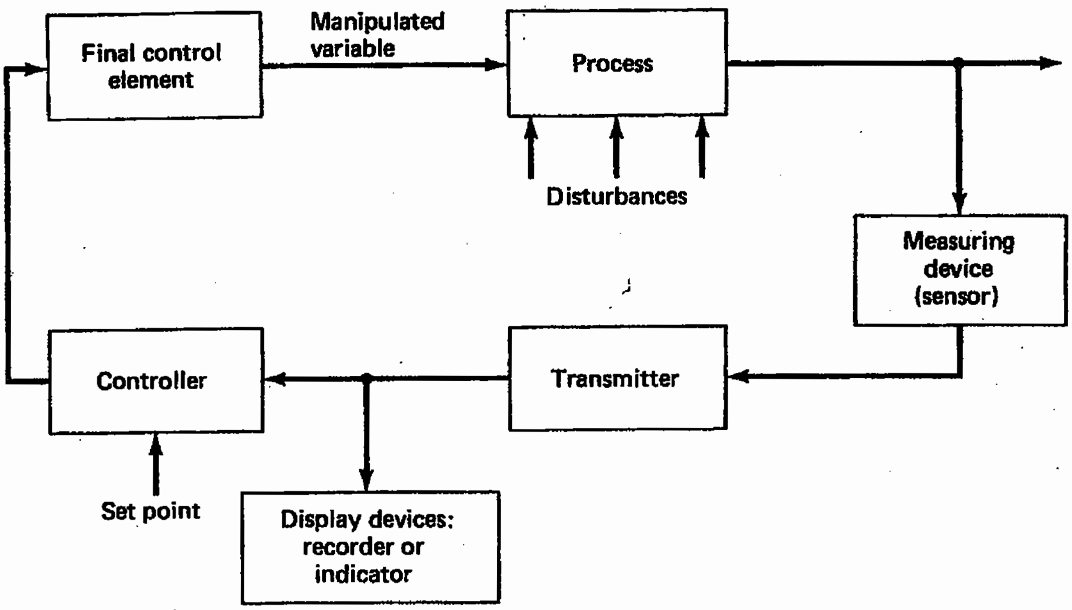

Figure 22-1 Typical Control System Components (from Ref. 5).

are compared with the value of the variables against a preset reference and then the controller implements the selected actions for the desired results. The control variables and the associated controls are discussed below. The typical control system components are illustrated in Figure 22-1.[5]

22-3-1 Control Variables

In a wastewater treatment plant, many variables are measured and controlled. These variables fall into three categories: physical, chemical, and biological. Common examples of these variables are listed below:

- physical: flow, pressure, level, temperature, etc.
- chemical: pH, oxidation-reduction potential (ORP), turbidity, specific conductance, dissolved oxygen, chlorine residual, etc.
- biological: oxygen consumption rate, TOC reduction rate, sludge growth rate, etc.

22-3-2 Associated Controls

The automatic control system is made up of six components.[5,6]

1. The measurement section to detect the change in the variables
2. Signal-transmitting device
3. Data display or readout
4. Control systems
5. Computer and central control room

TABLE 22-2 Common On-Line Process Measurement Devices and Their Applications in Wastewater Treatment

Measured Variables	Primary Device	Measured Signal	Common Application
Flow[a]	Venturimeter	Differential pressure	Gas, liquids
	Flow nozzle meter	Differential pressure	Gas, liquids
	Orifice meter	Differential pressure	Gas, liquids
	Electromagnetic meter	Magnetic field and voltage change	Liquids, sludges
	Turbine meter	Propeller rotation	Clean liquid
	Acoustic meter	Sound waves	Liquids, sludges
	Parshall flume	Differential elevation of water surface	Liquids
	Palmer-Bowlus flume	Differential elevation of water surface	Liquids
	Weirs	Head over weir	Clean liquids
Pressure	Liquid-to-air diaphragm	Balance pressure across a metal diaphragm	Pressure 0–200 kN/m^2
	Strain gauge	Dimensional change in sensor	Pressure 0–350,000 kN/m^2
	Bellows	Displacement of mechanical linkage connected to the indicator	Pressure 0–20,000 kN/m^2
	Bourdon tube	Uncurling motion of noncircular cross-sectional area of a curved tube	Pressure 0–35,000 kN/m^2
Liquid level	Float	Movement of a float riding on surface of liquid	Liquid head 0–11 m
	Bubbler tube	Measurement of back pressure in the tube bubbling regulated air at slightly higher pressure than the static head outside	Liquid head 0–56 m
	Diaphragm bulb	Pressure change in air on one side of the diaphragm caused by the liquid pressure on the other side of the diaphragm	Liquid head 0–15 m

	Ultrasonic	The sonic pulses are reflected from the target surface	Liquid level in channel or basin
Sludge level	Photocell	Detection of light in a probe by a photocell across sludge blanket	Primary sedimentation Final clarifier Gravity thickener
	Ultrasonics	Detection of the ultrasonic signal transmitted between two transducers	Primary sedimentation Final clarifier Gravity thickener
Temperature	Thermocouple	Current flows in a circuit made of two different metals	Anaerobic digester, hot-water boiler
	Thermal bulb	Absolute pressure of a confined gas is proportional to the absolute temperature	Sludge lines, water lines
	Resistance temperature detector	Change in electrical resistance of temperature-sensitive element	Bearing and winding temperatures of electrical machinery, anaerobic digesters, hot-water boilers
Speed	Tachometer (generator or drag cup type)	Voltage, current	Variable-speed pump, blower, or mixer
Weight	Weight beam, hydraulic load cell or strain gauge	Lever mechanism or spring, pressure transmitted across diaphragm, dimensional change in sensor	Chemicals
Density	Gamma radiation	Absorption of gamma rays by the liquid between radiation source and the detector	Mixed liquor suspended solids concentration; returned, thickened, and digested sludge solids

Continued

TABLE 22-2 Common On-Line Process Measurement Devices and Their Applications in Wastewater Treatment—cont'd

Measured Variables	Primary Device	Measured Signal	Common Application
Suspended solids	Ultrasonic sensor	Loss of ultrasonic signal by the liquid between the ultrasonic transmitter and receiver	Mixed liquor suspended solids concentration; returned, thickened, and digested sludge solids
pH	Selective ion electrode	Voltage produced by hydrogen ion activity	Influent, chemical solution, anaerobic digester, dewatering, effluent
Oxidation-reduction potential	Electrode	Change in potential due to oxidation or reduction	Influent, maintenance of proper DO in aeration basin, anaerobic digester
Total dissolved salts (solids)	Conductivity	Flow of electrical current across the solution	Influent, effluent
Dissolved oxygen	Membrane electrode	Electric current across the membrane because of reduction of molecular oxygen. Signal is temperature compensated.	Influent, aeration basin, plant effluent
Total organic carbon	Carbon analyzer	CO_2 produced from combustion of sample	Influent to the plant, influent to aeration basin, plant effluent
Chlorine residual	Sensor	Electrical output	Chlorine contact tank, plant effluent
Gases, NH_3, CO CO_2, H_2S, CH_4	Sensors	Various types of sensor modules utilize electrical impulses	Detection of hazardous condition in the room or around covered treatment units

Oxygen uptake rate	Respirometers using sensor	Decrease in DO level with respect to time	Aeration basin
Anaerobic biological condition	Sensor, combustion	CO_2 and CH_4 production rate	Anaerobic sludge digester
Phosphate	Spectrophotometer	Electrical signal proportional to degree of colored compound produced when stannous chloride reacts with a phosphate-molybdate complex.	Influent, effluent
Nitrate/Nitrite	Spectrophotometer	Electrical signal proportional to degree of color formed when nitrite and nitrate react with reagents to form colored compounds	Influent, effluent Dosing alkali to remedy acid production of nitrification
Ammonia	Spectrophotometer	Electrical signal proportional to degree of change that occurs when ammonia gas causes a change in the pH of an indication solution.	Influent, effluent

[a]Additional details may be found in Chapter 10. psi $\times$ 6.895 = kN/m^2.
Source: Adapted in part from Refs. 1, and 6–8.

Each of these components is discussed below.

Measurement Section or Sensing Devices. The sensing devices include instruments that sense, measure, or compute the process variables. The sensing device may be on-line or off-line, continuous or intermittent. Common on-line process measurement devices and their applications are summarized in Table 22-2.

Signal-Transmitting Devices. The purpose of the signal-transmitting device is to transmit the process variable signals from the sensing instrument to the readout device or a controller. The transmission may be accomplished mechanically, pneumatically, or electrically. Each type is discussed below.

Mechanical Transmission. The mechanical transmission is done by movement of a pen or indicator or by a float or cable. This method is generally limited to on-site display or location of controller.

Pneumatic Transmission. The pneumatic transmission system consists of a detector and an amplifier. The detector is a flapper-nozzle unit. Regulated input air is fed to the nozzle through a reducing tube. The pressure change is transmitted as the flapper is moved away from or toward the nozzle. Small changes in back pressure at the nozzle become proportional to the movement of the flapper. The pressure change is amplified and transmitted to the receiver or controller.

The advantage of pneumatic transmissions over electric transmissions include no electrical hazard, less effect of temperature and humidity, and no freezing problem. Also, they are reliable, less complicated, and easy to maintain. The limitations of pneumatic transmissions, however, are the signal lag in long tubing, the relatively short distance necessary between the detector and the controller (up to 300 m), the requirement of clean dry air, and air leakage problems. Detailed discussions on pneumatic transmission may be found in Refs. 9–11.

Electronic and Radio/Microwave Transmission. The electrical transmission of signals is achieved by voltage and current, pulse duration, or tone. In voltage and current transmission, the signals are transmitted by milliamp direct current (maDC) or by voltage signals. In pulse duration or time-pulse transmission, the length of time a voltage is transmitted is in proportion to the measured data. In tone transmission, standard telephone lines are normally used to allow signals to be transmitted. The signals are transmitted by turning the transmitter on and off or by slightly changing the frequency or pitch of the electronic tone.[6]

Recent advances in reliable radio and microwave equipment have encouraged the use of radio/microwave transmission where signals are transmitted over an assigned frequency per Federal Communications Commission (FCC) regulations. The radio/microwave transmission is particularly suited where the data-gathering points are scattered over a large area and where telephone lines are not available or are too expensive. Although radio/microwave transmission is expensive at present, its use will continue to grow, particularly for large systems.

Electronic and radio/microwave control systems are rapidly gaining popularity for a number of reasons. The basic advantages are as follows:[9] (1) the electronic signals operate over great distances without contributing time lags; (2) the electronic signals can easily be made compatible with a digital computer; (3) the electronic units can easily handle multiple signal inputs; (4) intrinsic safety techniques have virtually eliminated electrical hazards; and (5) electronic devices can take less space, are less expensive to install, can handle almost all process instruments, and can be essentially maintenance-free.

Data Display Readout. The transmitted information is displayed at a convenient location in a manner usable by the operating personnel. The most common type of readout devices are indicators, recorders, and totalizers on panels or computer screens. These devices may be driven mechanically from the sensing devices or from the pneumatic or electronic signals. The receiving instruments may display the movement, pressure change, or current change signals directly, or it may be servo-operated.[6]

Control Systems. Time control systems used in wastewater engineering can be divided into three categories: digital, analog, and automatic control. Each of these systems is described below.[1-3]

1. Digital: The digital systems have two positions. Some examples are on/off, open/closed, high/low, or alarm/normal. This type of signal is called status change. These signals may originate from a position, limit, float, or pressure switch.
2. Analog: The analog information has a range of values such as a flow rate for air or water. Level measurements or concentration measurements are also considered analog as a range of values may result. This type of data can be reused and transmitted in its analog form, or it may be converted to digital form or combinations of analog and digital instruments such as analog level meter with a high-level alarm switch.
3. Automatic Control: There are two basic types of automatic controls: discrete and continuous. The discrete control correlates equipment status (on/off) and status changes with a preset value or program of events. Discrete control may be represented by an automatic start/stop sequence such as pumping cycle based on wet well level. The discrete control may also be presented by a logically predefined program of events for a more complex process. The operation may be initiated manually by an operator using a pushbutton or automatically by an internal process-generated event. Programmable controllers are commonly used to perform discrete control tasks. The PC replaces multiple components (relays and timers) in hard-wired logic networks.

   The continuous control requires analog measurement for its input and manipulates a final control element as its output. Continuous control may utilize feedback and feed-forward control loops and control systems or controllers that perform the control functions. Many of these control loops and control systems are illustrated in Figure 22-2.[1]

Controlled and Uncontrolled Variables. The complexity of the control depends on factors such as treatment process, measurement required, frequency of information, envi-

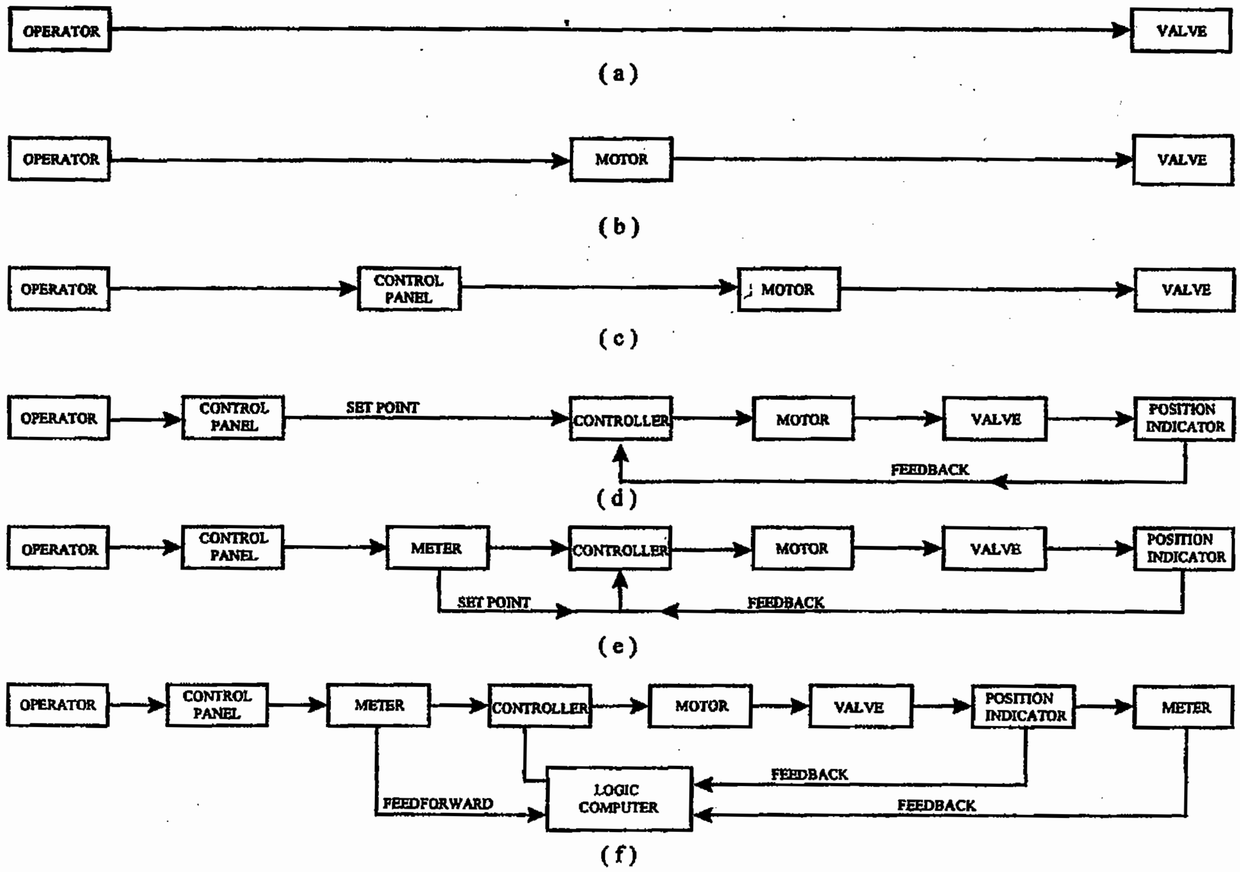

Figure 22-2 Examples of Control Systems: (a) manual control, (b) manual control made easier, (c) remote manual control, (d) automatic feedback control, (e) automatic feed forward control, and (f) automatic feed-forward–feedback computer control (from Ref. 1).

ronmental conditions, and technical limitations, maintenance manpower, and costs. In continuous process control, there are two types of process variables: (1) the uncontrolled variable (sometimes called the wild variable) and (2) the controlled variable. For example, the chlorine feed flow in the disinfection facility should be changed automatically to maintain a constant chlorine residual at the effluent weir of contact basin. In this control loop, the plant flow is the uncontrolled variable, and the chlorine feed becomes the controlled variable.[12]

Control Loops. A control system is a series of related or unrelated control loops for detection of variables that manipulate the process for desired results. A typical control system and its components are illustrated in Figure 22-1. An automatic control system is made of three parts:[1,5] (1) a measurement section to detect the change in the variable, (2) a reference source to compare the value of the variable with a reference value, and (3) a controller or mechanism to manipulate the variable until the measured signal reaches the reference value.

The control circuit or loop is composed of control elements and comes in two general types: closed or open loop. A closed loop or "feed-back loop" contains several automatic control units, and the process is connected to provide a signal path that includes a forward path, a feed-back path, and a summing point. The controlled variable is consistently measured, and if it deviates from the preset value, corrective action is applied to the final element to return the controlled variable to the desired value. An example of closed loop is an automatic chlorinator, as discussed in Chapter 14. An open loop or feed-forward loop is without a feedback system. The addition of chemical solution in proportion to flow is a good example of an open loop.

The control system consists of a series of individual control loops, each of which is relatively simple. However, these loops may be combined into independent, interdependent, or dependent open and closed control loops to constitute an elaborate and complex system. Additional discussion of control systems may be found in Refs. 6 and 9–12.

Continuous Control Systems, Controllers, and Functions. The device that automatically performs the control function in regulating the control variable is a controller. It may be of a different type, depending on the functional needs. Following is a brief description of some common types of controller functions. Detailed discussion may be found in Refs. 6, 9, and 10.

1. Two-Position or On-Off: The controller may be fully on or off. An example of such a system is the starting and stopping of a pump by a level controller.
2. Floating Control: Floating control is a variable on/off control with additional elements introduced: a second (negative) "on" position and a dead band; thus, all floating controls are three-position controls. Examples are air-conditioning systems having heating-off–cooling control action and valve systems with opening-off–closing action. Compared to two-position control, floating control allows tighter control (narrow range) and usually causes less cycling.
3. Proportional Control: Proportional control represents a continuous analog control where the output of the controller is directly proportional to the deviation from the setpoint. The degree of proportionality is called the proportional band. The process will be unstable if the proportional band is set too narrow. The proportional control is extensively used for filter level control in an effluent polishing plant.
4. Proportional-Integral: The proportional-integral mode (also called reset) is added to compensate for the deviation from the setpoint over the entire control range of the proportional band. The rate of output change is dependent on the amount and duration of the deviation (signal error). In essence, the integral function samples the difference between the fixed setpoint of the controller and the point at which the sensor stabilizes and adjusts the controller element in such a way as to return the parameter valve to the setpoint.
5. Proportional-Integral-Differential (PID): The proportional-integral-differential control mode adds the differential (derivative or rate) function. The purpose of the rate function is to modify the position of the controlled element in terms of the rate at which the parameter signal deviates from the fixed setpoint. These are, perhaps, the most elaborate single means of automatic process control systems.
6. Sample Data Control: The control action is delayed to provide a time lag for the effect of a previously applied correction to be sensed. An excellent example is a chlorination facility, where chlorine dosage is controlled by the residual measured after the contact period. Such a device is discussed in Chapter 14.

7. Timer Control: The timer control device operates on a set time schedule. An example of such a control system is pumping of waste activated sludge at preset time intervals (Chapters 12, 13, and 16–18).
8. Two- and Three-Mode Control: Two or three modes of control may be combined with the basic controls to achieve variable and complex control operations. They are commonly known as compound loops. A common example is chlorination followed by dechlorination.

22-3-3 Computers and Control Room and Data Acquisition System

Computers. Most of the medium-sized wastewater treatment plants have data acquisition systems for data logging.[1] These systems accumulate, format, record, and display large quantities of data effectively. Modern data acquisition systems, commonly referred to as supervisory control and data acquisition (SCADA) systems, can provide accurate, impartial documentation of all process measurements and operator actions. Computers can be used to develop a maintenance schedule for the plant operator based on actual operating time of a particular piece of equipment, such as a pump or motor. SCADA systems accumulate the process data, display them, and also produce the necessary process corrections for optimum operation. These corrections may include control of chemical solutions, air supply, scheduling of pumps and blowers, and the like. Although SCADA systems are in common use in many industries, they are now gaining wide application in small- and medium-sized wastewater treatment works. SCADA systems that are poorly designed or those with no previous application, such as a newly designed system, can potentially cause numerous problems.

Central Control Room. Central control is used to organize the plant operation in such a manner that all treatment information, important events, and alarms are displayed, indicated, and recorded at a centralized location. This location is called the central control room. In addition, most central facilities practice automatic or manual actuation of final control elements. Central control rooms reduce the number of personnel required to operate a large treatment facility. A graphic display on computer screen of a filter unit in operation is shown in Figure 22-3.

Expert System. With technology in situations where new process control strategies are being implemented or during new plant startup where the operators are relatively inexperienced with the particular plant, Expert Systems (ES) have been useful. ES technology captures with a computer program, the concepts, facts, and judgments used by an expert (experienced operator) in making a decision. Such decision-making applications often involve "heuristics," or rules of thumb, that express discrete elements of the expert's knowledge. These rules provide the basis for constructing an interactive dialogue with the program user to meet the information needs.[13-17] The expert systems are also used to assist the operators of smaller plants who have limited background about the complicated physical, chemical, and biological treatment processes.

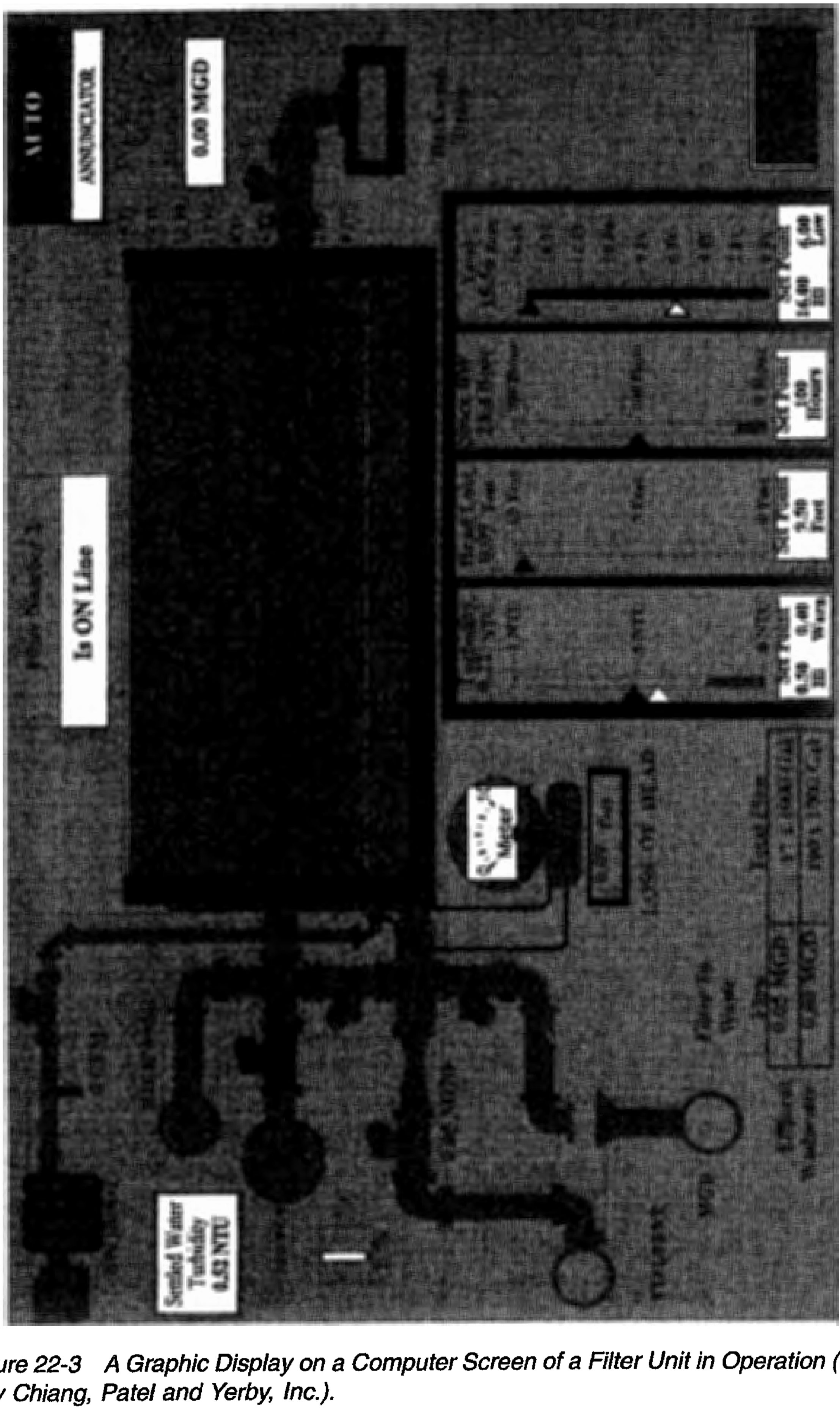

Figure 22-3 A Graphic Display on a Computer Screen of a Filter Unit in Operation (courtesy Chiang, Patel and Yerby, Inc.).

Fuzzy Control Systems. The next step in computer control systems is in the use of expert systems to control plant operations. This process, which is called fuzzy control system, expresses experts' operation methods with IF-THEN control rules. The fuzzy reasoning is performed by inputting values of required parameters or by remote sensing of needed data from the wastewater treatment process. The program executes the data and performs feed-back control of the process, which again is monitored. Fuzzy control systems have been used to control the chemical needs for odor control. When working properly, fuzzy controls permit minute and smooth control of processes, reduce operator workloads, reduce chemical usage, reduce energy demand, and improve overall system performance.[18]

Neutral Networks. One of the limitations of current expert systems technology is that they are unable to *learn,* in the sense that they cannot update their own knowledge bases. One technique for providing learning capability within computer systems is the neural network approach.[13] Neural networks are systems that attempt to model the networks of neurons in the brain in a manner that allows the computer to recognize patterns and to modify its own recognition criteria, such that it can be updated to improve its performance. A neural network consists of a series of arcs and nodes, the nodes representing neurons and the arcs representing axons and axon terminals of the neurons.

In the training of networks, the user supplies the system with a set of input data and a set of expected results or output data. The system *learns* by adapting its internal pattern recognition formulations (the weight factors for each of the node-to-node connections). Once the system has been trained, it solves problems by taking inputs, matching them as closely to its previous experience, thus providing a result that is as close to the expected solution as it can achieve.[13,19-21]

Applications of neural networks have not yet been extensively developed because of the very recent commercialization of this technology, but applications have already started to appear in the wastewater field. Neural networks could potentially be used to learn about plant behavior for diagnosis, prediction, alarm prioritization, chemical analysis, and other applications. Neutral networks and machine learning, in general, will be the source of much research in the future.[13]

22-3-4 Symbols and Identification of Process and Instrumentation Diagram (P&ID)

The symbols and identification used in designing the P&IDs are based on those provided by the Instrumentation Society of America (ISA), the Institute of Electrical and Electronic Engineers (IEEE), and the National Electrical Manufacturers' Association (NEMA). Commonly used symbols and identification systems may be found in Ref. 22.

22-4 MANUFACTURERS AND EQUIPMENT SUPPLIERS OF INSTRUMENTATION AND CONTROL SYSTEMS

The names and addresses of many manufacturers and equipment suppliers of instrumentation and control systems for wastewater treatment facilities are provided in Appendix

D. Important considerations for selection of equipment suppliers and the responsibility of the design engineer in providing data and making selection decisions are discussed in Sec. 2-10.

22-5 INFORMATION CHECKLIST FOR DESIGN AND SELECTION OF INSTRUMENTATION AND CONTROL SYSTEMS

Specific uses of instrumentation and the extent of automation provided at a wastewater treatment facility are decisions that must be made by the design engineer and management personnel. The extent of instrumentation may depend on the size of the plant, number of operating personnel, and available construction funds. The following information must be developed and decisions made for the selection of proper instrumentation and control systems.

22-5-1 General

1. Type of treatment process and size of the plant
2. Current design and effluent standards
3. Useful life and process obsolescence caused by rapid technological advances and unavailability of replacement parts
4. New plant or modifications to an existing plant
5. Hours of manned operation daily
6. Based on the size and complexity of the plant, should there be no control, manual control, supervisory control, automatic control, or computer control?
7. Availability of trained instrumentation maintenance personnel
8. Backup system for process monitoring or control if the instrumentation or whole system fails or must be taken out of service for repairs or preventive maintenance
9. If automatic controls are selected, decisions should be made concerning the number of control elements and loops, accuracy needed, operating range, response time of process variables, and frequency of operator input.
10. The economics of instrumentation and controls must be compared with the savings achieved by plant automation.
11. Types of vendor-supplied controls available and how these systems will interface with other control systems
12. Off-site operation and control capability (remote operation)

22-5-2 Design of the Instrumentation and Control (I&C) System

The design of the instrumentation and control system must be conducted by I&C engineers. However, the project engineer should maintain an open line of communication with the I&C engineers and perform a thorough check of the final design. Since the system must be operated and maintained by the plant operators, it is beneficial to elicit input from both the operators and the owner during the design phase.[2] The design work usually produces the following documents in the sequence listed below. The process flow sheets (PFS) are prepared by the project engineer. This flow sheet then becomes the basis for the P&ID.[2]

- P&ID is a functional schematic presentation of the treatment processes and the required instruments and controls. Its purpose is to illustrate the functions of the treatment plant without referring to the actual hardware.
- Process control diagram (PCD) specifies the monitoring and control loops.
- Instrumentation and input-output summaries (IIOSs) provide continuity between the P&ID and IIOS. These documents specify both the quantity and the characteristics of the analog and digital instrumentation.
- Instrumentation specification sheets (ISSs) provide detailed specifications for each task and panel instrument.
- Logic diagrams for the control panels and programmable logic controls (PLCs) are obtained by the instrumentation contractor.
- Panel layout drawings represent the schematic layout of P&ID.
- Loop interconnection drawings (LIDs) are primarily used in projects where the instrumentation and control systems must be interfaced with existing control equipment.
- Instrument installation details (IIDs) represent final installation details.

22-6 DESIGN EXAMPLE

22-6-1 Design Criteria Used

The basic design criteria for the instrumentation and controls selected for many treatment processes have been discussed in respective chapters. The criteria are based on providing an economical and reliable operation of various treatment facilities. These include (1) operational range, (2) accuracy desired, (3) operational flexibility and reliability, (4) space requirements, and (5) future expansions. The selected process systems incorporate proven industrial control equipment and techniques of contemporary design.

22-6-2 Selected Process Control Systems

The selected process control system will consist of the following: (1) It shall contain automatic and continuous analyzers of sensing devices at strategic locations throughout the plant for measurement of process control variables. Where a particular process control variable cannot be monitored, a systematic grab-sampling device shall be installed for piping samples directly into the laboratory. (2) The automatic control systems will be used to offer a high degree of accuracy and permit remote control of process equipment. The instrumentation will sense flows, absolute pressure differential, and temperature. The majority of the control equipment will be solid-state electronic. (3) The central control room shall contain analog indicating controllers, annunciator alarm panels; motor control and valve operator switches; trend recorders, indicators, and totalizer; converters; transducers; operator's control console; master communication equipment; telemetry transmitters and receivers; auxillary power supply for emergency use and data-logging systems; and so on. The heating and air-conditioning for this room will be designed to

Text continued on page 880.

TABLE 22-3 Brief Description of Various Functions Monitored for Different Treatment Processes in the Design Example

Treatment Unit	Sensing Device	Purpose	Display Local	Display Central	Description	Ref.
Bar screen	Limit switch	Monitor gate position		x	Display green light	Secs. 8-3-3 and 8-8-6
	Time clock	Operation of rake		x	Display green light	
	High-level override by float in stilling well (excessive head loss)	Emergency operation of rake		x	Display red light	
Pumping station	Bubbler system in wet well	Control pumping rate	x	x	Display of liquid level in wet well	Sec. 9-11-2 and Table 9-12
	Conductivity switch or float	Liquid level in sump of dry well		x	Trip alarm	
	Limit switches	Monitor gate position		x	Display green light	
	Pressure switch	Monitor seal water pressure for pump protection		x	Display red light	
	Flow switch	Monitor seal and water flow to pump for pump protection		x	Display red light	
	Pressure indicator	Monitor the discharge pressure of the pumps	x	x	Pressure gauges	
	Motor monitors (contacts provided in motor control center)	Monitor on-off condition of each major motor, and motor malfunction	x	x	Display green light, red light for malfunction	
	Liquid level sensor in dry pit	Leakage and flooding	x	x	Display red light, trip alarm	Sec. 9-9-2, Step E, 6

Continued

TABLE 22-3 Brief Description of Various Functions Monitored for Different Treatment Processes in the Design Example—cont'd

Treatment Unit	Sensing Device	Purpose	Display Local	Display Central	Description	Ref.
	Temperature monitor and/or amperage monitor	Motor protection against overheating and burn out		x	Display red light	Secs. 9-10-1 and 9-10-2
	Sampling and analysis	Sensors for pH, H_2S, CH_4, DO. Sampler to deliver water sample in the lab		x	Continuous recording on strip chart Routine chemical analyses. TSS, BOD, TOC, P, N	
	Main circuit breaker, auxiliary contact	Monitor power outage		x	Display red light (standby power required)	
Flow measurement	Venturi tube in force main	Monitor influent flow to the plant		x	Display, record, and totalize continuously	Sec. 10-9
Aerated grit chamber	Orifice plate with a square root extractor and display	Measure air flow to each chamber	x	x	Meter, display, record, and totalize flow continuously	Secs. 11-12-3
	Pressure indicators	Monitoring of discharge pressure of air blowers	x		Pressure gauges	
	Motor monitors (contacts provided in motor control center)	Monitor on-off condition of blower motor	x	x	Display green light	
	Torque monitors	Monitor spiral conveyor and bucket elevator	x	x	Display red light	
Primary sedimentation	Level detector	Control sludge and scum pumps and air agitator in scum box	x	x	Display green light	Secs. 12-4-9 and 12-4-10

	Pressure indicator	Monitor discharge from each sludge pump	x	x	Display red light	Sec. 12-8-2, Step F
	Magnetic flow meter	Sludge-pumping rate		x	Display, record, and totalize continuously	Sec. 12-10-5
	Motor monitor (contacts provided in motor control center)	Monitor on-off condition of each motor		x	Display green light	Secs. 12-10-3, 12-10-4, and 12-10-7
	Torque monitor	Monitor motor torque for sludge-raking mechanism		x	Alarm signal	Secs. 12-10-3 and 12-10-4
Anaerobic zone	Torque monitor	Monitor mixer motor for operation	x	x	Display red light	Sec. 13-11-7, Step A, 2 Sec. 13-13-2
	Speed regulator	Monitors and controls rotational speed of mixer blades		x	Display rotational speed	Sec. 13-11-7, Step A, 2 Sec. 13-13-2
	Thermocouple	Monitor temperature,		x	Display and record	Sec. 13-11-5, Step A
	Selective ion electrode	pH, ORP		x	Display	Sec. 13-11-5, Step A
Anoxic zone	Torque monitor	Monitor mixer motor for operation	x	x	Display red light	Sec. 13-11-7, Step A, 2 Sec. 13-13-2
	Speed regulator	Monitor and controls rotational speed of mixer blades		x	Display rotational speed	Sec. 13-11-7, Step A, 2 Sec. 13-13-2
	Thermocouple	Monitor temperature,		x	Display and record	Sec. 13-11-5, Step B
	Selective ion electrode	pH, DO, NO_3		x	Display and record	Sec. 13-11-5, Step B

Continued

TABLE 22-3 *Brief Description of Various Functions Monitored for Different Treatment Processes in the Design Example—cont'd*

Treatment Unit	Sensing Device	Purpose	Display Local	Display Central	Description	Ref.
Aerobic Zone and final clarifiers	Dissolved oxygen measurement	Control aeration rate		x	Display and continuous recording	Sec. 13-11-7, Steps C and D
	Flow meter, differential pressure type	Monitor air supply to each aeration basin	x	x	Meter display record and totalize flow continuously	Sec. 13-11-7, Step C
	Ultrasonic sludge blanket in final clarifier	Monitor the sludge level and actuate sludge return pumps		x	Display pump operation	Sec. 13-11-8, Steps G and H
	Magnetic flow meter	Control return sludge flow	x	x	Display, record, and totalize flow continuously	Sec. 13-11-8, Step H
	Suspended solids analyzer	Measure MLSS concentration in aeration basin and actuate waste sludge pumps	x	x	Display and record TSS, record and totalize flow	Sec. 13-11-7, Step E
	Pressure indicator	Monitor the discharge pressure of the pumps	x	x	Pressure gauges	Sec. 13-11-7, Step E
	Motor monitors (contacts provided in motor control) torque monitor	Monitor on-off condition of each motor		x	Display green light	Sec. 13-11-8, Step G
	Torque monitor	Monitor motor torque for sludge-raking mechanism, RAS pumps and blowers		x	Alarm signal	Sec. 13-11-8, Step G

UV disinfection system	Parshall flume	Measure flow upstream of UV disinfection facility to control light intensity	x	x	Display, record, and totalize flow continuously	Sec. 14-12-3, Step B
	Electrical power module	Monitor electrical power to UV lamp	x	x	Display red light	Sec. 14-15-1
	Thermocouples	Monitor temperature of UV lamp module	x	x	Display red light	Sec. 14-15-1
	Lamp status	Monitor burnt out lamp	x	x	Display red light	Sec. 14-15-1
	UV light intensity sensor	Measure UV light intensity	x	x	Display light intensity and UV dose alarm	Sec. 14-15-1
	Totalizer	Totalize time for each lamp/row/bank		x	Display total hours in operation	Sec. 14-15-1
	Level control	Monitor liquid level upstream and downstream of UV banks	x	x	Display and record the level. Sound alarm for low and high levels	Sec. 14-15-1
Gravity thickener	Magnetic flow meter	Flow measurement of feed sludge	x	x	Display, record, and totalize flow	Sec. 16-9-2, Step C, 6, and Figure 16-7
	Venturi meter	Flow measurement of dilution water	x	x	Display, record, and totalize flow	Sec. 16-9-2, Step A, 3
	Ultrasonic sludge blanket detector	Monitor the sludge blanket level in the gravity thickener	x	x	Display sludge blanket level	Sec. 16-9-2, Step G
	Ultrasonic density meter	Monitor solids concentration in feed and thickened sludge	x	x	Display and record	Sec. 16-9-2, Step G
	Pressure indicator, motor monitor, torque monitor	Similar to those for primary and final clarifiers	x	x	Similar to those for primary and final clarifiers	Sec. 16-9-2, Step C

Continued

TABLE 22-3 Brief Description of Various Functions Monitored for Different Treatment Processes in the Design Example—cont'd

Treatment Unit	Sensing Device	Purpose	Display		Description	Ref.
			Local	Central		
Anaerobic digestion	Magnetic flow meter	Flow measurement of feed sludge	x	x	Display, record, and totalize flow	Sec. 17-8-2, Step E, and Figure 17-10
	Ultrasonic density meter	Monitor solids concentration in feed and digested sludge, and mixing	x		Similar to those for gravity thickeners	Sec. 17-9-2
	Thermocouple, selective ion electrodes	Monitor temperature, pH, and ORP of digester content and the recirculating sludge		x	Display and record	Sec. 17-8-3
	Level indicator	Monitor liquid level in the digester and level of floating cover	x		Indicator	Sec. 17-9-1
	Pressure and temperature	Control hot water heating system	x	x	Display temperature, pump operation	Sec. 17-9-3
	Orifice meter	Measurement of digester gas flow		x	Display, record, and totalize	Sec. 17-8-2, Step I
	Pressure gauge	Gas storage pressure		x	Red light and alarm	Sec. 17-8-2, Step I, 2
	Flame detector	Monitor flame in building, flare status		x	Red light	Sec. 17-8-2, Step I
	Chromatograph and calorimeter	Determination of gas composition and calorific value		x	Tabulation of result and data logging	
	Venturi meter	Measurement of supernatant flow		x	Record and totalize	Sec. 17-8-2, Step E

	Pressure switch, flow switch pressure indicator, motor monitor	Monitor pump, motor, compressor as indicated above	x	x	Display red light	Secs. 17-9-2 and 17-9-3
Filter press	Pressure indicators	Measurement of pressure and shearing forces exerted by belt filters	x	x	Dial display	Sec. 18-8-4
	Belt speed	Frequency drive controller for belt speed adjustment	x	x	Display speed	Sec. 18-8-4
	Magnetic flow meter	Monitor the flow of conditioned feed sludge to each belt filter press		x	Record and totalize	Sec. 18-6-2, Step C, and Sec. 18-8-3
	Ultrasonic density meter	Monitor solids concentration in feed sludge		x	Record and totalize	Sec. 18-6-2, Step C
	Static scale	Measure weight of sludge cake	x		Record and log data	Sec. 18-6-2, Step C
	Ohm meter, lab test	Determine moisture content of the sludge cake	x		Record and log data	Sec. 18-6-2, Step C
	Float and level indicator	Measure level of polymer solution in storage and feed tanks	x		Record and log data	Sec. 18-6-2, Step C
	Magnetic flow meter	Measure flow of lime and polymer solution	x		Record and log data	Sec. 18-6-2, Step C
	Pressure switch, flow switch, pressure indicator, motor monitor	Monitor pump, motor, compressor, as indicated above	x		Display as indicated above	Sec. 18-6-2, Step C

maintain a uniform temperature and to provide a clean atmosphere with a stable relative humidity. (4) Various control and analyzer panels shall be located throughout the plant to provide the data needed to optimize the plant efficiency.

Each of the process unit and major equipment shall be monitored and, to some extent, controlled from the central control room. The general descriptions of various functions to be monitored for each treatment process are summarized in Table 22-3. A total of 11 simplified loop diagrams are shown in Figure 22-4. These loop diagrams show the operational principles of (1) differential head control for bar screen, (2) differential pressure control for flow measurement, (3) bubbler tube control in wet well for pump operation, (4) blower control for aeration basin, (5) air flow monitoring in grit chamber, (6) scum pump control in primary sedimentation basin, (7) dissolved oxygen monitoring in the aeration basin, (8) return sludge control, (9) waste activated sludge control, (10) post chlorination control, and (11) UV disinfection control.

P&ID diagrams for many other applications, such as thickened sludge withdrawal; sludge recirculation, mixing, and heating; chemical feed; and so on can be easily developed from the examples of 11 loops shown in Figure 22-4. It should, however, be mentioned that these loops are grossly simplified. Many complex loop diagrams are generally needed in the design of wastewater treatment facilities. The instrumentation engineer should develop such diagrams in consultation with the manufacturers of various instrumentation and control devices and the manufacturers of pollution control equipment.

22-7 PROBLEMS AND DISCUSSION TOPICS

22-1 A Venturi meter is used for continuous flow recording of industrial wastewater. The differential pressure is transmitted and recorded on a chart. Using Eq. (10-6), calculate the hourly flows and totalize the flow. Use the following data:

Time:	12 midnight	1	2	3	4	5	6	7	8	9	10	11
h, m H_2O:	0.20	0.21	0.25	0.26	0.30	0.40	0.50	0.70	1.20	1.80	2.20	2.30

Time:	12 noon	1	2	3	4	5	6	7	8	9	10	11	12 midnight
h, m H_2O:	2.00	1.80	1.60	2.10	2.80	3.00	2.50	2.00	1.80	0.80	0.40	0.30	0.20

22-2 An industrial plant operates from 8 A.M. to 5 P.M. The hourly wastewater flow and TOC concentrations are given below. Totalize the TOC results and express in kilograms per operating day.

Time:	7	8	9	10	11	12	1	2	3	4	5	6	7
Flow, m^3/s	0	0.02	0.05	0.10	0.15	0.20	0.21	0.22	0.25	0.15	0.10	0.02	0
TOC, mg/L	0	80	120	180	300	350	300	280	250	200	150	80	0

22-3 Using the typical control system components given in Figure 22-1, list various options available at each control system component for maintaining constant temperature in an

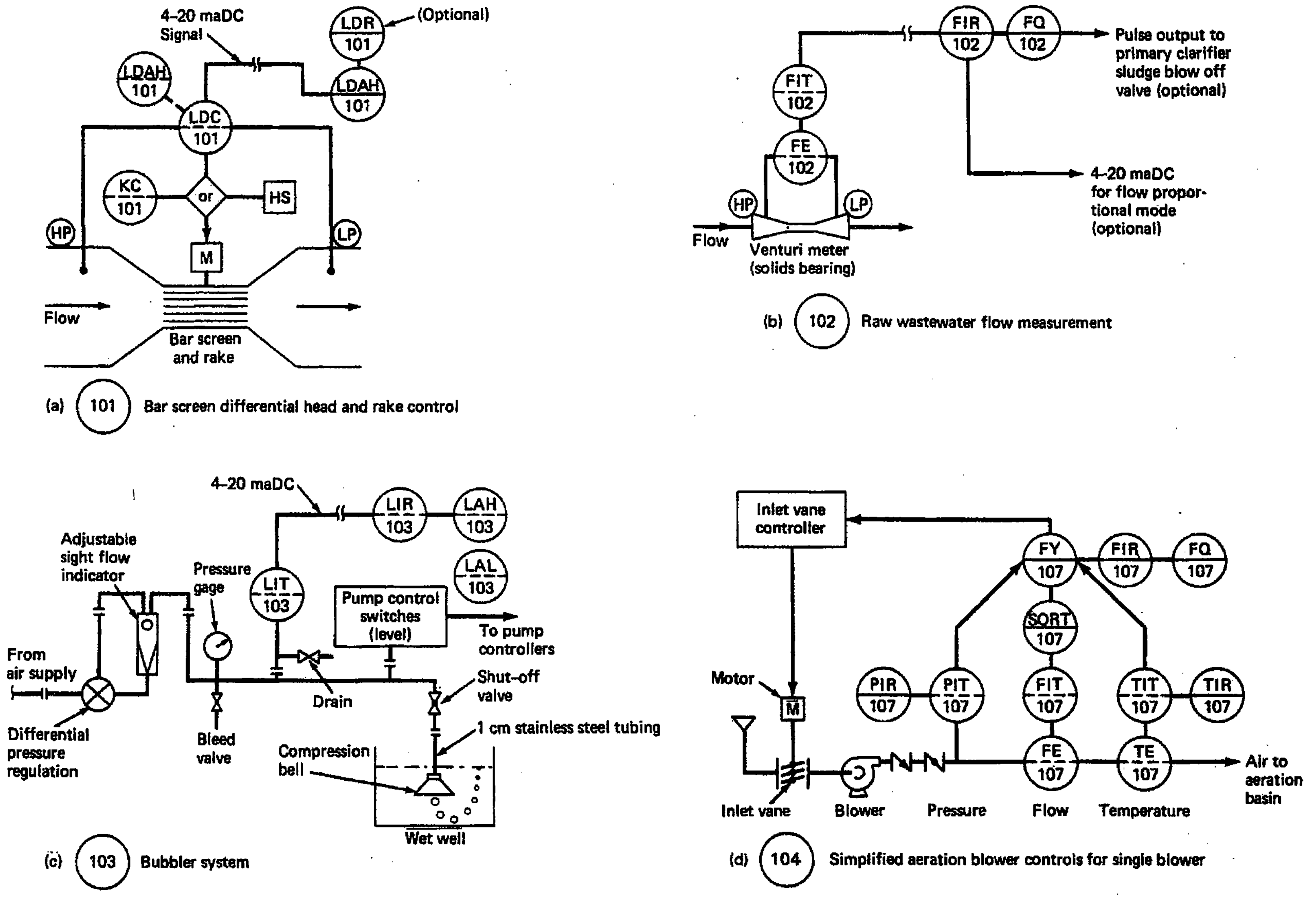

Figure 22-4 Simplified Control Loop Diagrams for Different Treatment Processes Designed in the Design Example.
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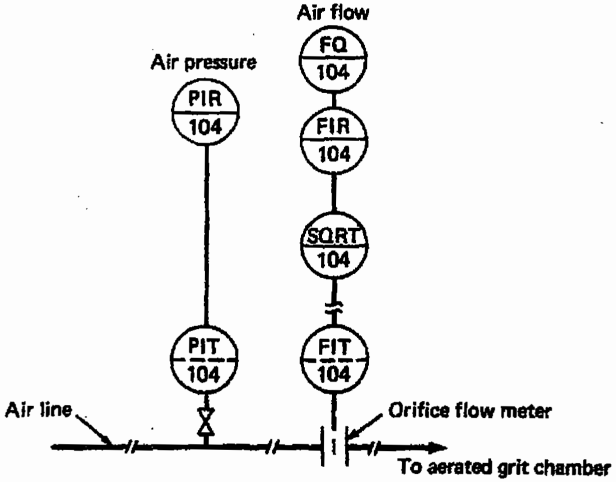
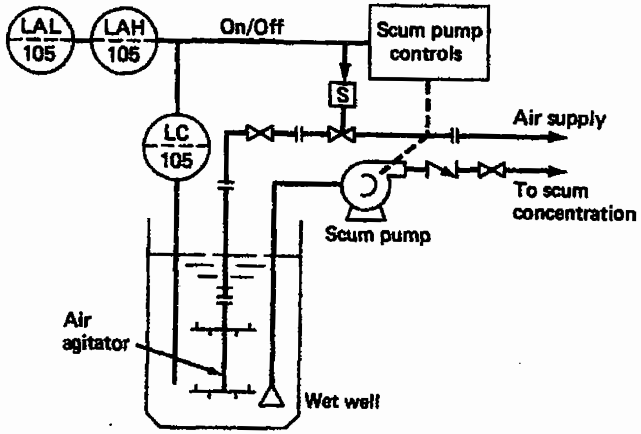
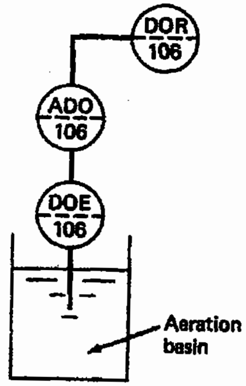


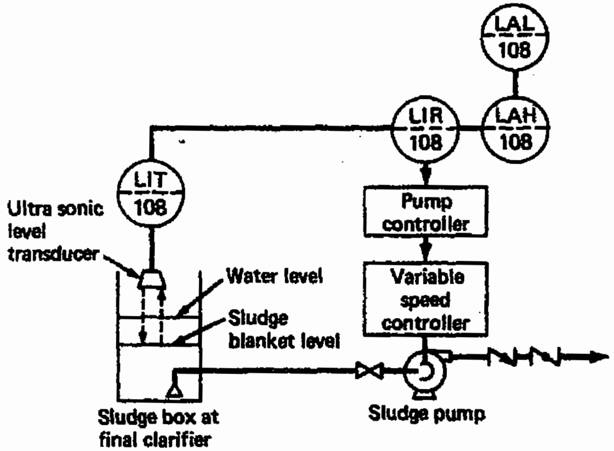
(e) 105 Air flow monitoring to grit chambers



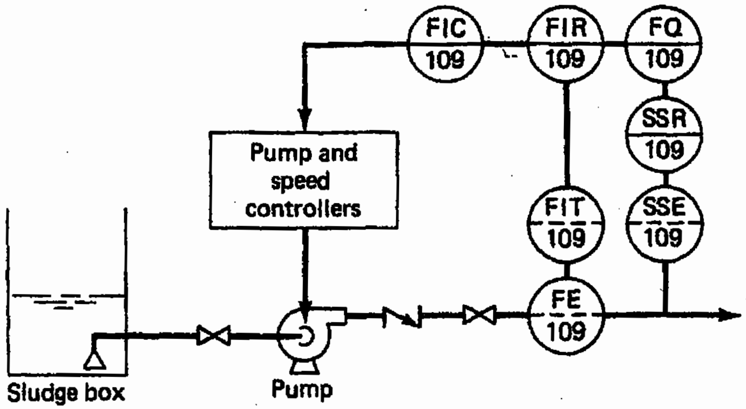
(f) 106 Scum pump and level control



(g) 107 Dissolved oxygen monitoring in aeration basin

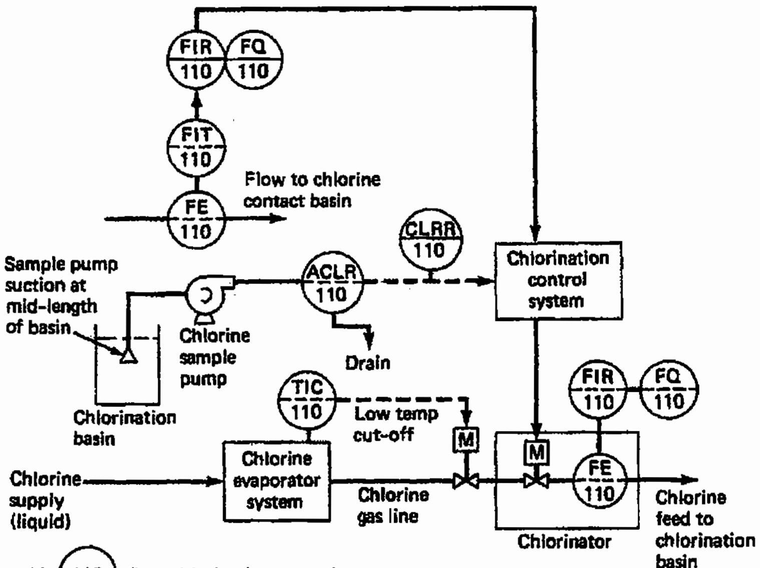


(h) 108 Sludge return pump controls

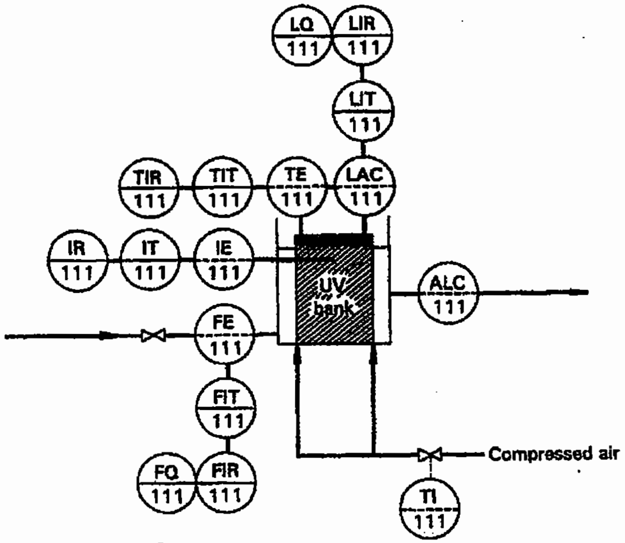


(i) 109 Waste activated sludge controls

Figure 22-4—cont'd



(j) 110 Post chlorination controls



(k) 111 UV disinfection

Figure 22-4—cont'd

- ALC Automatic water level controller
- ACLR Chlorine residual analyzer
- CLRR Chlorine residual recorder
- ADO Dissolved oxygen analyzer
- DOE Dissolved oxygen element
- DOR Dissolved oxygen recorder
- FE Flow element
- FIC Flow indicating controller
- FIR Flow indicating recorder
- FIT Flow indicating transmitter
- FQ Flow totalizer
- FY User's choice
- HP High pressure
- HS Hand switch
- KC Percentage timer
- IE UV intensity element
- IT UV intensity recorder
- IR UV intensity transmitter
- LAC UV lamps controller
- LAH Level alarm high
- LAL Level alarm low
- LC Level controller
- LDAH Differential level high alarm
- LDC Differential level control
- LDR Differential level recorder
- LIR Level indicating recorder
- LIT Level indicating transmitter with P/I (pneumatic/current transducer)
- LP Low pressure
- M Motor and controller
- PIR Pressure indicating recorder
- PIT Pressure indicating transmitter
- S Solenoid
- SQRT Squire root extractor
- SSE Suspended solids element
- SSR Suspended solids recorder
- TE Temperature element
- TIC Temperature indicating controller
- TIR Temperature indicating recorder
- TIT Temperature indicating transmitter
- Solid line denotes instrument mounted in central control panel
- Dashed line denotes instrument mounted in the field (local)

anaerobic digester. The sludge is recirculated through external heat exchanger for heating purposes.

22-4 Match the primary flow measuring devices with their most suitable applications.

Orifice meter	(a) Sludge
Flow nozzle	(b) Clean liquid
Parshall flume	(c) Raw wastewater
Electromagnetic meter	

22-5 Write measured signals for the following variables:
(a) Conductivity
(b) pH
(c) Density
(d) Oxidation reduction potential
(e) Dissolved oxygen
(f) Temperature
(g) Pressure

22-6 A process diagram of a BNR treatment process is given in Figure 13-24. Draw an integrated process mechanical and instrumentation diagram for an aeration basin and final clarifier. Assume that there are single units of aeration basin, final clarifier, return sludge pump, waste sludge pump, blower, and air meter. Use the simplified control loop diagrams for different processes given in Figure 22-4.

22-7 Discuss various types of signal-transmitting devices. Give advantages and disadvantages and applications of each type.

22-8 What is the basic difference between feedback and feed-forward automatic control loops? Give three examples of each in wastewater treatment plants.

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23

Design Summary

In this book, the step-by-step design procedure for a medium-sized wastewater treatment facility has been developed. One Design Example has been carried through 17 chapters (Chapters 6–22) to present the theory, design procedure, operation and maintenance, and equipment specifications for various components of the wastewater treatment facility. The purpose of this chapter is to consolidate the basic design data that have been developed in the Design Example. The design data are summarized in Table 23-1. Reference has been made to various sections in which the details may be found.

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
A. *Existing Wastewater Treatment Facility*		Chap. 6
The existing facility includes three small plants: (1) trickling filter on the east side, (2) stabilization pond on the west side, and (3) aerated lagoon on the northern end of town.		Table 6-4
Current sewered population served	46,750	Table 6-6
Current unsewered population	1,250	Table 6-6
Average flow to trickling filter, L/s	67	Table 6-6
Average flow to stabilization pond, L/s	95	Table 6-6
Average flow to aerated lagoon, L/s	80	Table 6-6
Total flow treated, L/s	242	
B. *Proposed Facility*		Chap. 6
The proposed facility includes construction of two gravity intercepting sewers to divert flows from the trickling filter plant and stabilization pond area to the aerated lagoon site. Construct a new treatment plant at this location to treat the combined flows. The process		Sec. 6-10

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
train includes bar screen, pumping station, grit removal, primary sedimentation, enhanced biological nutrient removal (BNR), disinfection, and river outfall. The sludge treatment includes gravity thickening of combined sludge, anaerobic digestion, and dewatering by belt filter presses.		
Initial year	2000	Table 6-9
Design year	2015	Table 6-9
Design flows		Table 6-9
Average wastewater flow, Lpcd	475	Table 6-9
Average flow, L/s	440	Table 6-9
Peak flow, L/s	1321	Table 6-9
Minimum flow, L/s	220	Table 6-9
Influent characteristics		
BOD_5, mg/L	250	Table 6-9
TSS, mg/L	260	Table 6-9
TS, mg/L	910	Table 6-9
pH	7.2	Table 6-9
Total P, mg/L	6.0	Table 6-9
NH_4^+-N, mg/L	19.0	Table 6-9
Org.-N, mg/L	17.0	Table 6-9
Total N, mg/L	36.0	Table 6-9
Effluent standards		Table 6-1
C. *Intercepting Sewer*		Chap. 7
Diversion sewer from trickling filter plant [line ii]		Table 7-5
Peak flow, m^3/s	0.385	Table 7-6
Diameter, m	0.76	Table 7-6
Diversion sewer from stabilization pond area [line i]		Table 7-5
Peak flow, m^3/s	0.590	Table 7-6
Diameter, m	0.91	Table 7-6
Existing intercepting sewer from central part of town [line iii]		Table 7-5
Peak flow, m^3/s	0.331	Table 7-6
Diameter, m	0.76	Table 7-6
Final sewer to the plant [line iv]		Table 7-5
Peak flow, m^3/s	1.321	Table 7-6
Diameter, m	1.53	Table 7-6
Bypass sewer to storage basin [line v]		Table 7-5
Peak flow, m^3/s	1.321	Table 7-6
Diameter, m	1.22	Table 7-6
Junction box. Three incoming sewers join, and one sewer carries the flow to the plant. Manually operated stop gates are provided to close the line and divert the flow into the relief sewer in case of power outages.		Sec. 7-5-2 and Figs. 7-8 and 7-9

TABLE 23-1 *Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21*

Item	Design Value or Description	Ref.
D. *Bar Screen*		Chap. 8
Number of mechanically cleaned bar screens	2	Sec. 8-6-1
Design capacity of each screen, m^3/s	1.321	Sec. 8-6-1
Width of screen chamber, m	1.74	Sec. 8-6-2, Step B, 1
Maximum depth above floor, m	6.45	Fig. 8-7
Length of screen chamber, m	11.5	Fig. 8-7
Slope of screen to horizontal, °	75	Fig. 8-7
Total number of clear openings	50	Fig. 8-5
Clear openings between bars, mm	25	Fig. 8-5
Number of bars, each bar 10 mm × 50 mm (deep)	49	Fig. 8-5
Depth of flow in channel at peak design flow, m	1.28	Table 8-4
Velocity through rack at peak design flow, m/s	0.83	Table 8-4
Head loss clean bars, m	0.03	Table 8-4
Head loss 50 percent clogging, m	0.15	Table 8-4
Average quantity of screenings, m^3/d	0.76	Sec. 8-6-2, Step E
Outlet structure, proportional weir, and free fall into wet well		Fig. 8-6
Rake cleaning	Front-cleaned	Sec. 8-8-3
Control system	Time cycle and high-level override	Sec. 8-8-6
E. *Pumping Station*		Chap. 9
Dry well dimensions, m × m	15.5 × 5	Fig. 9-15
Wet well dimensions, m × m	15.5 × 7	Fig. 9-15
Depth of pumping station above floor, m	11.38	Fig. 9-15
Total number of identical pumps, each vertical shafting, dry pit, mixed flow, centrifugal pump with variable-speed drive These pumps are arranged in parallel and discharge in a common header 92 cm in diameter. All pumps have 36-cm (14-in.) suction and discharge connections.	5	Sec. 9-9-1 and Fig. 9-17
Number of standby units	1	Sec. 9-9-1
Maximum static head (minimum wet well elev.), m	11.13	Figs. 9-13 and 21-1
Minimum static head (maximum wet well elev.), m	9.91	Figs. 9-13 and 21-1
Minimum station head and maximum station capacity when four pumps in operation (maximum wet well level), m and m^3/s	12.70, 1.56	Table 9-9
Maximum station head and minimum station capacity when one pump is in operation (minimum wet well level), m and m^3/s	11.30, 0.42	Table 9-9

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
Motor power, each variable-speed, kW	82	Sec. 9-9-2, Step C
Motor efficiency range, percent	81.7–83.4	Table 9-10
Method of pump control	Liquid level bubbler	Table 9-12
F. *Flow Measurement*		Chap. 10
Number of Venturi meter in the 92-cm diameter force main	1	Sec. 10-7-1
Flow range, m^3/s	1.321–0.152	Sec. 10-7-1
Head loss at maximum and minimum flow ranges (1.321 and 0.152 m^3/s), m	0.45 and 0.006	Sec. 10-7-2, Step C
The Venturi meter consists of a primary element with manual vent cleaners for high- and low-pressure connections, two pressure sensors and a differential pressure transmitter that translates differential pressure into an output signal. The sensor utilizes the isolating diaphragms to detect and transmit pressure.		Sec. 10-9-2
G. *Aerated Grit Chambers*		Chap. 11
Number of units	2	Sec. 11-10-1
Length, m	13.0	Sec. 11-10-2, Step A, 1
Width, m	3.5	Sec. 11-10-2, Step A, 1
Average water depth, m	3.65	Sec. 11-10-2, Step A, 1
Freeboard, m	0.8	Sec. 11-10-2, Step A, 1
Detention time at peak design flow (1.321 m^3/s) when both units are in operation, min	4.2	Sec. 11-10-2, Step A, 2
Influent structure consists of a 1-m-wide submerged channel with 1 m × 1 m orifice with sluice gates to divert the flow into one or both chambers.		Fig. 11-6
Effluent weir is a 2.5-m freefalling rectangular weir.		Fig. 11-6
Air supply, L/s per m length coarse bubble, swing-type diffusers provided on one side of the chamber for spiral roll action	7.8	Sec. 11-10-2, Step B, 1
Number of blowers	2	Sec. 11-10-2, Step B, 1
Capacity of blowers, at 27.6 kN/m^2 (gauge), sm^3/min	20	Sec. 11-10-2, Step B, 1
Grit is pushed into a hopper by a spiral conveyor and is removed by bucket elevator.		Sec. 11-12-4 and Fig. 11-6
Average quantity of grit removed, m^3/d	1.14	Sec. 11-10-2, Step I

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
H. *Primary Sedimentation*		Chap. 12
Number of rectangular basins	2	Sec. 12-8-2, Step A, 1
Length, m	46.33	Sec. 12-8-2, Step A, 1
Width, m	11.58	Sec. 12-8-2, Step A, 1
Average water depth at midlength, m	4.0	Sec. 12-8-2, Step A, 1
Freeboard, m	0.6	Sec. 12-8-2, Step A, 1
Detention time		
At average flow, both basins in operation, h	2.7	Sec. 12-8-2, Step A, 3
At peak design flow, both basins in operation, h	0.9	Sec. 12-8-2, Step A, 3
Overflow rate		
At average flow, both basins in operation, $m^3/m^2 \cdot d$	35.4	Sec. 12-8-2, Step A, 2
At peak design flow, both basins in operation, $m^3/m^2 \cdot d$	106.4	Sec. 12-8-2, Step A, 2
Influent structure consists of 1-m-wide channel with eight submerged orifices		Sec. 12-8-2, Step B, 1
Number of standard V-notch effluent weirs	765	Sec. 12-8-2, Step C, 3
Total length of weir plate, m	153.72	Sec. 12-8-2, Step C, 2
Average quantity of sludge produced, kg/d	6,227	Sec. 12-8-2, Step F, 2
Longitudinal sludge collector consists of chain, sprockets, wheels, flights, scrapers, and drive unit.		Sec. 12-10-3
Cross-collector consists of chain, sprocket, wheels, flights, and drive units.		Sec. 12-10-3
Number of self-priming centrifugal nonclog pumps per basin for sludge pumping	1	Sec. 12-8-1, Step F, 4, and Sec. 12-10-5
Skimmer consists of hand-operated scum trough, scum pit, and pump.		Sec. 12-10-7
I. *BNR Facility*		Chap. 13
Number of identical process trains	4	Sec. 13-11-2 Fig. 13-24
Influent quality after material mass balance analysis		
Q, m^3/d	42,000	Sec. 13-11-5
COD, mg/L	350	Sec. 13-11-5

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
BOD_5, mg/L	200	Sec. 13-11-5
TSS, mg/L	150	Sec. 13-11-5
Org.-N, mg/L	15	Sec. 13-11-5
NH_4^+-N, mg/L	20	Sec. 13-11-5
NO_3^--N, mg/L	0	Sec. 13-11-5
TN, mg/L	35	Sec. 13-11-5
TP, mg/L	6	Sec. 13-11-5
Anaerobic Zone		
Number of square chambers in each process train arranged in series	3	Sec. 13-11-6, Step A, 1
Dimensions $L \times W$, m	5.5×5.5	Sec. 13-11-6, Step A, 1
Depth, m	7.25	Sec. 13-11-6, Step A, 1
Freeboard, m	0.8	Sec. 13-11-6, Step A, 1
HRT based on Q, h	1.5	Sec. 13-11-5, Step A, 2
Anoxic Zone		
Number of square chambers in each process train arranged in series	3	Sec. 13-11-6, Step A, 2
Dimension $L \times W$, m	5.5×5.5	Sec. 13-11-6, Step A, 2
Depth, m	7.25	Sec. 13-11-6, Step A, 2
Freeboard, m	0.8	Sec. 13-11-6, Step A, 2
HRT based on Q, h	1.5	Sec. 13-11-5, Step B, 3
Aerobic Zone		
Number of rectangular basins in each process train	1	Sec. 13-11-6, Step A, 3
Dimensions $L \times W$, m	34×17	Sec. 13-11-6, Step A, 3
Depth, m	5	Sec. 13-11-6, Step A, 3
Freeboard, m	0.8	Sec. 13-11-6, Step A, 3
Aeration period based on Q, h	6.6	Sec. 13-11-5, Step C, 6
Food to MO ratio, kg BOD_5/kg VSS·d	0.18	Sec. 13-11-5, Step G, 1
Organic loading, kg BOD_5/m^3·d	0.53	Sec. 13-11-5, Step G, 2

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
MLVSS, maintained, mg/L	3000	Sec. 13-11-1, Step A, 16
Ratio of MLVSS to MLSS	0.8	Sec. 13-11-1, Step A, 16
Mean cell residence time, d	12	Sec. 13-11-1, Step A, 16
Solids wasted from the aeration basin (MLSS), kg/d	2820	Sec. 13-11-5, Step D, 4
WAS, m^3/d	752	Sec. 13-11-5, Step D, 2
Number of identical waste sludge pumps	5	Sec. 13-11-7, Step E, 2
Number of standby units	1	Sec. 13-11-7, Step E, 2
Solids lost in the effluent, kg/d	413	Sec. 13-11-5, Step D, 4
Returned sludge rates, Q_r/Q	0.6	Sec. 13-11-5, Step F, 1
Recycle ratio, $Q_{recycle}/Q$	1.7	Sec. 13-11-5, Step F, 3
Number of return sludge pumps	5	Sec. 13-11-8, Step H
Number of standby units	1	Sec. 13-11-8, Step H
Rated capacity of each pump, m^3/s	0.182	Sec. 13-11-8, Step H
Volume of air required m^3/min per basin	219	Sec. 13-11-7, Step B, 1
Diffusers are swing-type in 15 rows along the width of the basin. The diffusers are Dacron sock, standard tube, discharging 0.21 m^3 standard air per min per tube.		Fig. 13-27
Total number of diffusers per basin	1080	Sec. 13-11-7, Step C, 2
Total number of centrifugal blowers	5	Sec. 13-11-7, Step D, 3
Total number of standby units	1	Sec. 13-7-4, Step D, 3
Power requirement of each blower, kW	277	Sec. 13-11-7, Step D, 4
Rated air supply capacity of each blower, m^3/min	230	Sec. 13-11-7, Step D, 3
J. *Final Clarifier*		Chap. 13
Number of clarifiers	4	Sec. 13-11-1, Step B, 1

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
Diameter of each clarifier, m	40.7	Sec. 13-11-8, Step A, 4
Average water depth, m	5.0	Sec. 13-11-8, Step B, 4
Freeboard	0.5	Sec. 13-11-8, Step B, 5
Detention time		
At average design flow plus recirculation, h	8.5	Sec. 13-11-8, Step C, 2
At peak design flow plus recirculation, h	4.0	Sec. 13-11-8, Step C, 2
Overflow rate		
At average design flow plus recirculation, $m^3/m^2{\cdot}d$	12.8	Sec. 13-11-8, Step A, 5
At peak design flow plus recirculation, $m^3/m^2{\cdot}d$	26.8	Sec. 13-11-8, Step A, 7
Limiting solids flux		
At average design flow plus recirculation, $kg/m^2{\cdot}d$	47.8	Sec. 13-11-8, Step A, 8
At peak design flow plus recirculation, $kg/m^2{\cdot}d$	100.4	Sec. 13-11-8, Step A, 8
K. *UV disinfection*		Chap. 14
Number of UV disinfection channels	4	Sec. 14-12-2 and Fig. 14-18
Length, m	12	Fig. 14-21
Width, m	1.02	Sec. 14-12-3, Step A, 7, and Fig. 14-21
Depth variable, at peak design flow, m	0.72	Sec. 14-12-3, Step A, 7, and Fig. 14-21
Freeboard, m	0.6	Sec. 14-12-3, Step A, 7
Effluent structure consists of an automatic level control gate in each channel	1	Sec. 14-12-3, Step A, 14
Number of UV banks in each channel	2	Sec. 14-12-3, Step A, 7
Number of modules per bank	17	Sec. 14-12-3, Step A, 7
Number of lamps pr module	12	Sec. 14-12-3, Step A, 7
Lamp length, m	1.5	Sec. 14-12-1, Step B, 3

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
UV output, W/m·arc	18.2	Sec. 14-12-1, Step B, 3
Diameter of quartz jacket, cm	2.3	Sec. 14-12-1, Step B, 4
Flow measurement, Parshall flume		Sec. 14-12-3, Step B
Throat width, m	1.22	Sec. 14-12-3, Step B, 3
L. *Outfall Structure*		Chap. 15
The outfall structures consist of a concrete-lined trapezoidal channel, collection box, outfall pipe, and diffuser pipe		
Outfall channel		
Bottom width, m	0.5	Fig. 15-2
Side slope	3H-1V	Fig. 15-2
Collection box,		
Length and width $L \times W$, m	2×2	Fig. 15-2
Outfall pipe		
Diameter, m	0.92	Sec. 15-4-2, Step C
Diffuser pipe		
Diameter, m	0.92	Sec. 15-4-2, Step D, 3
Number of diffuser ports	6	Sec. 15-4-2, Step D, 3
Diameter of diffuser ports, m	0.27	Sec. 15-4-2, Step D, 3
M. *Sludge Thickeners*		Chap. 16
Number of gravity thickeners for combined primary and secondary sludge	2	Sec. 16-9-1
Diameter, m	12.2	Sec. 16-9-2, Step A, 4
Freeboard, m	0.6	Sec. 16-9-2, Step B, 4
Sidewater depth, m	3.9	Sec. 16-9-2, Step B, 4
Depth at the center, m	4.9	Sec. 16-9-2, Step B, 5
Solids loading, $kg/m^2 \cdot d$	43.7	Sec. 16-9-2, Step A, 5
Hydraulic loading, $m^3/m^2 \cdot d$	9.1	Sec. 16-9-2, Step A, 5
Dimensions of blending tank, depth $\times$ diameter, m	3×8.2	Fig. 16-6

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
Thickened sludge withdrawal rate, m^3/d	70.3	Sec. 16-9-2, Step G, 3
No. of sludge withdrawal pumps, plunger-type with time clock control	2	Sec. 16-9-2, Step G, 4
Thickened sludge solids, percent	6	Sec. 16-9-2, Step G, 3
Characteristics of blended and thickened sludge and supernatant		Table 16-10
N. *Sludge Stabilization*		Chap. 17
Number of anaerobic sludge digesters	2	Sec. 17-7-1
Diameter, m	13.7	Sec. 17-7-2, Step B, 1
Side water depth, m	8.5	Sec. 17-7-2, Step B, 1
Digestion period		
At average flow, d	17.9	Sec. 17-7-2, Step C, 1
At extreme high flow, d	11.1	Sec. 17-7-2, Step C, 1
At extreme low flow, d	28.1	Sec. 17-7-2, Step C, 1
Solids loading		
At average loading condition, kg $VS/m^3 \cdot d$	2.5	Sec. 17-7-2, Step C, 2
At extreme minimum loading condition, kg $VS/m^3 \cdot d$	2.1	Sec. 17-7-2, Step C, 2
At extreme high loading condition, kg $VS/m^3 \cdot d$	2.6	Sec. 17-7-2, Step C, 2
Gas production, m^3/d	2550	Sec. 17-7-2, Step D, 2, d
Diameter of gas storage sphere, m	15	Sec. 17-7-2, Step I, 1
Number of gas compressors	2	Sec. 17-7-2, Step I, 2
Power of gas compressors, kW	7.5	Sec. 17-7-2, Step I, 2
Digester mixing achieved by gas recirculation		Sec. 17-7-1
Total number of compressors for gas mixing, including one standby unit	3	Sec. 17-7-2, Step J, 1
Compressor power each unit, kW	15	Sec. 17-7-2, Step J, 1
Gas flow per digester, m^3/s	0.14	Sec. 17-7-2, Step J, 2

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
Digested sludge		
Quantity of digested solids produced, kg/d	5021	Sec. 17-8-2, Step E, 3, and Table 17-7
Volume of digested sludge, m^3/d	98	Sec. 17-8-2, Step E, 3, and Table 17-7
Characteristics of digested sludge and digester supernatant		Table 17-7
O. *Sludge Dewatering*		Chap. 18
Belt filter operation, h/d (5-day week)	8	Sec. 18-6-1
Total number of belt filter assembly	3	Sec. 18-6-2, Step A, 2
Belt width, m	1.2	Sec. 18-6-2, Step A, 2
Sludge solids processed, kg/h	879	Sec. 18-6-2, Step A, 1
Hydraulic loading	1.3 L/m belt width·h	Sec. 18-6-2, Step A, 3
Polymer, kg/h	3.5	Sec. 18-6-2, Step C, 3
Total	882	Sec. 18-6-2, Step A, 1
Sludge cake moisture content, percent	25	Sec. 18-6-1
Total average quantity of sludge cake, kg/h	596	Sec. 18-6-2, Step B, 2
Average volume of sludge cake, 5 d/week basis, m^3/d	18	Sec. 18-6-2, Step B, 2
Filtrate volume, m^3/d	258	Sec. 18-6-2, Step B, 3
Characteristics of sludge cake and filtrate		Table 18-7
P. *Biosolids Reuse and Residue Disposal*		Chap. 19
Quantity of biosolids utilized over farmland, mt/yr	1350	Sec. 19-7-1, Step A, 1
Crops grown: corn and grain sorghum		Sec. 19-7-2, Step A, 3
Biosolids application rate, mt/ha·yr	6.2	Sec. 19-7-2, Step A, 5
Lime requirement, mt	1012	Sec. 19-7-2, Step A, 8
K supplement, kg/ha·yr	109	Table 19-13
P in excess, some N supplement needed		Table 19-13
Land area needed, ha	220	Sec. 19-7-2, Step A, 7

TABLE 23-1 Summary of Basic Design Data and Dimensions of the Wastewater Treatment Facility Designed in Chapters 6–21

Item	Design Value or Description	Ref.
Disposal of screenings, grit, and skimmings and unused biosolids landfilled on the site		Sec. 19-7-1, Step B
Method of landfilling	Narrow trench cells	Sec. 19-7-1, Step B, 4
Number of trenches located perpendicular to flood protection levees	80	Sec. 19-7-2, Step B, 1, d
Total quantity of residues landfilled, m^3/d	2.22	Table 19-11
Dimensions of the trench cells		
Top dimension, m × m	20.4 × 8.4	Sec. 19-7-2, Step B, 1, d, and Fig. 19-8
Bottom dimensions, m × m	14 × 2	Sec. 19-7-2, Step B, 1, d, and Fig. 19-8
Excavated depth, m	3.5	Sec. 19-7-2, Step B, 1, d, and Fig. 19-8
Side slope	1:1	Sec. 19-7-2, Step B, 1, d, and Fig. 19-8
Impervious liner, compacted clay, drainage material, and drainage pipe		Sec. 19-7-2, Step B, 1, c
Top cover, m	1	Sec. 19-7-2, Step B, 1, e
Equipment needed		
Number of backhoes with loader	1 each	Sec. 19-7-2, Step B, 4, b
Life of fill, year	14.6	Sec. 19-7-2, Step B, 2, c

24

Advanced Wastewater Treatment and Upgrading Secondary Treatment Facility

24-1 INTRODUCTION

Upgrading of a wastewater treatment facility may be necessary to meet the existing effluent quality and/or to meet the stricter future effluent quality requirements. An inability to meet the existing effluent quality may result from lack of proper plant operation and control, inadequate plant design, and increased hydraulic or organic loading caused by a change in wastewater flow or characteristics. Such deficiencies can be overcome by staffing the plant with an adequate number of competent operators and laboratory personnel, a proper maintenance program, checking the key design components, and conducting sufficient wastewater sampling and treatability studies. Existing facilities can be expanded to handle higher hydraulic and organic loads by process modifications or building additional treatment units. The principles of design and operation of a secondary wastewater treatment facility have been extensively covered in the earlier chapters of this book.

Stricter effluent quality requirements are generally met by adding new treatment processes that provide the desired removal and efficiencies in conjunction with the existing secondary treatment facility. The stricter effluent quality may require organic and suspended solids removal beyond the capability of a secondary treatment plant. Under such requirements, additional treatment processes such as filtration, carbon adsorption, chemical precipitation of phosphorus, nitrification-denitrification, or biological nutrient removal may be necessary.

In the continual search for a simple, reliable, and inexpensive wastewater treatment system, natural systems have received much interest in recent years. They provide not only an efficient method of wastewater treatment and enhancement of secondary effluent quality, but also provide some indirect beneficial uses of the facility such as green space, wildlife habitat, and recreational areas. In addition to the natural systems, many advanced wastewater treatment systems are also used to retrofit existing POTWs to upgrade the effluent for removal of nitrogen, phosphorus, and other constituents. Therefore, this chapter is devoted to providing an overview of (1) natural treatment systems and (2) advanced wastewater treatment technology to enhance the removal of desired contaminants.

24-2 NATURAL SYSTEMS

In the natural environment, physical, chemical, and biological processes occur when water, soils, plants, microorganisms, and atmosphere interact. To utilize these processes, natural systems are designed. The processes involved in the natural systems include sedimentation, filtration, gas transfer, adsorption, ion exchange, chemical precipitation, chemical oxidation and reduction, and biological conversions, in addition to other unique processes such as photosynthesis, photooxidation, and aquatic plant uptake.[1] In natural systems, the processes occur at natural rates and tend to occur simultaneously in a single ecosystem reactor as opposed to mechanical systems in which processes occur sequentially in separate reactors or tanks in accelerated rates as a result of energy input. Generally, a natural system might typically include pumps and piping for wastewater conveyance but would not depend exclusively on external energy sources to maintain the major treatment responses.[2]

In this section a general overview of natural systems for wastewater treatment are presented. The natural systems are divided into (a) terrestrial systems and (b) aquatic systems. Both systems depend on physical and chemical responses, as well as on unique biological components.[2] Each of these systems is discussed below.

The constructed wetlands are becoming a viable wastewater treatment alternative for small communities, individual homes, and rest areas. Therefore, a great deal of information has been presented on site selection, design of physical facilities, performance expectations, hydraulic and organic loading rates, and cost of the constructed wetlands.

24-2-1 Terrestrial Treatment Systems

Land application is the sole terrestrial treatment system used to remove various constituents from the wastewater. It utilizes natural physical, chemical, and biological processes within the soil-plant-water matrix. The objectives of the land treatment system includes irrigation, nutrient reuse, crop production, recharge of groundwater and water reclamation for reuse. There are three basic methods of land application: (1) slow-rate irrigation, (2) rapid infiltration-percolation, and (3) overland flow. Each method can produce renovated water of different quality, can be adapted to different site conditions, and can satisfy different overall objectives.[3,4] Typical design features and performance expectations for the three basic terrestrial systems are compared in Table 24-1.[2,3,4-7]

Slow-Rate Irrigation. Irrigation is the most widely used form of land treatment system. The wastewater is applied to crops or vegetation, either by sprinkling or by surface techniques. In this process surface runoff is not allowed. A large portion of water is lost by evapotranspiration, whereas some water may reach the groundwater table. Groundwater quality criteria may be a limiting factor for system selection. Some factors that are given consideration in design and selection of an irrigation method are (1) availability of suitable site, (2) type of wastewater and pretreatment, (3) climatic conditions and storage needed, (4) soil type and organic or hydraulic loading rate, (5) crop production, (6) distribution methods, (7) application cycle, and (8) ground and surface water pollution.[2,5,8,9] Figure 24-1 illustrates the surface distribution and sprinkler application commonly used in irrigation systems.

Rapid Infiltration-Percolation. Rapid infiltration-percolation differs from the irrigation method in that wastewater percolates through the soils and treated effluent reaches the groundwater. Plants are not used for evapotranspiration as in irrigation systems. The objectives of rapid infiltration-percolation are (1) groundwater recharge, (2) natural treatment followed by withdrawal by pumping or underdrain systems for recovery of treated water, (3) natural treatment with groundwater moving vertically and laterally in the soil, and (4) recharge a surface water course.[5,8,9]

The operational principles of rapid infiltration-percolation systems are shown in Figure 24-2. Sandy and loamy soils perform better. BOD_5 and TSS removals are comparable to those of an irrigation system. However, nitrogen and phosphorus removals are not sufficient. Groundwater quality criteria may limit the use of this method. Table 24-1 gives a comparison of effluent quality and design considerations with other methods.[13]

Overland Flow. In overland flow systems, the wastewater is applied over the upper reaches of the sloped terraces and allowed to flow overland and is collected at the toe of the slopes. The collected effluent can be either reused or discharged to the receiving waters. Biochemical oxidation, sedimentation, filtration, and chemical adsorption are the primary mechanisms for removal of contaminants. Nitrogen removal is achieved through denitrification. Plant uptake of nitrogen and phosphorus are significant if crop harvesting is practiced.[2,5,8,9] Table 24-1 summarizes the design features, site characteristics, and effluent quality. Figure 24-3 illustrates the operational principles and hydraulic pathway for the overland system.

Design Considerations. Design and selection of land treatment systems are complex. Many engineering, environmental, health, physical and social science, legal, and economic factors warrant detailed investigation. In recent years much emphasis has been

TABLE 24-1 Comparison of Design Features, Site Characteristics, and Effluent Quality of Treated Wastewater from Land Treatment Processes

	Principal Processes		
Conditions	**Slow-Rate Irrigation**	**Rapid Infiltration**	**Overland Flow**
Design Features			
Application techniques	Sprinkler or surface[a]	Usually surface	Sprinkler or surface
Vegetation	Yes	No	Yes
Treatment goals	Secondary or AWT[b]	Secondary or ground water recharge	Secondary or AWT
Field area required, hectare[c]	23–250	2–23	6–40
Typical weekly application rate, cm/wk	1.3–10	10–305	6–15[d] 15–40[e]
Organic loading, kg BOD_5/ha·d	4–40	45–180	<95
Minimum pretreatment provided in U.S.	Primary sedimentation[f]	Primary sedimentation	Screen and grit removal
Disposition of applied wastewater	Evapotranspiration and percolation	Mainly percolation	Surface runoff and evapotranspiration with some percolation
Need for vegetation	Required	Optional	Required
Site Characteristics			
Slope	Less than 20 percent on cultivated land; less than 40 percent on noncultivated land	Not critical; excessive slopes require much earthwork	Finish slopes 2–8 percent
Soil permeability	Moderately slow to moderately rapid	Rapid (sands, loamy sands)	Slow (clays, silts, and soils with impermeable barriers)
Depth to groundwater	0.6–1.0 m	3 m (lesser depths are acceptable where underdrainage is provided)	Not critical

TABLE 24-1 Comparison of Design Features, Site Characteristics, and Effluent Quality of Treated Wastewater from Land Treatment Processes—cont'd

	Principal Processes		
Conditions	**Slow-Rate Irrigation**	**Rapid Infiltration**	**Overland Flow**
Climate restrictions	Storage often needed for cold weather and precipitation	None (possibly modify operation in cold weather)	Storage often needed for cold weather
Quality of Treated Wastewater[g]			
BOD_5 (mg/L)	2	5	10
TSS (mg/L)	1	2	10[h]
Ammonia N (mg/L)	0.5	2	0.8
Total N (mg/L)	3[i]	8	10
Total phosphorus (P) (mg/L)	0.1	1[j]	1[j]
Fecal coliform (number/100 mL)	<1	10	6

[a]Includes ridge-and-furrow and border strip.
[b]Advanced wastewater treatment.
[c]Field areas in hectares for 43.8 L/s (1 mgd) flow. This area does not include land required for buffer zone, roads, or ditches.
[d]Range for application of screened wastewater.
[e]Range for application of a lagoon and secondary effluent.
[f]Depends on the use of the effluent and type of crop.
[g](1) All values for slow rate are obtained for percolation of primary or secondary effluent through 1.4 m of soil. (2) All values for rapid infiltration are obtained for percolation of primary or secondary effluent through 4.5 m of soil. (3) All values for overland flow are obtained for runoff of comminuted wastewater over about 45 m of slope. (4) The first and second columns in slow-rate, rapid infiltration, and overland flow systems are the upper limits of average and maximum values, respectively.
[h]Removal depends on type of wastewater applied.
[i]Removal depends on type of crop and management.
[j]Measured in immediate vicinity. Increased removal with longer travel distance.
1 in. = 2.54 cm; 1 ft = 0.305 m; 1 acre = 0.405 hectare.
Source: Refs. 2, 3, 5–7, and 9.

given to land treatment as an alternative method of wastewater treatment and reuse to be evaluated in the overall wastewater treatment management program. Accumulation of toxic chemicals in soils and the food chain, groundwater contamination, and odor problems have received wide concern. Table 24-2 provides a summary of many factors that must be considered in the evaluation and selection of land treatment systems. In all respects the land treatment of wastewater must comply with the regulations promulgated under the Resource Conservation and Recovery Act (as amended) to protect human health and the environment from the improper management of wastewater.[1] Many additional references are available on the technical and environmental factors and economics of land treatment of wastewater.[6,11,12]

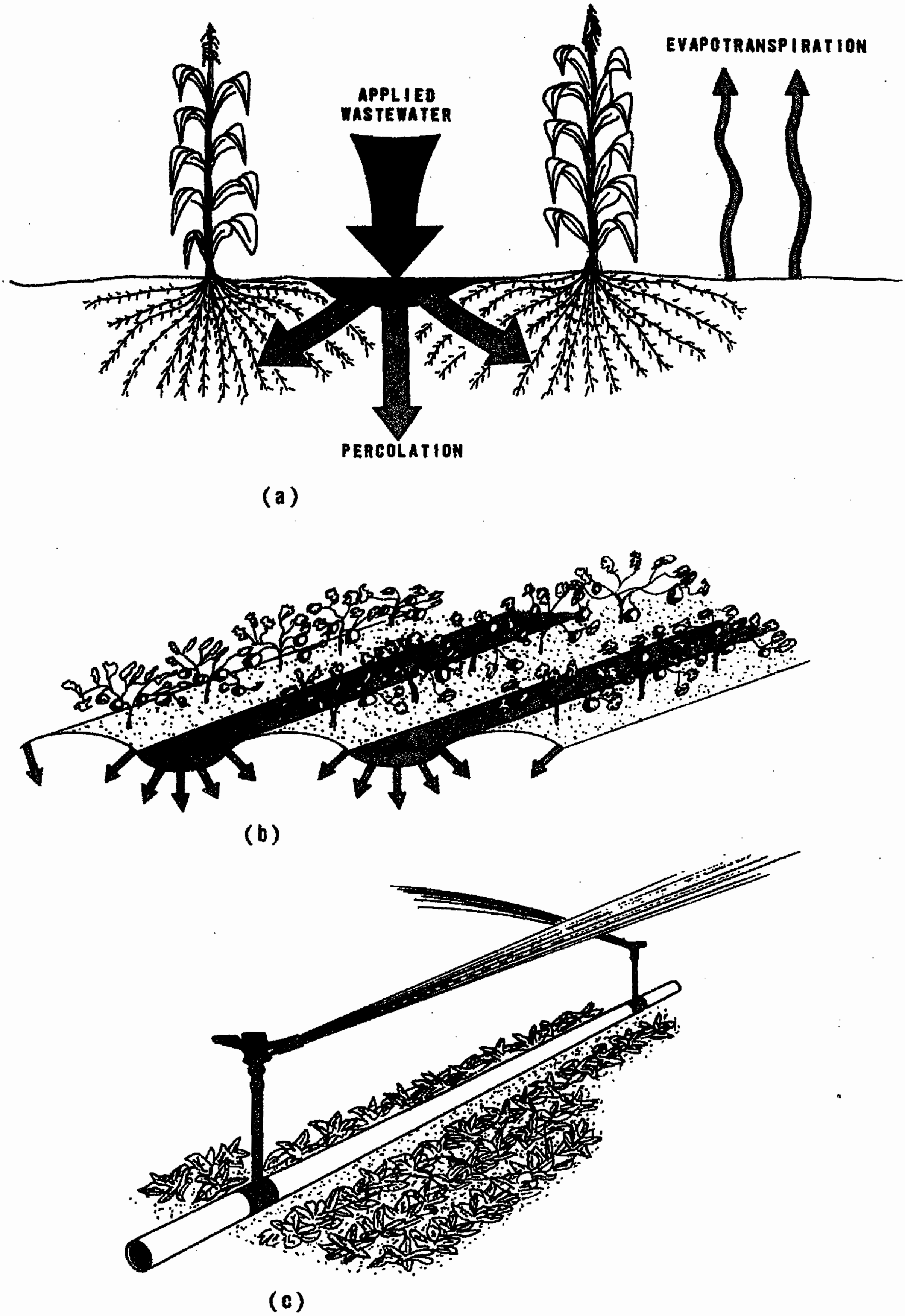


Figure 24-1 Hydraulic Pathway and Methods of Wastewater Application in Slow-Rate Irrigation System: (a) hydraulic pathway, (b) surface distribution, and (c) sprinkler distribution (from Ref. 5).

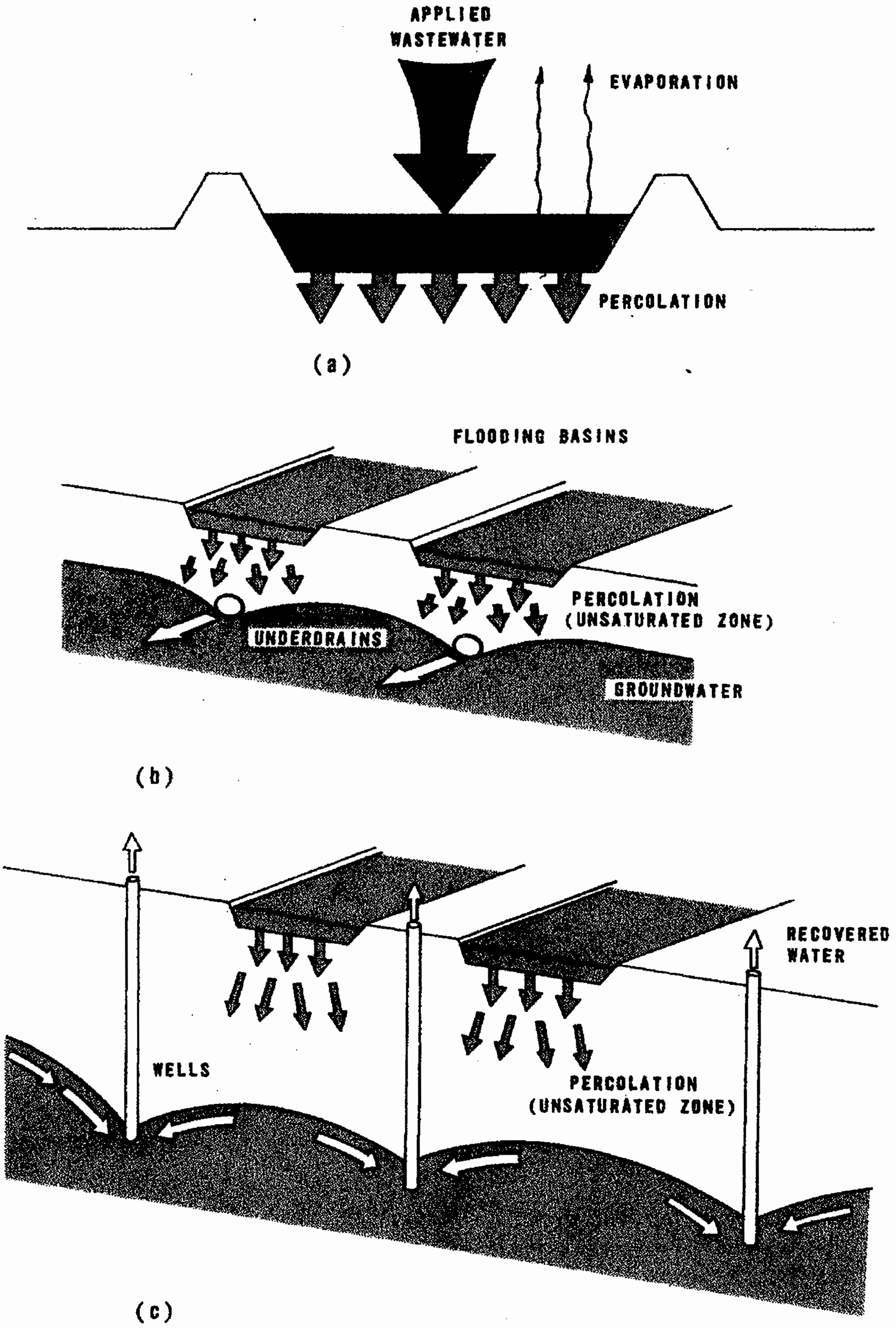


Figure 24-2 Hydraulic Pathway, Wastewater Application, and Recovery Methods of Renovated Water by Wells in Rapid Infiltration-Percolation System: (a) hydraulic pathway, (b) recovery of renovated water by underdrains, and (c) recovery of renovated water by wells (from Ref. 5).

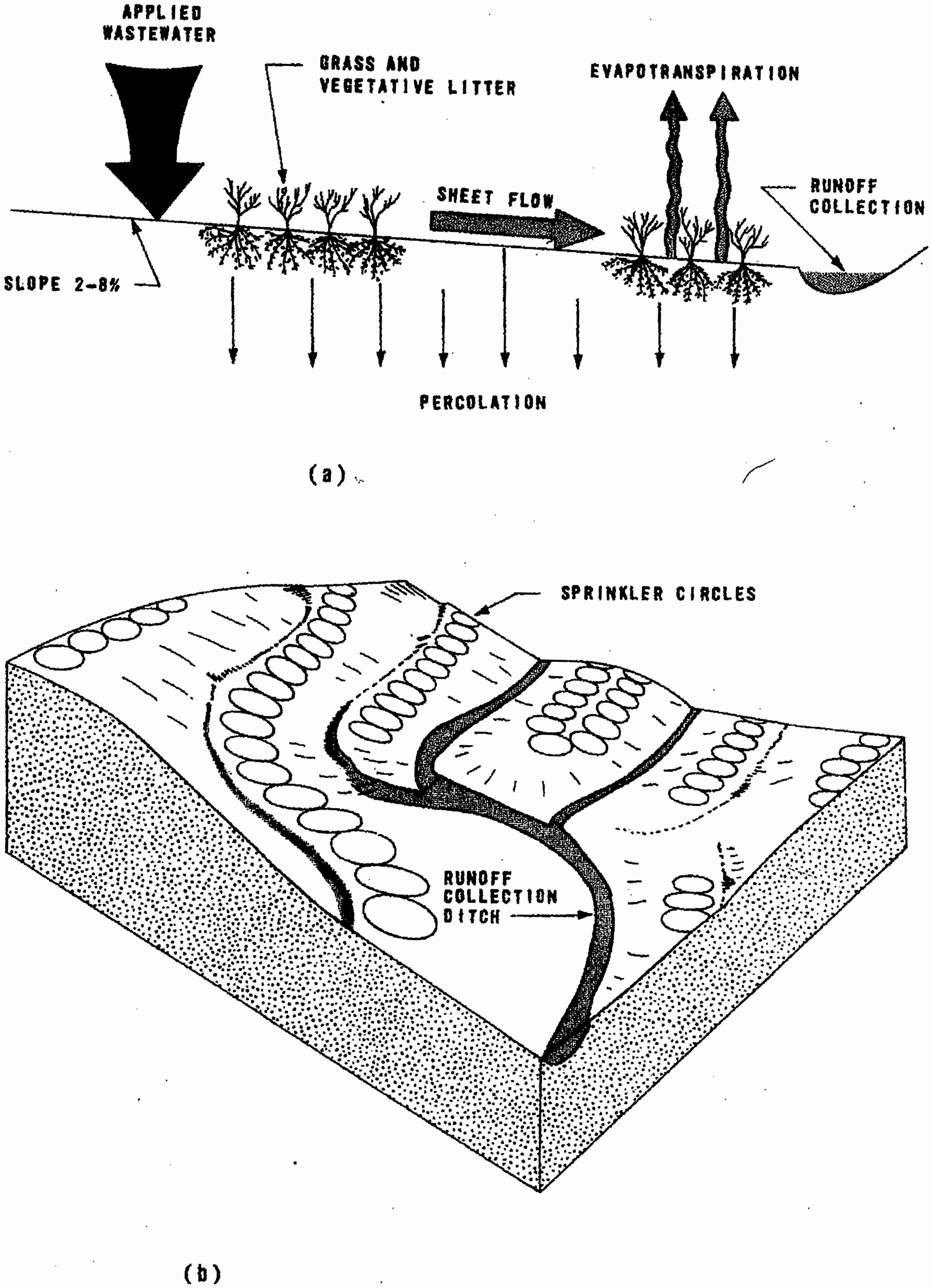


Figure 24-3 Hydraulic Pathway and Method of Application in Overland Flow System: (a) hydraulic pathway and (b) pictorial view of sprinkler application (from Ref. 5).

TABLE 24-2 General Design Considerations That Must Be Studied in Design and Selection of Land Treatment Methods

Wastewater Characteristics	Climate	Geology	Soils	Plant Cover	Topography	Application	Environmental
Volume	Total precipitation and frequency	Groundwater	Type	Indigenous to region	Slope	Method	Odors
Constituent load BOD Nitrogen Phosphorus Heavy metals	Evapotranspiration Temperature Growing season Occurrence and depth of frozen ground Storage requirements Wind velocity and direction	Seasonal depth Quality Points of discharge Bedrock, type, depth, and permeability	Gradation Infiltration/permeability Type and quantity of clay Cation exchange capacity Phosphorus adsorption potential Heavy-metal adsorption potential pH Organic matter	Nutrient removal capability Toxicity levels Moisture and shade tolerance Marketability	Aspect of slope Erosion hazard Crop and farm management	Type of equipment Application rate Types of drainage	Legal Socio-economic Health

Source: Adapted in part from Ref. 9.

24-2-2 Aquatic Treatment Systems

Aquatic systems utilize pond systems (also called stabilization ponds or lagoons) and wetlands. The performance of pond systems depends on microbial life and lower plants and animals, while wetlands support the growth of rooted plants. Both pond systems and wetlands are discussed below.

Pond Systems. Within the aquatic systems, pond systems are the most widely accepted method of wastewater storage and partial treatment. Basically, pond systems can be of three types based on oxygen requirements: aerobic, anaerobic, and facultative. In all cases the major treatment responses are a result of the biological components. In most of the pond systems, both performance and final water quality are dependent on the algae present in the system. Algae are functionally beneficial, providing oxygen to support other biological responses, and the algae–carbonate reactions are the basis for the effective nitrogen removal in the ponds. However, algae can be difficult to remove from the effluent. Since the ponds are typically biological systems, they are covered in Chapter 13. Readers should refer to Sec. 13-3-2 for more information on this subject. In most natural systems, ponds are generally used for storage and partial treatment of wastewater.[2-5,7]

Wetland Systems. The wetlands are inundated land areas in which the water table is at or above the ground surface (usually 0.6 m or more). This water table stands a long enough time each year to maintain saturated soil conditions and also to support the growth of related vegetation.

Vegetation provides a surface for the attachment of bacteria films, and aids in the filtration and adsorption of wastewater constituents. Vegetation also translocates oxygen from leaves to the root systems. A wide range of aerobic and facultative bacteria are supported by roots, and growth of algae is controlled by restricting the penetration of sunlight.[13,14] The unique ability of wetland plants to translocate oxygen to support a wide range of bacteria in the wetlands is shown in Figure 24-4. Wetlands can be of two types: natural wetlands and constructed wetlands. Both natural and constructed wetlands have been used for wastewater treatment, although the use of natural wetlands is generally limited to the polishing or further treatment of secondary or advanced wastewater-treated effluent.[2]

Natural Wetlands. From a regulatory standpoint, natural wetlands are usually considered as part of the receiving waters. Consequently, discharges to natural wetlands, in most cases, must meet applicable regulatory requirements, which typically stipulate secondary or advanced wastewater treatment.[15] Furthermore, the principal objective when discharging to natural wetlands should be enhancement of existing habitat. Modification of existing wetlands to improve treatment capability is often very disruptive to the natural ecosystem and, in general, should not be attempted.[15,16]

Constructed Wetlands. Constructed wetlands offer all of the treatment capabilities of the natural wetlands but without the constraints associated with discharging to a natural

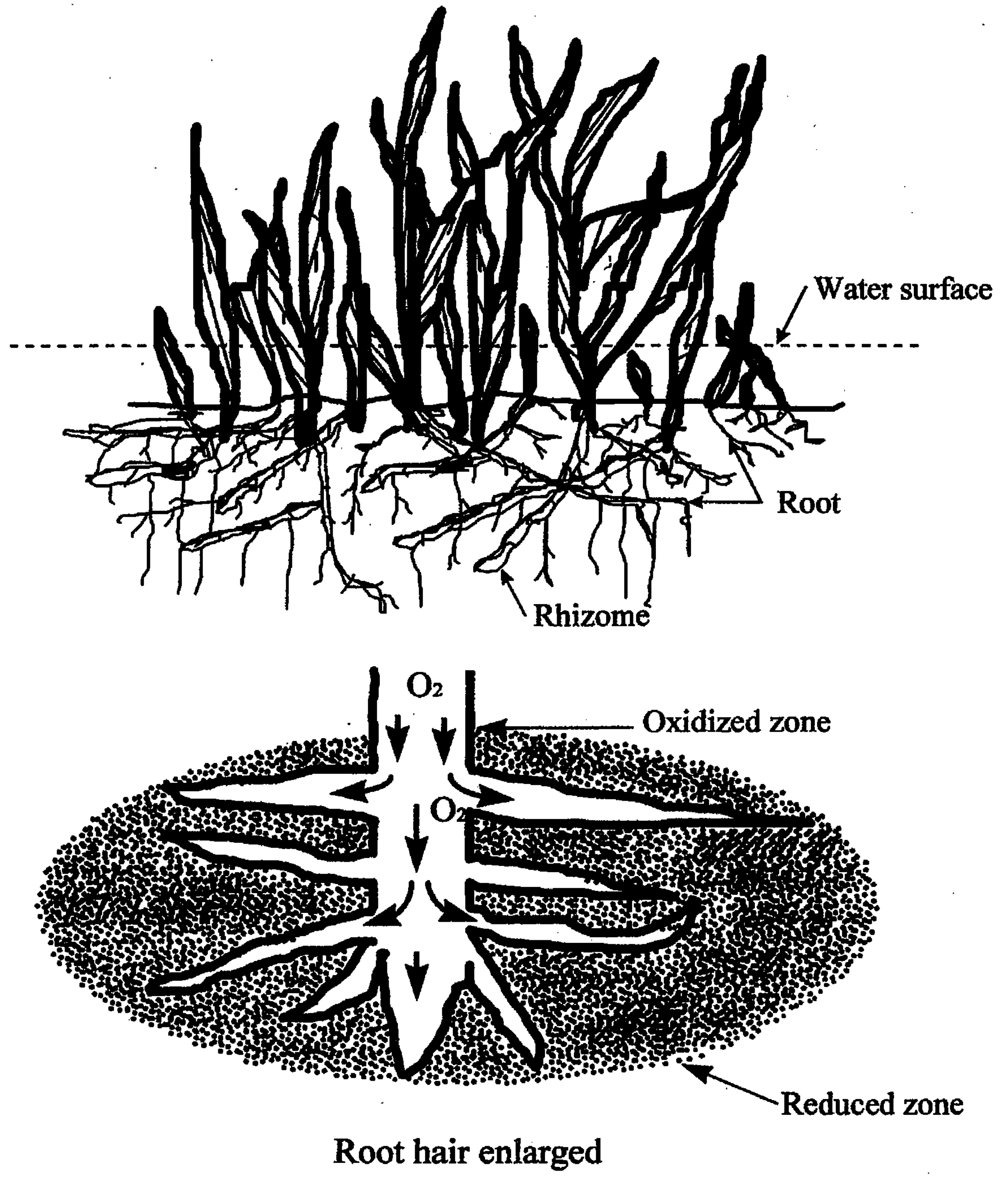
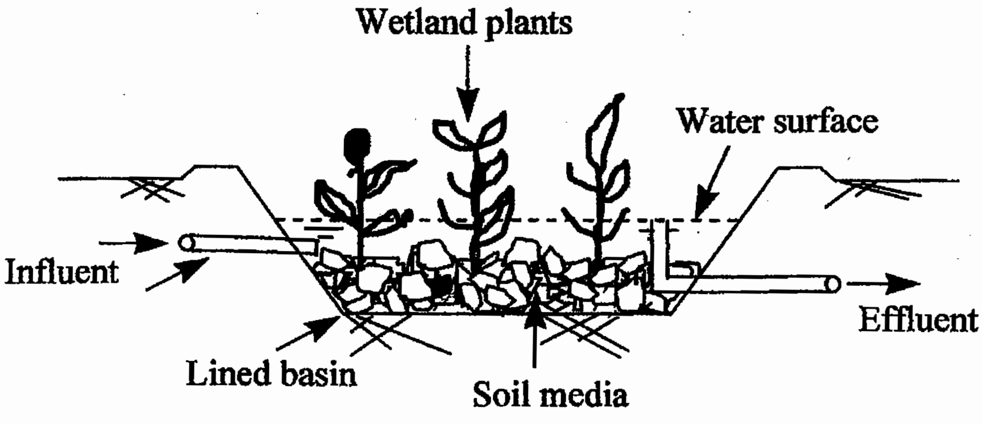


Figure 24-4 Aquatic Plants and an Enlarged Root Hair of Wetland Plants.

ecosystem. Additionally, the constructed wetland treatment units are not restricted to the special requirements on influent quality. They can ensure much more reliable control over the hydraulic regime in the system and therefore perform more reliably than the natural wetlands.[15,17] Two types of constructed wetlands have been developed for wastewater treatment: (1) free water surface (FWS) systems and (2) subsurface flow (SF) systems. The schematic flow diagrams of both types of systems are shown in Figure 24-5.[18] A general description of both types of systems is provided below. The basic design considerations such as (a) site selection, (b) plant types, (c) physical facilities, (d) hydrologic



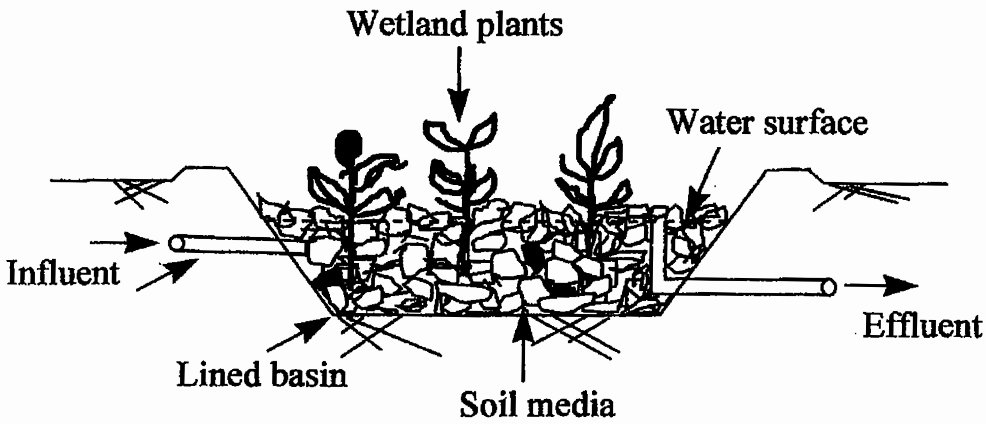


Figure 24-5 Schematic Flow Diagrams of Constructed Wetland Systems: (a) free water surface and (b) subsurface flow (from Ref. 18).

factors, (e) organic loading factors, and (f) performance expectations are presented under a separate section entitled Basic Design Considerations of Constructed Wetlands.

Free Water Surface (FWS) Wetlands. The FWS wetlands typically consist of a basin or channel with some type of barrier to prevent seepage, soil to support the root systems of the emergent vegetations, and water through the system at a relatively shallow depth.[2,14] The water surface in FWS wetlands is exposed to the atmosphere, and the intended flow path through the system is horizontal. Pretreated wastewater is applied continuously to such systems, and treatment occurs as the water flows slowly through the stems and roots of emergent vegetation. Free water surface systems may also be designed with the objective of creating new wildlife habitats or enhancing nearby existing natural wetlands. Figure 24-5(a) provides a schematic flow diagram of free water surface wetlands.

Subsurface Flow (SF) Wetlands. The SF wetlands also consist of a basin or channel with a barrier to prevent seepage. The basin or channel is filled to a suitable depth with a porous media. Rock and gravel are the most commonly used media types. The medium also supports the root systems of the emergent vegetation. The design of these systems assumes that the water level in the bed will remain below the top of the rock or gravel media. The flow path through the operational systems is usually horizontal.[17] The schematic flow diagram of submerged flow wetlands is provided in Figure 24-5(b).

Comparing the two types of constructed wetland systems, the SF type of wetlands offers several advantages over the FWS type. If the water surface is maintained below the media surface, there is little risk of odors, public exposure, or insect vectors. In addition, it is believed that the medium provides a larger available surface area for attached growth organisms. As a result, the treatment response may be faster, and a smaller surface area may be needed for the same wastewater conditions.[2,10] Furthermore, the subsurface position of the water and the accumulated plant debris on the surface of the SF bed offer great thermal protection in cold climates as compared to the FWS type.[19] The reported disadvantage of the SF type, however, is clogging of the media and possible overflow.

Basic Design Considerations of Constructed Wetlands. Constructed wetlands are a relatively recent development and are gaining popularity for the treatment of wastewater from small communities and residential and commercial areas. In this section the basic design information and economics of constructed wetlands are compared.

Site Selection. A constructed wetland can be developed in most locations. In selecting a site for a free water surface wetland, the underlying soil permeability must be considered. The most desirable soil permeability is 10^{-6} to 10^{-7} m/s.[15] Sandy clay and silty clay loams can be suitable when compacted. Sandy soils are too permeable to support wetland vegetation unless there is an impermeable restricting layer in the soil profile. Highly permeable soils can be used for wastewater flows by forming narrow trenches and lining the trench walls and bottoms with clay or an artificial liner. In heavy clay soils, the addition of peat moss or topsoil will improve soil permeability and accelerate initial plant growth.

Plants. Although natural wetlands typically contain a wide diversity of plant life, there is no need to attempt to reproduce the natural diversity in a constructed wetland. Such attempts in the past have shown that, eventually, cattails alone or in combination with either reeds or bulrushes will dominate in a wastewater system because of the high nutrient levels.[2] The emergent plants most frequently found in the wastewater wetlands include cattails, reeds, rushes, bulrushes, and sedges. Some of the major environmental requirements of these plants are given in Table 24-3.

Physical Facilities. The constructed wetlands behave typically like a plug-flow reactor. Constructed wetlands offer significant potential for optimizing performance through se-

TABLE 24-3 Emergent Aquatic Plants for Constructed Wetlands

Common Name (Scientific Name)	Temperature, °C		Maximum Salinity Tolerance (ppt[b])	Optimum pH
	Survival	Desirable[a]		
Cattail (*Typha* spp.)	10–30	12–24	30	4–10
Common reed (*Phragmites communis*)	12–33	10–30	45	2–8
Rush (*Juncus* spp.)	16–26	—	20	5–7.5
Bulrush (*Scirpus* spp.)	16–27	—	20	4–9
Sedge (*Carex* spp.)	14–32	—	—	5–7.5

[a]Temperature range for seed germination: roots and rhizomes can survive in frozen soils.
[b]ppt = parts per thousands.
Source: Adapted in part from Ref. 20.

lection of proper system configuration, including aspect ratios, compartmentalization, and location of alternate and multiple discharge sites.

The aspect ratio, defined as the ratio of length to width, typically varies from 4:1 to 10:1. However, based on research data developed on experimental constructed wetlands, the aspect ratios approaching 1:1 may be acceptable.[18] Several alternative flow diagrams and configurations of constructed wetlands are provided in Figure 24-6.

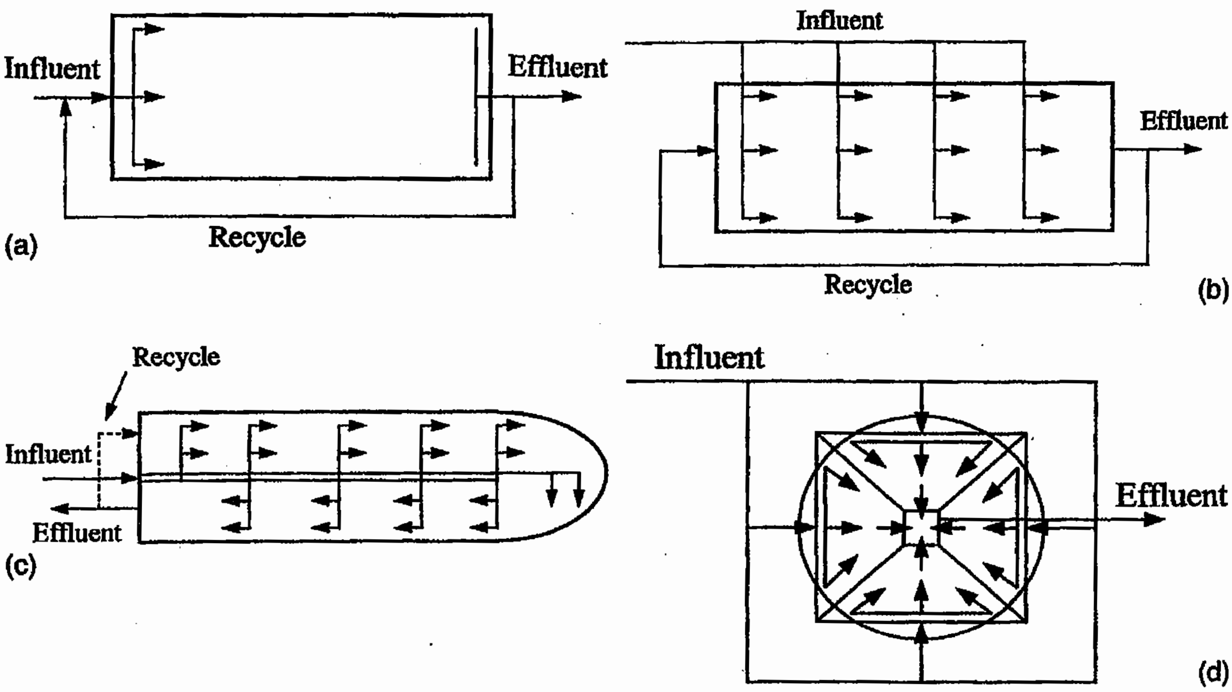


Figure 24-6 Alternate Flow Diagram and Configurations of Constructed Wetlands: (a) plug-flow with recycle, (b) step feed with recycle, (c) step feed in wraparound pond, and (d) peripheral feed with center drawoff (adapted in part from Refs. 2, 14, and 18).

Hydrologic Factors. The performance of the constructed wetlands system is dependent on the system hydrology as well as many other factors such as precipitation, infiltration, evapotranspiration, hydraulic loading rate, and water depth. These factors affect the removal of organics, nutrients, and trace elements, not only by altering the detention time, but also by either concentrating or diluting the wastewater.[14,15] For a constructed wetlands, the water balance can be expressed by Eq. (24-1):

$$[dV/dt] = Q_i - Q_e + P - ET \tag{24-1}$$

where

Q_i = influent wastewater flow, m^3/d
Q_e = effluent wastewater flow, m^3/d
P = precipitation, m^3/d
ET = evapotranspiration, m^3/d
$[dV/dt]$ = change in volume of water per unit time, m^3/d
t = time, d

The hydraulic loading rate for an FWS system is closely tied to the hydrologic factors and conditions specific to the site. A typical hydraulic loading rate of 198 $m^3/d \cdot ha$ (21,000 gpd/acre) is considered sufficient for optimum treatment efficiency.

BOD_5 Loading Rates. There are two goals for organic load control in the constructed wetlands. The first is the provision of a carbon source for denitrifying bacteria. The second goal is to prevent overloading of the oxygen transfer ability of the emergent plants. High organic loading, if not properly distributed, will cause anaerobic conditions, and plants may die. The maximum organic loading rate for both types of systems (FWS and SF) should not exceed 112 kg $BOD_5/ha \cdot d$.[2,14,15]

Performance Expectations. Constructed wetland systems can significantly remove the biochemical oxygen demand (BOD), total suspended solids (TSS), nitrogen, and phosphorus, as well as metals, trace organics, and pathogens. The basic treatment mechanisms include sedimentation, chemical precipitation, adsorption, and microbial decomposition, as well as uptake by vegetation. Removal of BOD_5, TSS, nitrogen, phosphorus, heavy metals, and toxic organics have been reported.[21,22] The performance of many well-known constructed wetlands in terms of BOD_5, TSS, ammonia nitrogen, and total phosphorus is summarized in Table 24-4.

Mosquito control and plant harvesting are the two operational considerations associated with constructed wetlands for wastewater treatment. Mosquito problems may occur when wetland treatment systems are overloaded organically and anaerobic conditions develop. Biological control agents such as mosquitofish *(Gambusia affins)* die either from oxygen starvation or hydrogen sulfide toxicity, allowing mosquito larvae to mature into adults. Strategies used to control mosquito populations include effective pretreatment to reduce total organic loading; stepfeeding of the influent wastewater stream with effective influent distribution and effluent recycle; vegetation management; natural con-

TABLE 24-4 Performances of Some Well-Known Constructed Wetlands

System Name	Location	Type	Records (years)	Area (ha)	Flow (m^3/d)	BOD_5 (mg/L)		TSS (mg/L)		NH_4^+-N (mg/L)		TP (mg/L)	
						In	Out	In	Out	In	Out	In	Out
Lakeland	FL	FWS	2	498	26978	3	2.5	4	3.5	0.9	0.4	9.5	4.1
Reedy Creek	FL	FWS	11.2	35.2	12058	5.3	1.9	8.9	2.4	3.0	0.7	1.4	1.8
Reedy Creek	FL	FWS	11.2	5.9	2423	5.8	1.6	10.9	2.4	3.3	0.1	1.8	0.8
Ironbridge	FL	FWS	1	486	34254	4.8	2.1	10.5	65.9	4.0	0.9	0.5	0.1
Apalachicola	FL	FWS	3	63.7	3936	15.2	1.1	107	8	3.6	0.1	3.0	0.2
Fort Deposit	AL	FWS	0.67	6	628	29.9	5.4	78.7	10.4	13.6	1.0	—	—
West Jackson	MS	FWS	0.5	8.91	1953	21.6	10.5	65.9	24.5	2.7	0.2	5.1	3.6
Leaf River 1	MS	FWS	1.2	0.13	225	15.8	14	54.8	30.1	9.9	7.2	8.9	8.2
Leaf River 2	MS	FWS	1.2	0.13	254	15.8	15.7	54.8	34.9	9.9	6.3	8.9	8.2
Leaf River 3	MS	FWS	1.2	0.13	220	15.8	13.9	54.8	25.5	9.9	6.8	8.9	5.9
Cobalt	Canada	FWS	1	0.10	49	20.7	4.6	36.2	28	3.0	1.0	1.7	0.8
Pembroke	KY	FWS	0.75	1.48	188	67.4	9.4	91.9	8.2	13.8	3.4	6.0	3.2
Central Slough	SC	FWS	4	32	5372	16.3	6.5	27.7	14.8	7.5	1.4	4.1	1.5
Gustine 1A	CA	FWS	1	0.39	163	130	49.8	73.5	39.6	17.4	16.1	—	—
Gustin 2A	CA	FWS	1	0.39	174	151	44.8	99.8	33.8	18.0	23.2	—	—
Philips School	AL	SF	2	0.2	58.7	15.3	1	63.7	2	11	1.7	6	0.3
Kingston	TN	SF	0.7	0.93	76	56	9	83	3	22	16	3.4	2.1
Denham Sprg.	LA	SF	1.5	2.1	2548	28.2	10.5	53	17	11	4.3	—	—
Monterey	VA	SF	1.1	0.023	83	38	15	32	7	9.3	8.7	—	—

Source: Adapted in part from Ref. 18.

trols, principally mosquitofish, in conjunction with the above techniques; and application of approved and environmentally safe chemical control agents.

The usefulness of plant harvesting in wetland treatment systems depends on several factors, including climate, plant species, and the specific wastewater objectives. Plant harvesting can affect treatment performance of wetlands by altering the effect that plants have on the aquatic environment. In addition, because harvesting reduces congestion at the water surface, control of mosquito larvae using fish is enhanced. It has been reported in the literature that a total of 29,000 kg/ha dry weight of harvestable biomass of *Phragmites* shoots can be harvested for a single harvesting in a year. A higher yield is achievable with multiple harvesting.

The BOD_5, TSS, nitrogen, and phosphorus removal efficiencies of constructed wetlands are discussed below.

BOD_5 Removal in FWS Wetlands. In the FWS constructed wetlands, the soluble BOD_5 removal is caused by microbial growth attached to plant root, stems, and leaf litter that has fallen into the water. BOD_5 removal is generally expressed by a first-order reaction kinetic [Eq. (24-2)].[2,3,15]

$$[C_e/C_o] = \exp(-K_T\, t) \tag{24-2}$$

where

C_e = effluent BOD_5, mg/L
C_o = influent BOD_5, mg/L
K_T = reaction rate constant for FWS wetland, d^{-1}
t = hydraulic retention time, d

BOD_5 Removal in SF Wetlands. The major oxygen source for the subsurface components (soil, gravel, rock, and other media in trenches or beds) is the oxygen transmitted by the vegetation to the root zone. In most cases, there is very little direct atmospheric reaeration because the water surface remains below the surface of the media.[22,23] Removal of BOD_5 is expressed by Eq. (24-3). This is also a first-order equation and can be rearranged to calculate the area required for the subsurface flow system:

$$C_e/C_o = \exp[-A_s\, K_t d \cdot e]/Q \tag{24-3}$$

where

C_e = effluent BOD_5, mg/L
C_o = influent BOD_5, mg/L
K_t = reaction rate constant for SF, d^{-1}
Q = flow rate through the system, m^3/d
d = depth of submergence, m
e = porosity of the bed
A_s = surface area of the system, m^2

Suspended Solids Removal. Suspended solids removal is very effective in both types of constructed wetlands. Most of the removal occurs within a few meters beyond the inlet. Control dispersion of the inlet flow will enhance removal near the inlet zone. Proper dispersion of solids can be achieved by low inlet velocities, even cross-sectional loadings, and uniform flow without stagnation.[10,24]

Nitrogen Removal. Nitrogen removal is very effective in both the free water surface and submerged flow constructed wetlands. The nitrification-denitrification is the major path of nitrogen removal. Total nitrogen removals of up to 79 percent are reported at nitrogen loading rates (based on elemental N) up to 44 kg/(ha·d) [39 lb/(acre·d)] in a variety of constructed wetlands. If plant harvesting is practiced, a higher rate of nitrogen removal can be expected.[14,24]

Phosphorus Removal. Phosphorus removal in many wetlands is not very effective because of limited contact opportunities between the wastewater and the soil. The exceptions are in the submerged bed design when proper soils are selected as the medium for the system.[26] A significant clay content and the presence of iron and aluminum will enhance the potential for phosphorus removal.[14,25,26]

Cost. Cost is often a significant factor in selecting the type of treatment system for a particular application. Unfortunately, the availability of reliable cost data for wetland treatment systems is limited. The cost of wetland treatment systems varies, depending on wastewater characteristics, the type of wetland system, and the type of bottom preparation required. Subsurface flow systems are generally more expensive than free water surface systems. It has been reported that the construction cost of wetlands developed by the Tennessee Valley Authority (TVA) ranged from $3.58/$m^2$ to $32.03/$m^2$. Estimates are that the construction, operation, and maintenance costs of constructed wetland systems are quite competitive with other wastewater treatment options.[27]

24-3 OVERVIEW OF ADVANCED WASTEWATER TREATMENT TECHNOLOGY

Advanced wastewater treatment technology is designed to remove those constituents that are not adequately removed in the secondary treatment plants.[28,29] These include nitrogen, phosphorus, and other soluble organic and inorganic compounds. Nitrogen and phosphorus constitute nutrients that accelerate the plants' growth in the receiving waters. Ammonia is also toxic to fish, exerts nitrogenous oxygen demand, and increases chlorine demand. Heavy metals, hydrogenated hydrocarbons, phenolic compounds, and more are toxic to fish and other aquatic life, concentrate in the food chain, and may create taste and odor problems in water supplies. Many of these constituents must be removed to meet more stringent water quality standards and also to reuse the effluent for municipal, industrial, irrigation, recreation, and other water needs. A brief discussion of effluent reuse is given in Chapter 15.

The most commonly used advanced wastewater treatment processes are biological nutrient removal, chemical coagulation and precipitation of phosphorus, ammonia stripping, breakpoint chlorination, filtration, carbon adsorption, ion exchange, and membrane processes. Each of these unit operations and processes is discussed below. Additional information concerning principal applications and degree of removal achieved by these systems may be found in Tables 4-1 and 4-2.

24-3-1 Biological Nutrient Removal Processes

Biological nutrient removal (BNR) processes provide enhanced nutrient removal and are generally integrated with a conventional biological treatment process used for BOD and TSS removal. The process utilizes anaerobic, anoxic, and aerobic zones or sequences to achieve phosphorus release and uptake and nitrification and denitrification. The process is capable of achieving effluent quality of BOD_5/TSS/TP/TN: 5/5/1/5. In-depth coverage of theory and the design of BNR systems is provided in Chapter 13. It is expected that in the future the BNR technology will be utilized extensively to upgrade existing facilities and to build new ones. Readers should refer to Chapter 13 for full coverage on biological phosphorus removal and nitrification-denitrification.

24-3-2 Chemical Coagulation and Precipitation of Phosphorus

Process Description. Coagulation involves the reduction of surface charges and the formation of complex hydrous oxides. Flocculation involves combining the coagulated particles to form settleable floc. The coagulant (alum, ferric chloride, ferrous sulfate, ferric sulfate, etc.) is mixed rapidly and then stirred to encourage formation of floc prior to settling. The objective is to improve the removal of BOD, TSS, and phosphorus. In-depth coverage of chemical coagulation and precipitation is presented in Chapter 12 (Sec. 12-5-2). To accomplish phosphorus removal, chemicals are added prior to primary sedimentation, in suspended and attached growth biological treatment processes, and in separate facilities following the biological treatment processes (Figure 24-7). Polymers have also been used in conjunction with these chemicals. The chemical dosage is adjusted to give the desired amount of floc formation and BOD, TSS, or phosphorus removal. The exact application rate is determined by the jar test. The approximate average alum and ferric chloride dosages in municipal wastewater are 170 and 80 mg/L, respectively. Lime may be needed in conjunction with iron salts.[29,30]

Proper mixing of chemicals at the point of addition and proper flocculation prior to clarification are essential for maximum effectiveness. Flocculation may be accomplished in a few minutes to half an hour in basins equipped with mixers, paddles, or baffles. The coagulants react with alkalinity to produce insoluble metal hydroxide for floc formation. If sufficient alkalinity is not present, lime or soda ash (sodium carbonate) is added in the desired dosages.

In biological treatment units, the addition of coagulants has a marked influence on biota. The population of protozoans and higher animals is adversely affected. However, the BOD, TSS, and phosphorus removal is significantly improved. Overdosing of chemicals may cause toxicity to microorganisms.

Large quantities of sludge are produced from chemical precipitation. The chemical sludges may also cause serious handling and disposal problems.

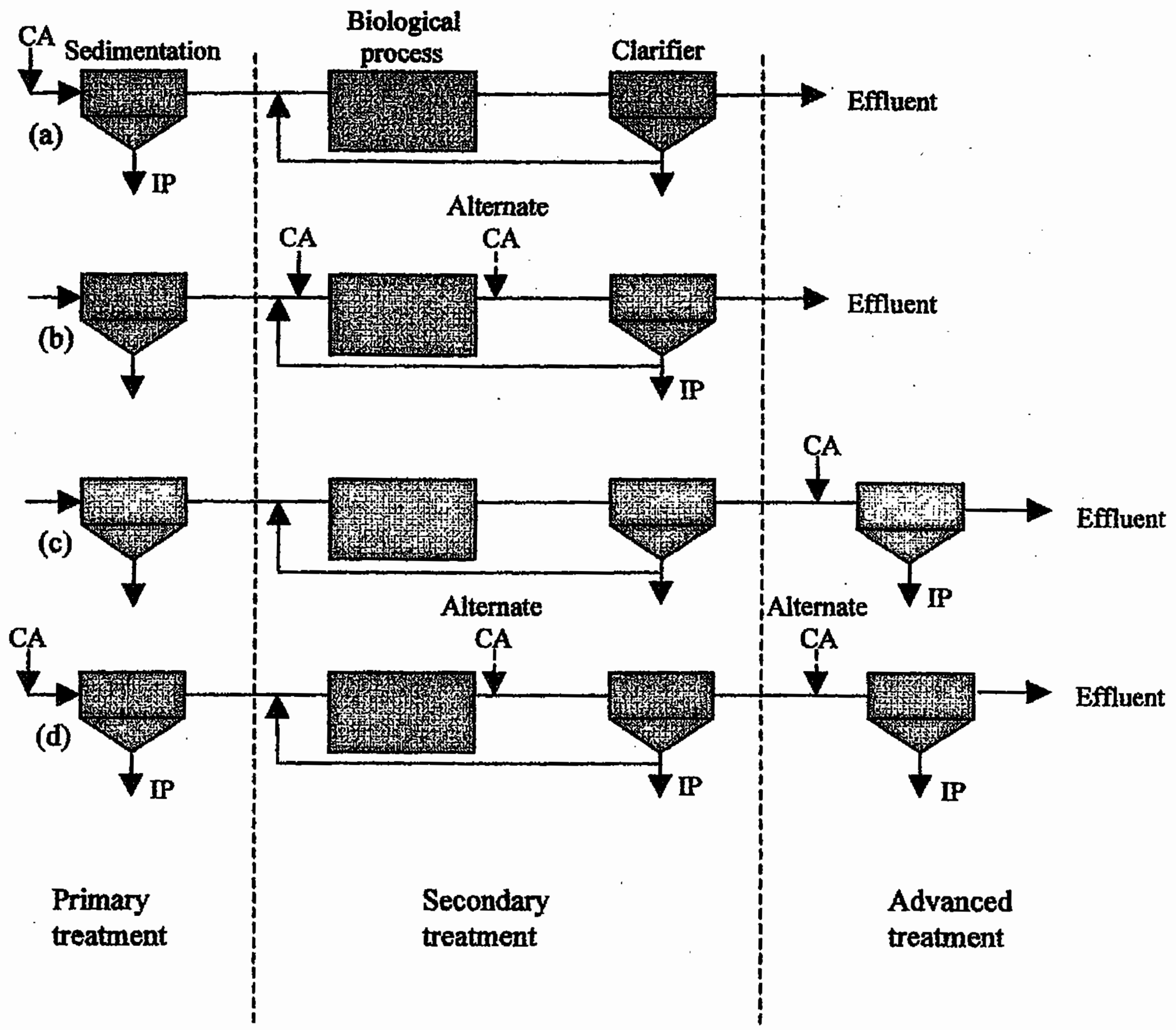


Figure 24-7 Alternative Points of Chemical Addition (CA) for Insoluble Phosphorus (IP) Removal: (a) before primary sedimentation, (b) before and/or following biological treatment, (c) following secondary treatment, and (d) at several locations in a process (known as "split treatment") (from Ref. 7).

Equipment. Major equipment for a coagulation-flocculation system includes chemical storage, chemical feeders, pipings and control systems, flash mixer, flocculator, and sedimentation basin.

Design Parameters. The important design parameters for coagulation and flocculation units are average and peak design flows, chemical dosage and feed system, flash mix tank, and flocculation and sedimentation basins. The design parameters for coagulation, flocculation, and sedimentation devices have been presented in Chapters 12, 13, and 16.

24-3-3 Lime Precipitation

Process Description. Lime reacts with bicarbonate alkalinity and orthophosphate, causing flocculation. The objectives of lime addition are to increase removal of BOD, TSS, and phosphorus. Lime is added prior to primary sedimentation, in biological treatment, or in a separate facility after secondary treatment. Lime addition may be single-stage or two-stage.[31,32]

Single-Stage. Lime in a single stage is used in primary, biological treatment or after secondary treatment. The procedure is the same as in coagulation. The pH of the wastewater is raised to about 10, and the wastewater is flocculated and then settled. Normal lime dosage is about 180–250 mg/L as CaO. The actual dosage depends primarily on phosphorus concentration, hardness, and alkalinity. The biological system is not adversely affected by lime addition in moderate amounts (80–120 mg/L as CaO). The microbial production of carbon dioxide is sufficient to maintain a pH near neutral. High dosages may upset the biological process.

Two-Stage. Lime addition prior to primary and after secondary treatment may be achieved in two stages. The pH of the wastewater is raised to greater than 11, flocculated, and settled. Lime dosages up to 450 mg/L as CaO may be necessary. The effluent is carbonated by adding CO_2 to lower the pH and then flocculated and settled. Higher BOD, TSS, and phosphorus removal is achieved in a two-stage lime process. However, use of excess lime causes scale formation in tanks, pipes, and other equipment. Also, handling and disposal of large quantities of lime sludge are problems.

Equipment. Major equipment for a lime precipitation process includes lime storage; lime feeders, pipings, and control system; flash mixer; flocculator; sedimentation basin; and, for a two-stage system, a CO_2 source such as an incinerator or an internal combustion engine.

Design Parameters. Design parameters are the same as those listed for the coagulation-flocculation system.

24-3-4 Ammonia Stripping

Process Description. Ammonia gas can be removed from an alkaline solution by air stripping as expressed by Eq. (24-4):

$$NH_4^+ + OH^- \rightarrow NH_3 \uparrow + H_2O \qquad (24\text{-}4)$$

The process requires (1) raising the pH of the wastewater to about 11, (2) formation of droplets in the stripping tower, and (3) providing air–water contact and droplet agitation by countercurrent circulation of large quantities of air through the tower. Ammonia-stripping towers are simple to operate and can be very effective in ammonia removal, but their efficiency is highly dependent on air temperature. As the air temperature decreases, the ammonia removal efficiency drops significantly. This process, therefore, is not recommended for a cold climate. A major operational disadvantage of stripping is the need for neutralization and calcium carbonate scaling on the tower. Also, there is some concern over discharge of ammonia into the atmosphere.[32-36]

Equipment. The basic equipment for an ammonia-stripping system includes chemical feed, stripping tower, pump and liquid spray system, forced air draft, and recarbonation system.

Design Parameters. Important design parameters for an ammonia-stripping system are average and peak design flow, surface loading rate, chemical dosage, tower height, and air-to-water ratio.

24-3-5 Breakpoint Chlorination

Process Description. Oxidation of ammonia-nitrogen to nitrogen gas is achieved by breakpoint chlorination (Eq. 24-5):

$$2NH_3 + 3HOCl \rightarrow N_2 + 3H_2O + 3HCl \tag{24-5}$$

The ratio of chlorine (Cl_2) to ammonia (NH_3) is 7:1–10:1.[36,37] Optimum pH is in the range of 6–7. The advantages of breakpoint chlorination are a low capital cost, high degree of efficiency and reliability to ammonia removal, disinfection, and insensitivity to cold weather. The disadvantages are the formation of high residual chloride concentration and chlorinated organic compounds.[38] Dechlorination is often necessary.

Equipment. The major equipment necessary for breakpoint chlorination is the same as those for chlorine disinfection and are discussed in Chapter 14. These include flow measurement, chemicals and equipment for pH adjustment, chlorine storage, chlorine feed piping and mixers, and contact tank.

Design Parameters. The important design parameters are average and peak design flow, concentrations of ammonia and chlorine, and contact period.

24-3-6 Ion Exchange for Ammonia Removal

Process Description. Wastewater is passed through a bed of clinoptilolite (a zeolite resin), which selectively removes the ammonium ion. Although NH_4^+ is selectively removed, other ions such as K also occupy the active site. The exchange capacity of clinoptilolite for NH_4^+ ions may vary from 0.14–0.38 meq/g.[39-41] When the resin becomes saturated, it is regenerated with a lime slurry containing sodium chloride. The lime solution after regeneration of the resin must be processed to remove the ammonium ions so that the solution can be reused. Air stripping of ammonia from the solution is a feasible method because of the small flow involved. The stripped ammonia gas is passed through an absorber material, which has high selectivity for ammonia. Proper disposal of ammonia-bearing absorber material is necessary. Nitrification-denitrification of retained NH_4^+ ion has also been attempted.[40,41]

Equipment. Major equipment for an ion exchange system includes an ion exchange bed, bed regeneration system, ammonia-stripping tower, and ammonia absorber material.

Design Parameters. The design parameters are average and peak design flow, ion exchange capacity of the resin bed, concentration of ammonia, percolation rate, and so on.

24-3-7 Filtration

Process Description. Filtration has been used to polish secondary effluent or to produce effluent that is low in suspended solids. Filtration is not considered an advanced wastewater treatment process; instead, it is used as a pretreatment device prior to other advanced treatment processes.

Filtration consists of passing wastewater effluent through a filtering medium that can strain out the colloidal particles. The filtering media may be fine sand, anthracite, mixed media, diatomaceous earth, or filter fabric. The filtration system may be gravity filter or pressure filter. The solids accumulate in the filter media; therefore, backwashing using clean water in the opposite direction of flow must be accomplished periodically to control the head loss through the filtration system. Different types of filter arrangements and components are shown in Figure 24-8.

Filter Types. The principal types of granular-media filters are[7,42,43]

1. Mono-medium filters that utilize one type of filtering media such as sand or anthracite. These filters may have shallow, medium, or deep beds. They may also be stratified or unstratified.
2. Dual-media or multimedia bed filters that use a combination of sand, anthracite, and garnet or ilmenite. These filters may also be stratified or unstratified.

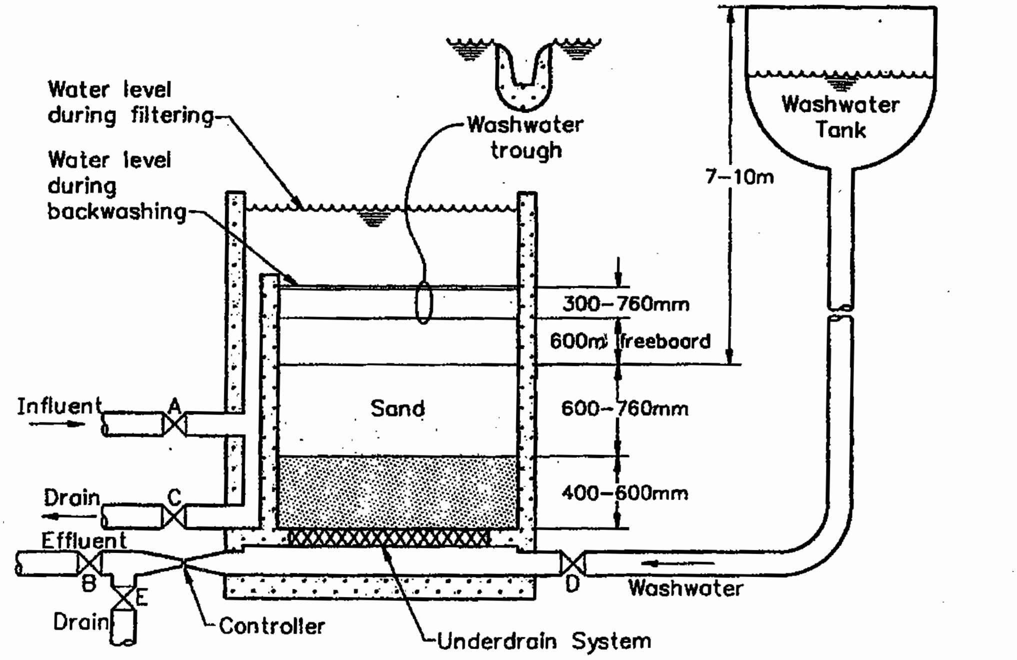
Filters are also classified according to the direction of flow—upflow, downflow, and center feed—and according to driving force—gravity and pressure. The filter bed can be made stratified or unstratified by backwashing. If air–water is used simultaneously, the fine and coarse particles will remain mixed in the settled bed. In mono-, dual-, or multibed filters, if air is stopped near the end of the backwash cycle and water flow is continued, the bed will stratify.[44,45]

Filter Media. The most important characteristics for selection and performance of filter media are effective grain size and uniformity coefficient. The effective size d_{10} is the sieve size that passes 10 percent media by weight. The uniformity coefficient UC is the ratio of 60 percentile size (d_{60}) and 10 percentile (d_{10}) sizes.

Filter Backwash System. Filter backwash design is important to properly clean the filter media and conserve the backwash water. Auxiliary surface washers are used to break and disturb the media. The surface-wash cycle generally starts 1–2 min before the backwashing cycle and is stopped within 2 min after the start of the backwash cycle. Auxiliary air scouring is also used for vigorous washing action and is continued well into the backwashing cycle. A typical air flow rate is 10–16 $m^3/m^2{\cdot}min$.

Design Parameters. Filters are now extensively used to polish secondary effluent. It is also an important pretreatment process before carbon adsorption, ion exchange, and mem-

(a)



Operation cycle	Valve				
	A	B	C	D	E
Normal filtration	Open	Open	Closed	Closed	Closed
Backwash	Closed	Closed	Open	Open	Closed
Rinse	Open	Closed	Closed	Closed	Open

(b)

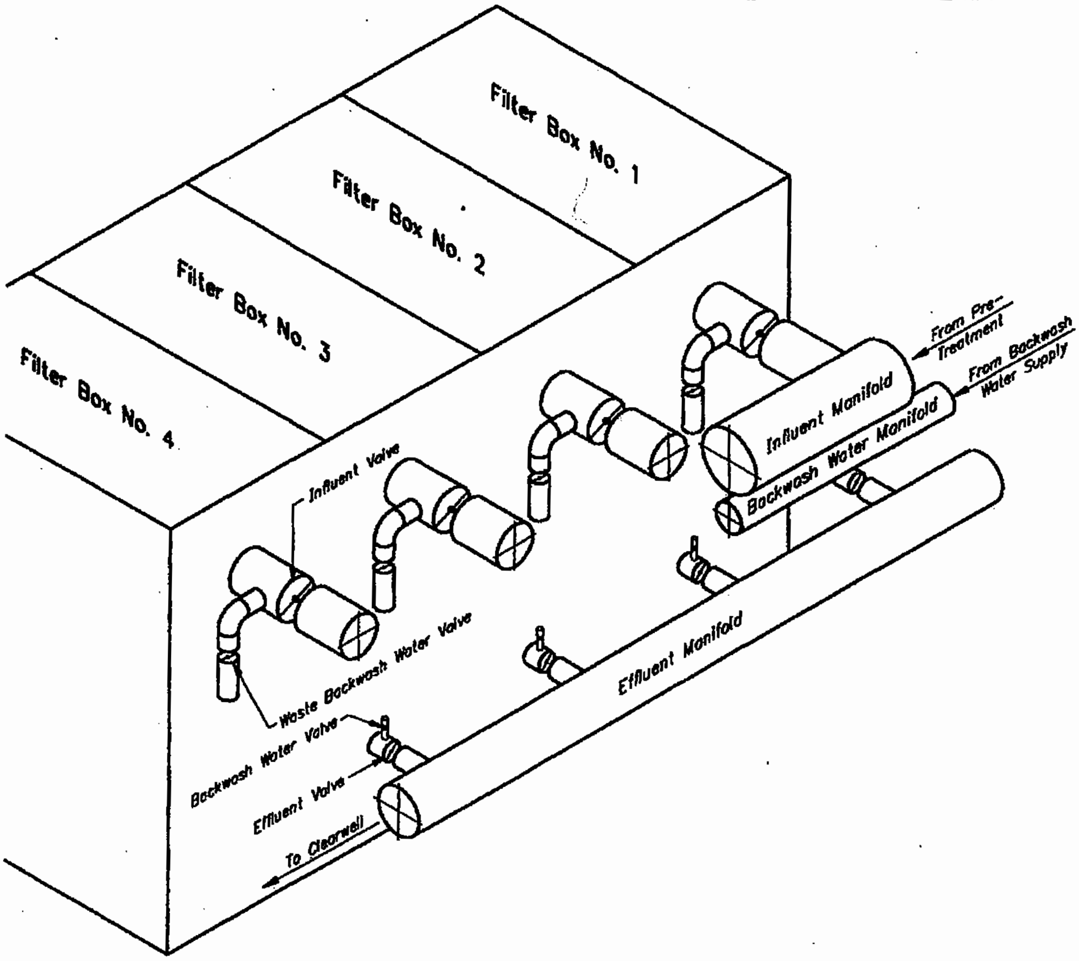


Figure 24-8 Filter components and details: (a) cross section of a rapid sand filter and operational features; (b) typical arrangement of four filters and pipings.

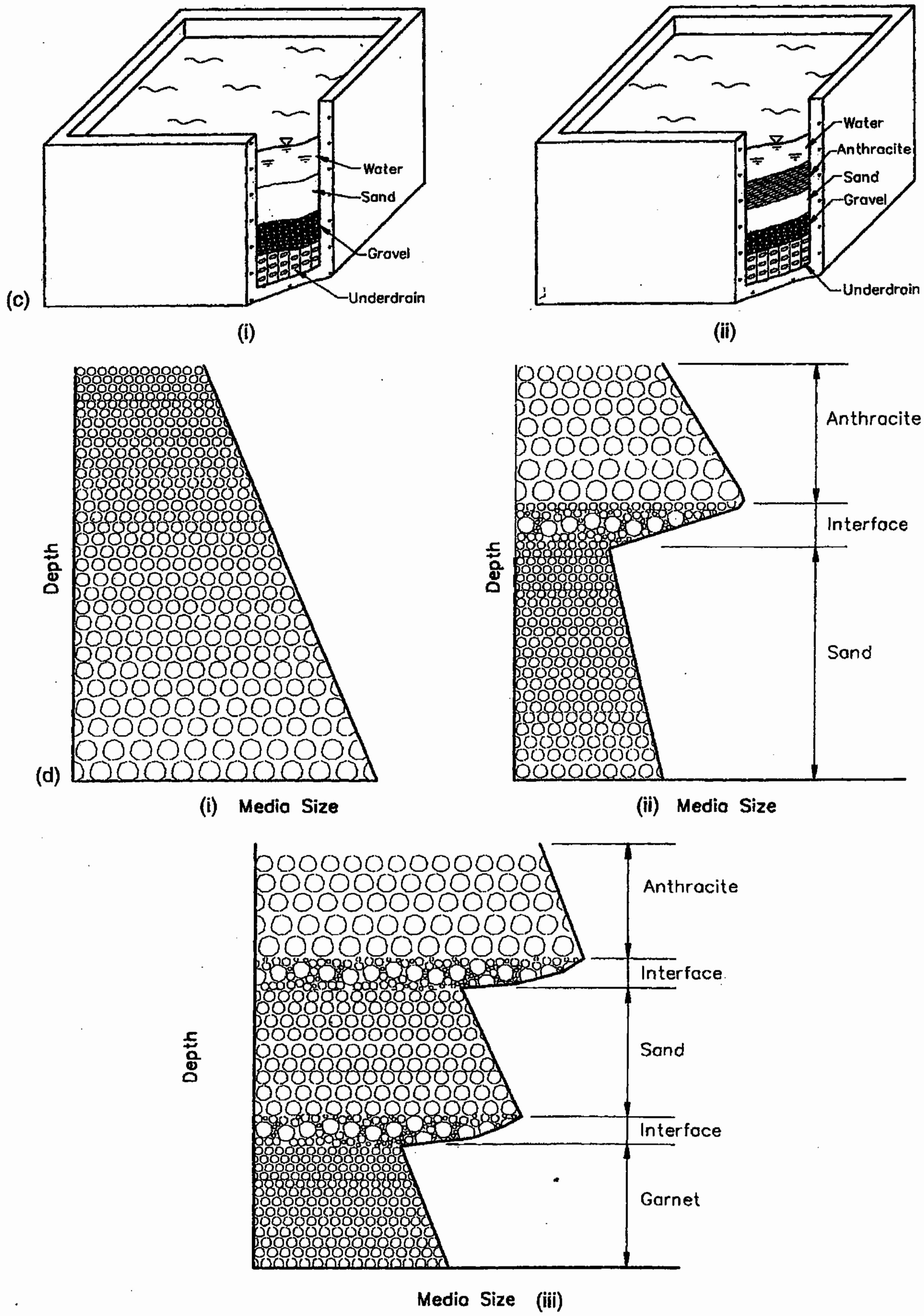


Figure 24-8—cont'd (c) cross sections of rapid and high-rate filters: (i) rapid sand filter, (ii) high-rate dual media filter; (d) bed stratification after filter backwash: (i) mono-medium, (ii) dual-medium, (iii) tri-medium.

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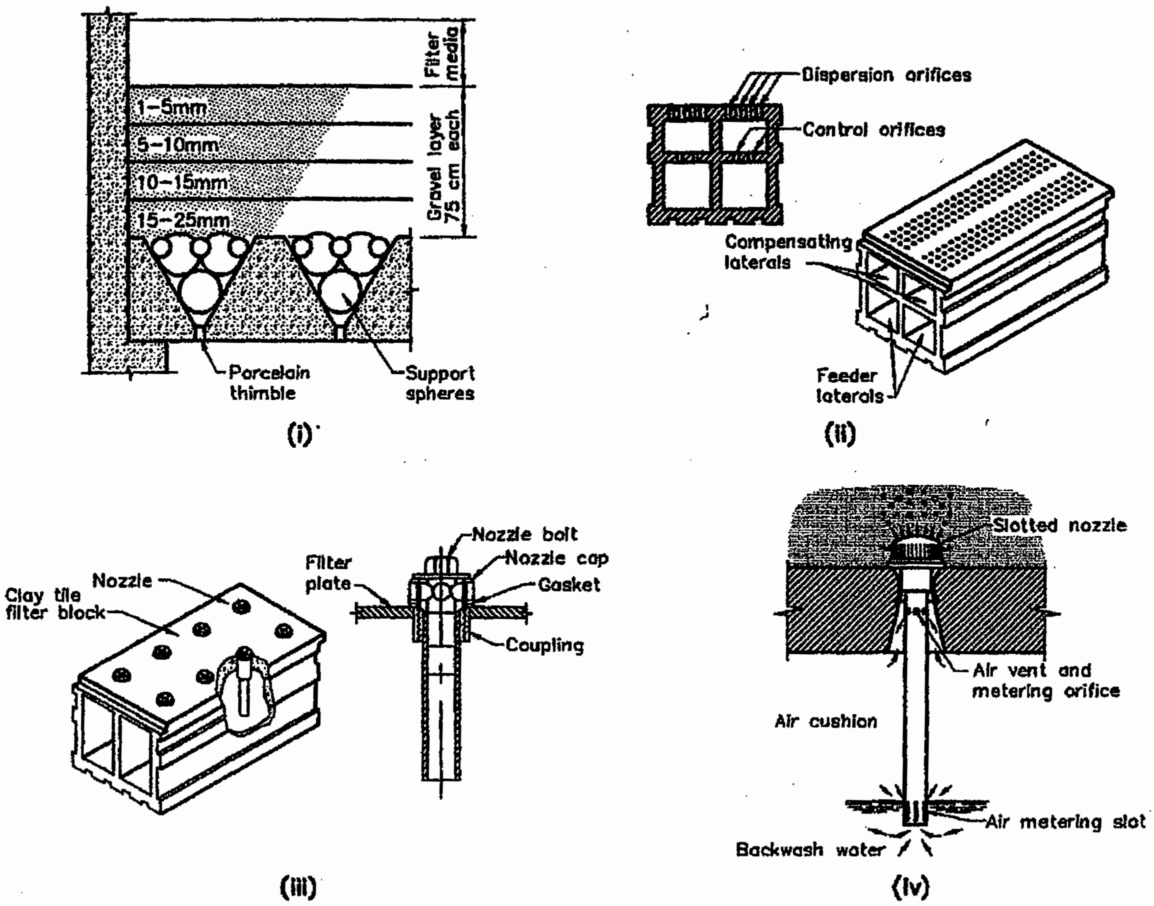


Figure 24-8—cont'd (e) various types of under drain systems with and without gravel support: (i) and (ii) with gravel support and (iii) and (iv) without gravel support.

brane processes. Proper design and operation of filtration systems is therefore very essential. Typical design data on mono and dual filters are summarized in Table 24-5. Basic design features are shown in Figure 24-8. The filter backwash flow to fluidize the filter bed depends upon the grain size and the type of media. The minimum backwash velocity needed to fluidize a 2-mm-size granular sand media is 2 $m^3/m^2 \cdot min$. The minimum velocity to fluidize dual media is 1 $m^3/m^2 \cdot min$. Air–water backwash requires a much lower water velocity. Sand media of 1.00 mm effective size will require water and air flow rates of 0.4 and 13 $m^3/m^2 \cdot min$, respectively.[a] An anthracite bed of 1.10 mm effective size will need water and air flow rates of 0.3 and 8 $m^3/m^2 \cdot min$, respectively.[7,42-45]

Protrietary Filter Design. Several major types of propriety filters are currently available that are in use for polishing the effluents from biological or physical-chemical treatment processes. Among these are traveling-bridge, continuous backwash filter; continuous backwash, top- or bottom-feed filter; center flow filter; pulse-bed filter; and pressure filter. Many of these filters are shown in Figure 24-9.

[a] $m^3/m^2 \cdot min \times 24.54 = gal/ft^2 \cdot min$.

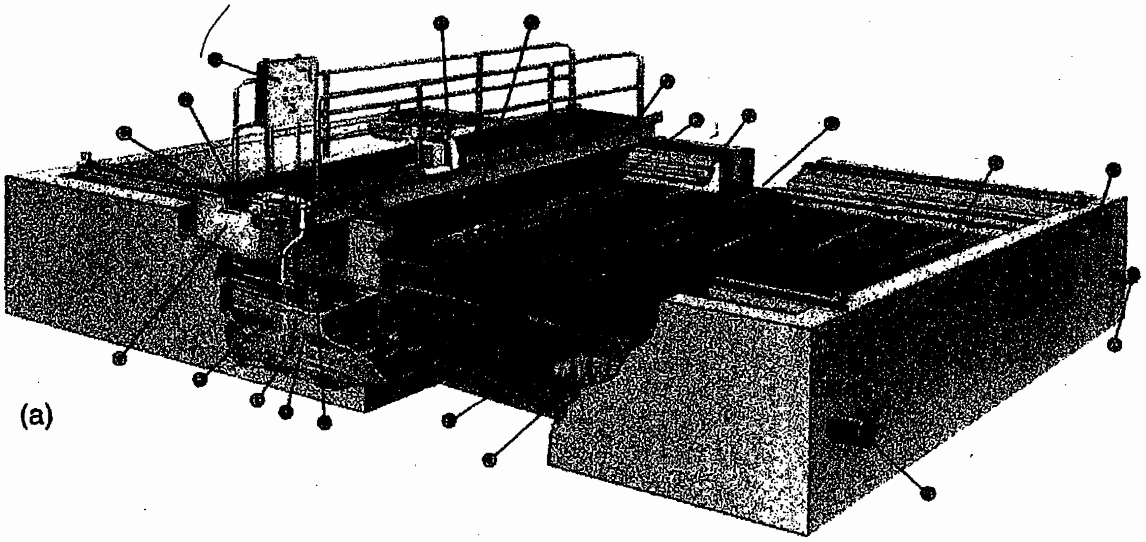
TABLE 24-5 Basic Design Features of Mono- and Dual-Media Filters

Filter Type	Design Factor	Value: Range	Value: Typical
Mono-bed			
Shallow-bed of sand	Total depth, cm	25–30	28
	Filtration rate, $m^3/m^2 \cdot min$	0.08–0.16	0.12
	d_{10}, mm	0.3–0.6	0.5
	UC	1.2–1.6	1.5
Shallow-bed of anthracite	Total depth, cm	30–50	40
	Filtration rate, $m^3/m^2 \cdot min$	0.1–0.2	0.15
	d_{10}, mm	0.8–1.5	1.3
	UC	1.3–1.8	1.6
Medium-bed[a] of sand	Total depth, cm	50–75	60
	Filtration rate, $m^3/m^2 \cdot min$	0.08–0.17	0.12
	d_{10}, mm	0.4–0.8	0.65
	UC	1.2–1.6	1.5
Medium-bed of anthracite	Total depth, cm	60–90	75
	Filtration rate, $m^3/m^2 \cdot min$	0.12–0.35	0.17
	d_{10}, mm	0.8–2.0	1.30
	UC	1.3–1.8	1.60
Deep-bed of sand	Total depth, cm	90–180	120
	Filtration rate, $m^3/m^2 \cdot min$	0.13–0.42	0.20
	d_{10}, mm	2–4	3
	UC	1.3–1.8	1.6
Dual-medium			
Anthracite	Filtration rate, $m^3/m^2 \cdot min$	0.1–0.45	0.25
	Depth, cm	30–75	60
	d_{10}, mm	0.8–2.0	1.3
	UC	1.3–1.8	1.6
Sand	Depth, cm	15–30	30
	d_{10}, mm	0.4–0.8	0.7
	UC	1.2–1.6	1.5

[a]Also called conventional.
$m^3/m^2 \cdot min \times 24.54 = gal/ft^2 \cdot min$.
Source: Refs. 7 and 43–45.

Microstrainers. Microstrainers consist of a woven stainless steel or polyester plastic fabric having openings ranging from 20–35 μm, mounted over a rotating drum that is held horizontally in a tank. The influent enters the drum interior through one end and flows out through the filtering fabric. The solids are retained in the inside of the screen

A. Influent Line
B. Influent Ports
C. Influent Channel
D. Compartmented Filter Bed
E. Sectionalized Underdrain
F. Effluent and Backwash Ports
G. Effluent Channel
H. Effluent Discharge Line
I. Backwash Shoe
J. Backwash Pump Assembly
K. Washwater Hood
L. Washwater Hood Assembly
M. Wastewater Discharge Pipe
N. Washwater Trough
O. Washwater Discharge
P. Mechanism Drive Motor
Q. Backwash Support Retaining Springs
R. Pressure Control Springs
S. Control Instrumentation
T. Traveling Backwash Mechanism



(a)

Figure 24-9 Propriety Filtration Systems: (a) traveling bridge automatic backwash ABW® filter (courtesy Infilco Degremont Inc.).

Continued

rotating up to 4 rpm. The solids are washed by a jet of filtered water on the top of the entire length of the drum. The washwater is collected in a hopper and returned to the plant. The washwater is 3–5 percent of the total effluent volume.[43]

Equipment. Major equipment for filtration devices includes filter housing, filter media, underdrain system, backwash system, flow controller and so on. The components of gravity filtration systems are illustrated in Figure 24-8.

Design Parameters. The basic design factors for the filtration device are average and peak design flow, influent quality, filtration media, filtration rate, applied head, allowable head losses, and backwash flow rate.

24-3-8 Carbon Adsorption

Process Description. Carbon adsorption is used to remove soluble refractory organics. The process consists of entrapping organic material onto the carbon surface. The most common method currently used is granular activated carbon column. The treated wastewater is percolated through the column until the column becomes saturated with organic material. It is then removed from service, and the activated carbon is regenerated by burning off the organic material in a special furnace. Approximately 5 percent loss of carbon can be expected with each cycle; thus, new material must be added.

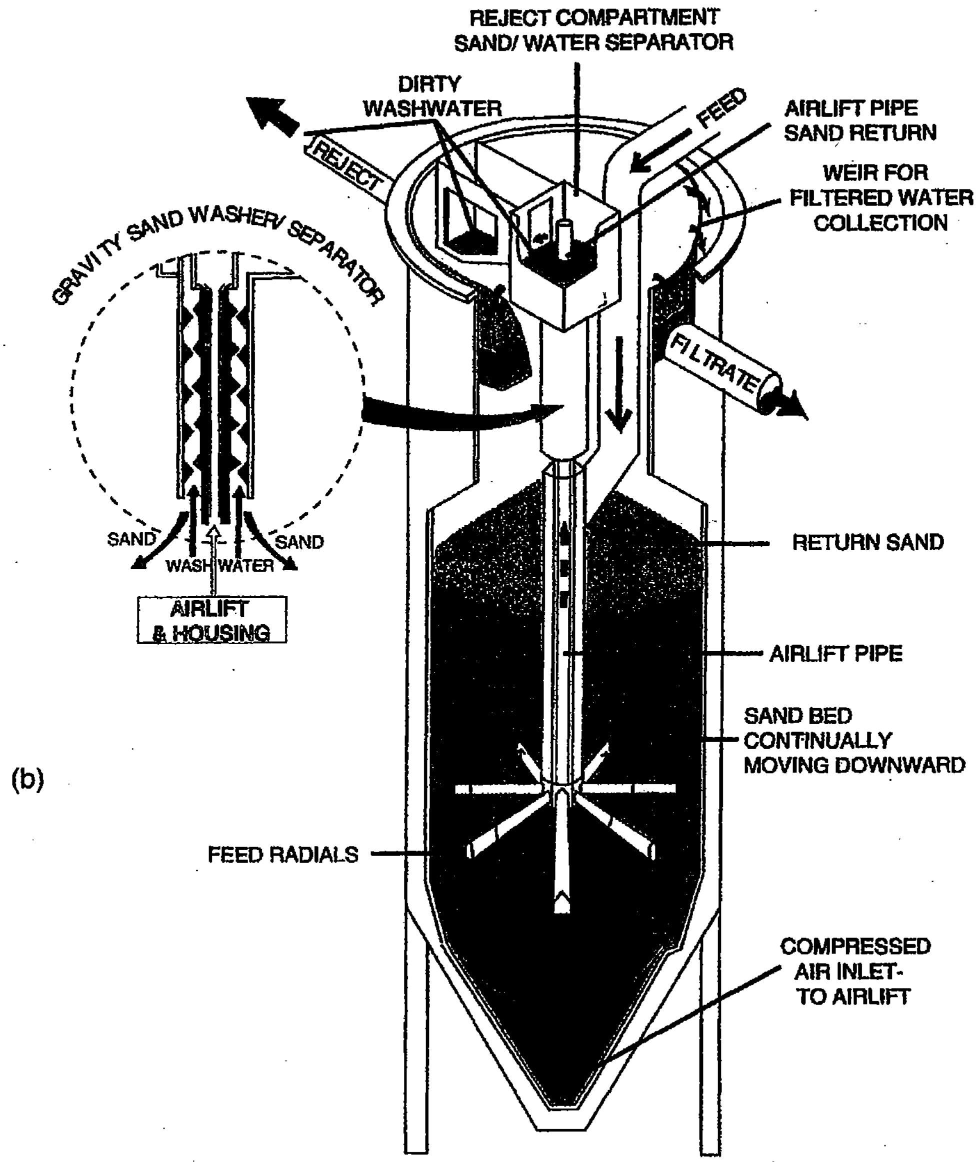


Figure 24-9—cont'd (b) DynaSana top-feed filter (courtesy Parkson Corp.).

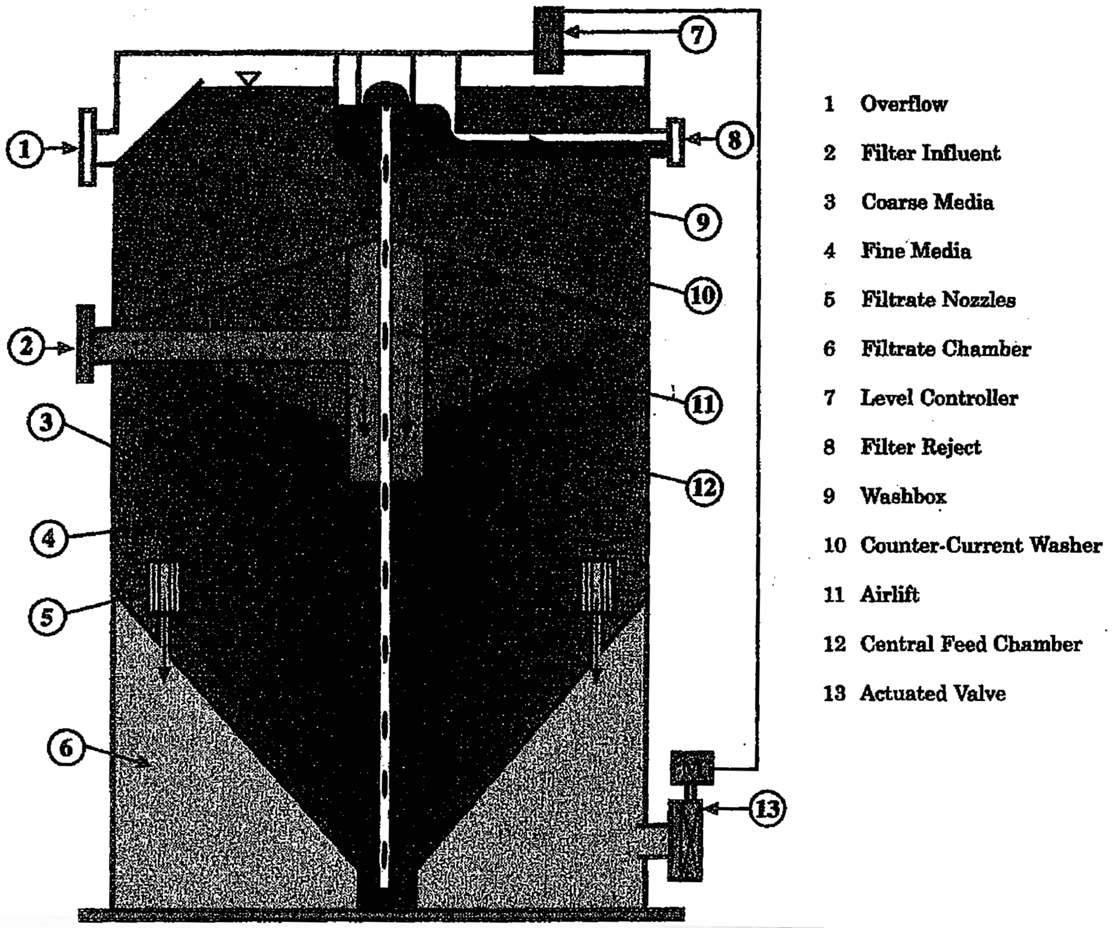
Continued

Activated carbon can effectively remove bacteria and viruses. It also removes organometallic compounds, pesticides, chlorinated compounds, chlorine, and many other compounds that are not removed in a conventional secondary treatment plant.[46-48]

Equipment. Following is the list of equipment commonly needed for carbon adsorption system: carbon column, granular activated carbon, feed and backwash pumps and piping, and carbon regeneration system.

Design Parameters. Common design parameters include average and design flow, influent characteristics, effluent quality, contact time, and adsorption capacity of carbon.

(c)



(d)

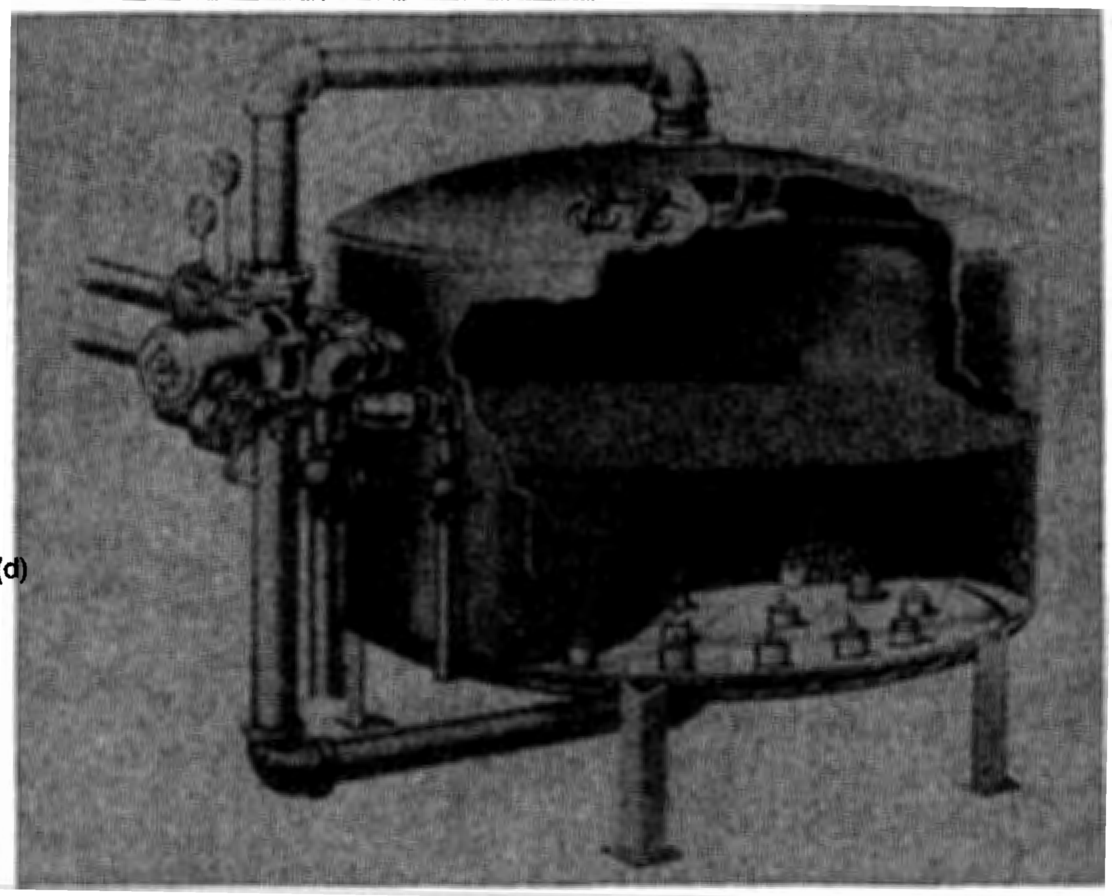


Figure 24-9—cont'd (c) Centra-flo™ Gravity Sand Filter (courtesy Applied Process Technology, Inc.); (d) pressure filter assembly (courtesy The Permutit Company, Inc.).

(e)

Figure 24-9—cont'd (e) operational and equipment details of microstrainer (courtesy Envirex, U.S. Filter).

24-3-9 Ion Exchange for Demineralization

Process Description. Ion exchange is a demineralization process in which the cations and anions in wastewater are selectively exchanged for ions in the insoluble resin material. When the resin capacity is used up, it is regenerated by using high concentrations of the original ion that is exchanged from the resin.

Ion exchange resins are cationic if they exchange positive ions and anionic if they exchange negative ions. In the hydrogen cation exchange process, hydrogen ions are exchanged for positive ions (Ca^{2+}, Mg^{2+}, Na^{+}, etc.). The hydrogen cation exchanger is regenerated with a mineral acid. The most widely used and the cheapest regenerant is sulfuric acid.[49,50]

In anion exchangers, the negative ions (Cl^{-}, NO_3^{-}, SO_4^{2-}, etc.) are exchanged by OH^{-} ions. The main types of anion exchangers are (1) weakly basic anion exchangers and (2) strongly basic anion exchangers. The weakly basic anion exchangers remove strongly ionized acids (HCl, H_2SO_4, etc.), but they will not remove weakly ionized acids (H_2CO_3, H_2SiO_3, etc.). These exchangers are regenerated with a solution of soda ash (Na_2CO_3). Strongly basic anion exchangers remove both strongly and weakly ionized acids. These resins are regenerated with caustic soda solution.[50]

Ion exchangers are granular, packed-bed columns. The wastewater is trickled downward under pressure. When exchange capacity of the resin is exhausted, the column is backwashed to remove trapped solids, and then the beds are regenerated. It is possible to produce high-quality demineralized water. Synthetic cation and anion resins perform better than the naturally occurring zeolites. The cation and anion resin beds for demineralization may be arranged in series in separate exchange columns or may be mixed in a single reactor. The typical bed depths are 1–2 m (3.3–6.5 ft), and the flow rate is 0.20–

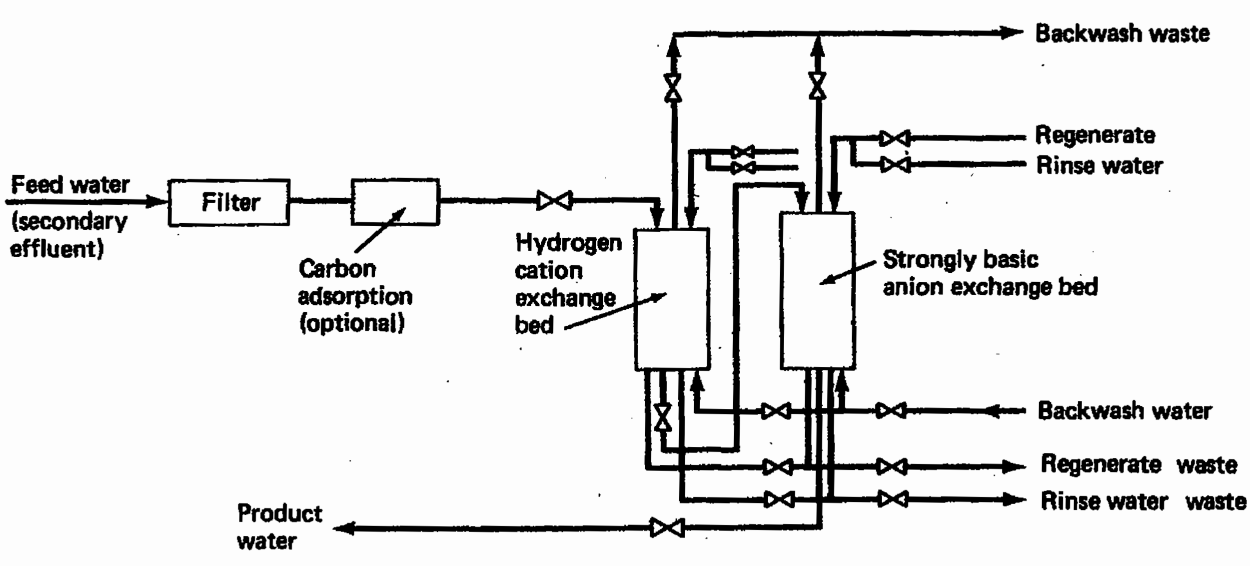


Figure 24-10 Typical Flow Diagram of Ion Exchange Demineralization System.

0.40 $m^3/m^2 \cdot min$ (5–10 $gal/ft^2 \cdot min$). Total ion exchange capacity of commercially available resins are 50,000–80,000 g/m^3 (as $CaCO_3$).[49]

A high concentration of organic matter in effluent may plug the bed and blind the resin. Filtration and carbon adsorption of secondary effluent may be necessary prior to ion exchange. A schematic flow diagram of an ion exchange system is given in Figure 24-10.

Equipment. Equipment commonly needed for the ion exchange demineralization process includes gravity filters, carbon adsorption (may not be needed), ion exchange beds, pressure pumps, regenerant solution and pumps, and backwash and rinse water systems.

Design Parameters. Common design parameters include average design flow, total dissolved solids, ion exchange capacity of the resins, degree of demineralization needed, and flow rate through the beds.

24-3-10 Distillation

Process Description. Distillation is the oldest demineralization process. It consists of evaporating part or all of the water from a saline solution and subsequent condensation of mineral-free vapor. The energy requirements are very high for the system. One modification of the simple distillation process is multiple-effect evaporation where water is evaporated in different stages under a small vacuum created by the condensed steam from the previous effect.[51] The latent heat of condensation is used to preheat the incoming water. This method has been under extensive commercial development for many years and used for production of fresh water from the sea. This process is shown in Figure 24-11(a). Another modification is distillation with vapor compression [Figure 24-11(b)]. This system utilizes the latent heat of the compressed steam to preheat and

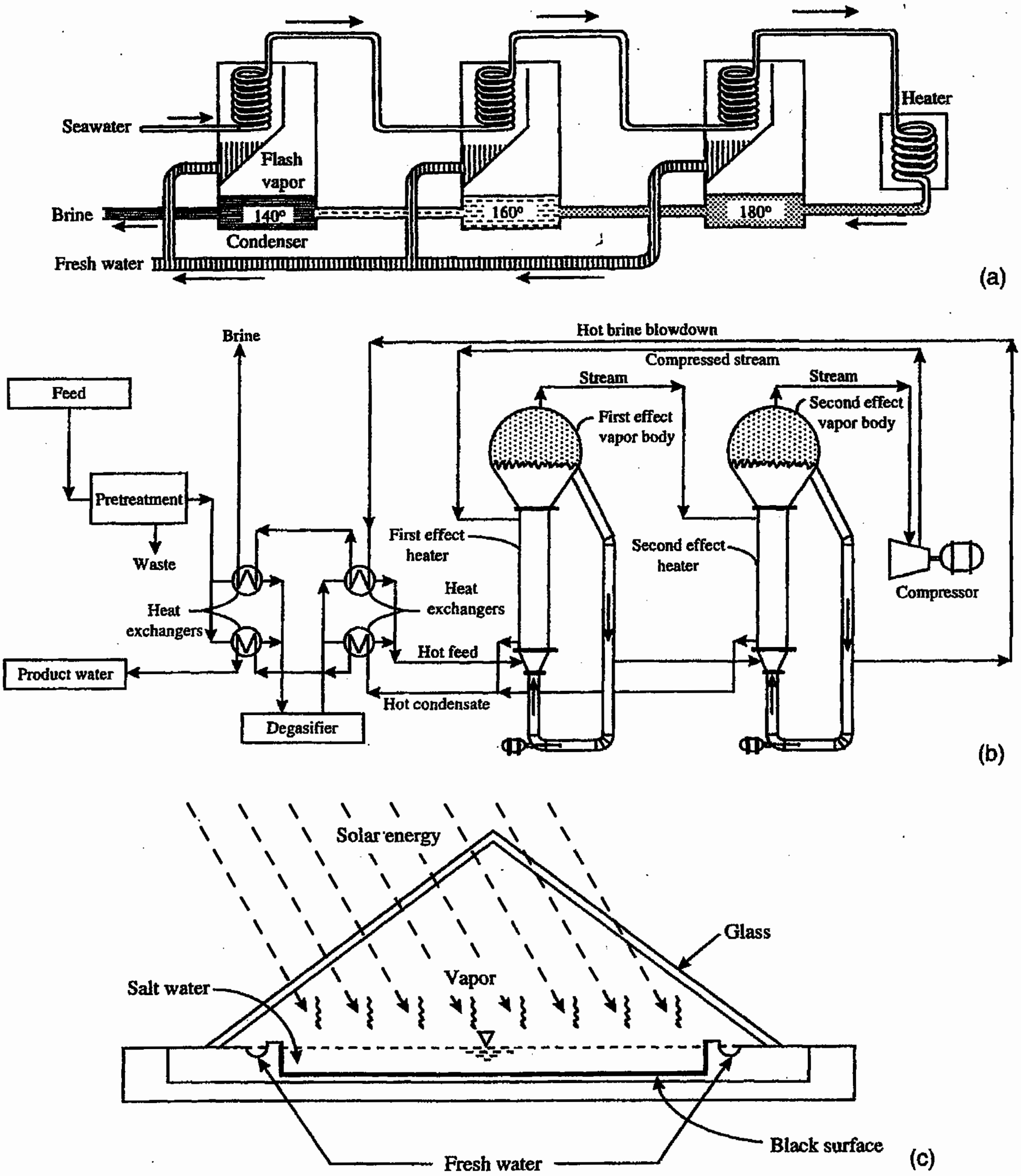


Figure 24-11 Water Distillation: (a) multi-effect distillation, (b) forced-circulation vapor-compression distillation, and (c) solar distillation (from Ref. 30).

evaporate the incoming water. Both modifications conserve energy, but the energy requirements are still quite large.

Solar distillation offers the advantage of free energy. The basic principles of solar distillation had been known for many years before the first significant installation unit was made in Chile in 1872 for recovery of fresh water from saline water. The simplest solar distillation unit consists of a shallow blackened pan with sloping glass exposed to the sun [Figure 24-11(c)]. The evaporated water condenses on the sloping glass sheets (since it is cooler) and runs down to the collecting channels for recovery of condensate. With the present techniques, about 3 kg of distillate can be produced per day per m^2 of evaporating pan. This gives an approximate operating efficiency of 35 percent at an average solar radiation of 47.5 calories/$cm^2 \cdot d$. Such yields have been achieved in practice with experimental and full-scale large units operating over long periods.[52,53]

Equipment. The equipment commonly needed for distillation includes a raw liquid feed system, still or boiler, liquid heating and evaporating system, vapor separation and condensation, and condensate and brine recovery systems.

Design Parameters. Common design parameters for the distillation process are design flow, raw water temperature, number of stages, the temperature drop allowed in each stage, efficiency of heat exchangers, and a brine handling and disposal system.

24-3-11 Reverse Osmosis and Ultrafiltration

Process Description. *Reverse osmosis* is a membrane-separation process in which a semipermeable membrane is used to permeate high-quality water while rejecting the passage of dissolved solids. *Osmosis* is the natural passage of water from a weaker solution to a stronger solution to equalize the chemical potentials of the water in the membrane-separated solution. *Osmotic pressure* is the driving force for osmosis to occur. In reverse osmosis an external pressure greater than the osmotic pressure is applied to the solution, causing water to flow against the natural direction through the membrane, thus producing high-quality demineralized water. The membrane rejects most of the ions and molecules while permitting acceptable rates of water passage.

Many types of membranes have been developed, but cellulose acetate and polyamide (nylon) are currently the most widely used membrane materials. Reverse osmosis modules suitable for water treatment involve arrangement of membranes and their supporting structures so that feed water under high pressure up to 10,000 kN/m^2 (1500 lb/in^2) can pass through the membrane surface while product water is collected from the opposite face without brine contamination. Four different types of module designs have been developed: plate and frame, large tube, spiral wound, and hollow fine fiber. The principle of reverse osmosis and one commercially available membrane assembly are shown in Figure 24-12.[54-56]

Typical membranes are approximately 100 μm thick, having a surface skin of about 0.2 μm thick that serves the rejecting surface. The remaining layer is porous and spongy and serves as backing material. The hollow fine fibers have outer and inner diameters of 50 and 25 μm. Basic design and operational information of reverse osmosis systems are summarized in Table 24-6.

Ultrafiltration is a process similar to reverse osmosis. It applies to larger molecules in solution (0.002–10 μm range). Ultrafiltration is actually a physical screening process using a relatively coarse membrane. The applied pressure is normally below 1000 kN/m^2 (150 lb/in^2).[54] Other membrane processes used in the wastewater treatment field are microfiltration and nanofiltration.

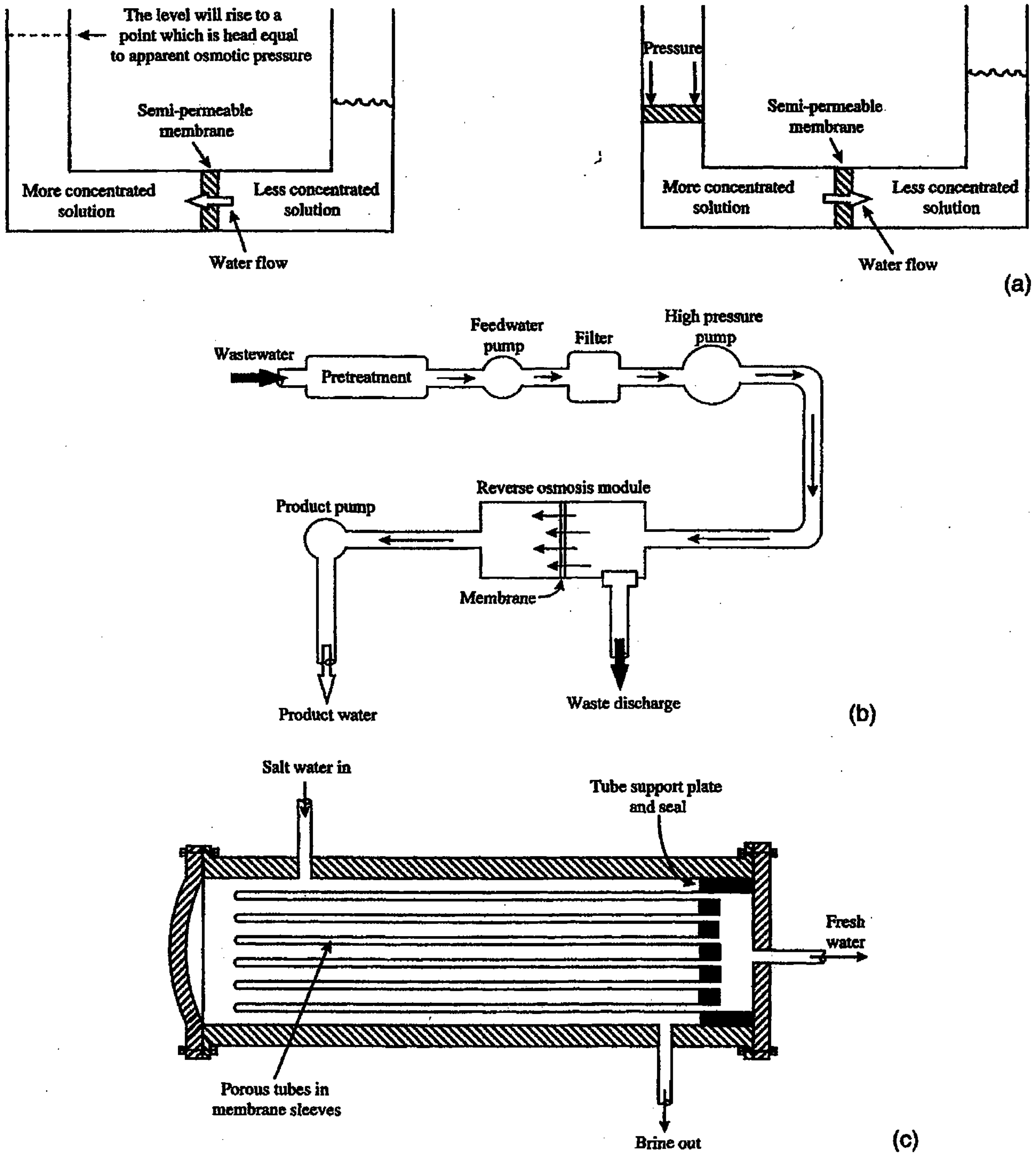
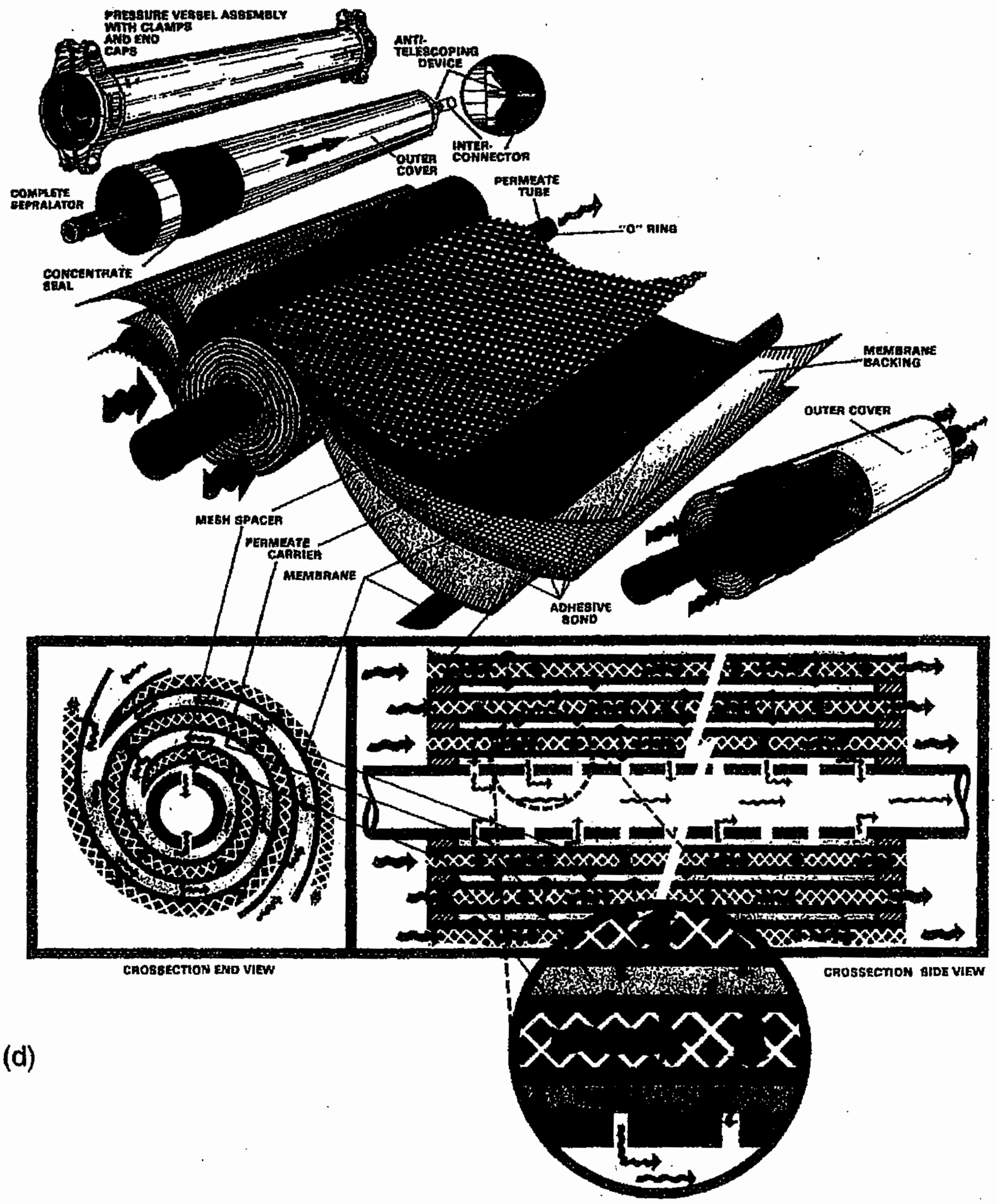


Figure 24-12 Principle of Reverse Osmosis and Commercially Available Membrane Assembly: (a) principle of osmosis and reverse osmosis, (b) schematic process diagram of reverse osmosis system, (c) membranes mounted on porous rod.

Continued



(d)

Figure 24-12—cont'd (d) spiral wound or scroll configuration (courtesy Osmonics, Inc.).

Equipment. The major equipment for a reverse osmosis system includes a membrane module with support system, pretreatment system, high-pressure pump and piping, and a brine-handling and disposal system.

Design Parameters. The design of a reverse osmosis system to handle water treatment requires consideration of the following variables: plant capacity, salinity of raw water and pretreatment needs, product recovery rate, rejection rate, applied pressure, feed water temperature, and method of brine disposal.

The design procedure for a reverse osmosis plant includes the following: (1) determine raw water quality and select pretreatment processes, (2) select a reverse osmosis system after consultation with the equipment manufacturers, (3) select

TABLE 24-6 Summary of Design and Operation Parameters of Reverse Osmosis System

	Parameters	Range	Typical
Pressure	Higher pressure gives greater flux, but pressure capability of membranes is limited.	2000–7000 kN/m^2 (300–1000 psig)	4140 kN/m^2 (600 psig)
Temperature	Higher temperatures give greater flux. The life of membrane is reduced at temperatures greater than 38°C.	16–38°C	21°C
Packing density	It is the area of the membrane that can be placed per unit volume of the pressure vessel.	164–1640 m^2/m^3 (50–500 ft^2/ft^3)	820 m^2/m^3 250 ft^2/ft^3
Flux	Flux is recovery of product water per unit area of the membrane. Flux tends to decrease with length of run. Flux may decrease by 50 percent after 2 years' operation.	0.1–5 $m^3/m^2{\cdot}d$ (2.5–120 gpd/ft^2)	0.5–1.5 $m^3/m^2{\cdot}d$ (12–35 gpd/ft^2)
Recovery factor	Recovery factor is the ratio of the volume of product water of total volume treated. At a higher recovery factor, there may be more salt in the product water.	75–95 percent	80 percent
Salt rejection	Salt rejection depends on the type of membrane and salt concentration gradient.	85–99 percent	95 percent
Pretreatment and feed water	The feed water must be free of scale-forming constituents (calcium, magnesium, iron, manganese, silicon, etc.). Concentrations of such ions should be reduced by pretreatment. TDS should be less than 10,000 mg/L. Organics, microorganisms, and oil and grease must be removed to prevent coating and fouling of membranes. Cellulose acetate membranes are subject to hydrolysis at high and low pH.	4.5–5.5	4.7

Continued

TABLE 24-6 Summary of Design and Operation Parameters of Reverse Osmosis System—cont'd

Parameters		Range	Typical
	Turbidity and particle size must be controlled.		
	pH	4.5–5.5	4.7
	Turbidity	—	<1 NTU
	Particle size	—	<25 μm
Feed water stream velocity	High velocities and turbulent flow are necessary to minimize concentration polarization at the membrane surface.	0.01–0.8 m/s (0.04–2.5 ft/s)	—
Life	Life of the membrane depends on quality of feed water.	—	2 years
Cleaning	Membranes must be cleaned periodically by depressurization, high-velocity water, air–water mixture, and chemical solutions.	periodic	—
Power	Power is needed for pumping at high operational pressures.	2–5 kWh/m^3 (9–17 kWh/1000 gal)	—

operating parameters (flux rate, rejection factor, applied pressure, system life, performance level, etc.), (4) calculate system size, (5) determine power requirements, (6) select a brine disposal system, and (7) estimate system economics, including amortization of capital cost, labor, supplies and chemicals, and brine disposal.

Example. A municipal water supply source has a total dissolved solids (TDS) concentration of 1000 mg/L. Develop the design and size various components of a reverse osmosis system to produce finished water having a TDS concentration of less than 300 mg/L. The plant capacity is 19,000 m^3/d. Use the following data:

Plant information	
Plant design capacity	19,000 m^3/d (5 mgd)
Feed water temperature	27°C
TDS raw water	1000 mg/L
TDS finished water	300 mg/L
Manufacturer information	
Recovery factor, R	75 percent
Salt rejection factor, S	95 percent
Design pressure	4140 kN/m^2 (600 psig)
Flux rate	0.82 m^3/m^2·d (20 gpd/ft^2)

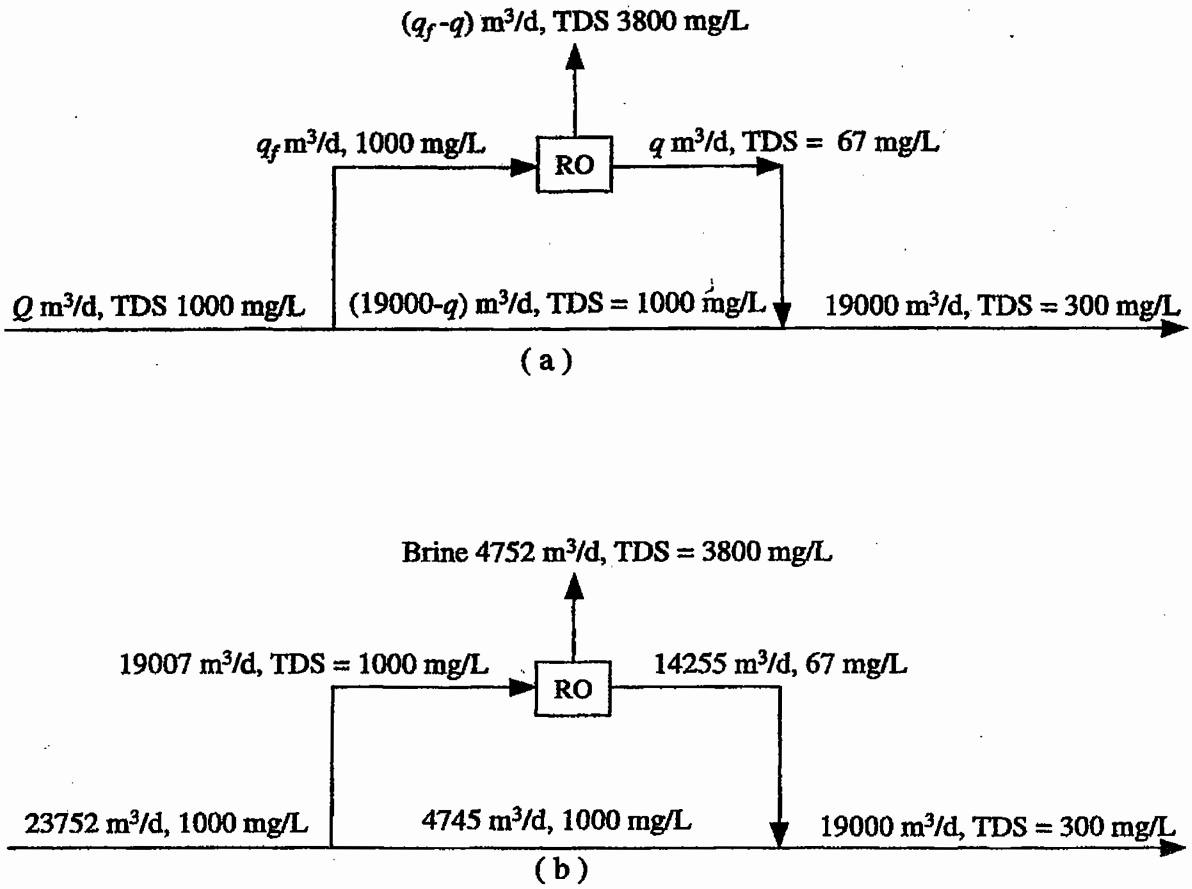


Figure 24-13 Mass Balance around the RO Unit in the Example: (a) mass balance with capacity "q" of RO unit and (b) final result of mass balance.

Solution

1. Compute the system size using split treatment (Figure 24-13).

$$\begin{aligned}\text{TDS after RO unit} &= 1/R\ (\text{raw water TDS})\ (1 - S)\\ &= 1/0.75\ (1000\ \text{mg/L})\ (1 - 0.95)\\ &= 67\ \text{mg/L}\end{aligned}$$

Using the flow and concentration for the split treatment in Figure 24-13(a), calculate the capacity "q" of the RO unit.

$$\begin{aligned}(19{,}000 - q)\ \text{m}^3/\text{d} \times 1000\ \text{mg/L} + q\ \text{m}^3/\text{d} \times 67\ \text{mg/L}\\ = 19{,}000\ \text{m}^3/\text{d} \times 300\ \text{mg/L}\\ q = 14{,}255\ \text{m}^3/\text{d}\end{aligned}$$

$$\text{Feed water flow to RO system, } q_f = \frac{14{,}255}{0.75} = 19{,}007\ \text{m}^3/\text{d}$$

$$\begin{aligned}\text{Brine flow } (q_f - q) \text{ to be disposed of} &= (19{,}007 - 14{,}255)\ \text{m}^3/\text{d}\\ &= 4752\ \text{m}^3/\text{d}\end{aligned}$$

$$\text{TDS in brine} = \frac{0.95}{1 - 0.75} \times 1000 \text{ mg/L}$$

$$= 3800 \text{ mg/L}$$

$$\text{Total raw water flow, } Q = q_f \text{ m}^3\text{/d} + (19{,}000 - q) \text{ m}^3\text{/d}$$

$$= 19{,}007 \text{ m}^3\text{/d} + (19{,}000 - 14{,}255) \text{ m}^3\text{/d}$$

$$= 23{,}752 \text{ m}^3\text{/d}$$

The final results of material mass balance is given in Figure 24-13(b).

For flux rate of 0.82 $m^3/m^2{\cdot}d$ (20 gpd/ft^2), the area of the RO membrane $= \dfrac{14{,}255 \text{ m}^3\text{/d}}{0.82 \text{ m}^3\text{/m}^2{\cdot}\text{d}} = 17{,}384 \text{ m}^2$

Assuming a packing density of 820 m^2/m^3,

$$\text{Total module volume} = 17{,}384 \text{ m}^2/820 \text{ m}^2/\text{m}^3 = 22 \text{ m}^3$$

Assuming a 0.03 m^3/module,

$$\text{Total number of module required} = 22 \text{ m}^3/0.03 \text{ m}^3\text{/module} = 733 \text{ module}$$

If 15 modules are used per pressure vessel,

Total number of pressure vessels needed

$$= \frac{733 \text{ module}}{15 \text{ module/pressure vessel}} = 48.9 \text{ pressure vessel}$$

Provide 50 pressure vessels.

2. Compute the power consumption.

Water power = design pressure × flow pressurized $= 4140 \dfrac{\text{kN}}{\text{m}^2} \times \dfrac{19007 \text{ m}^3\text{/d}}{(60 \times 60 \times 24) \text{ s/d}} = 911 \text{ kN·m/s or kW}$

Brake power assuming 95 percent pump efficiency $= \dfrac{911 \text{ kW}}{0.95} = 959 \text{ kW}$

Motor power assuming 88 percent motor efficiency $= \dfrac{959 \text{ kW}}{0.88} = 1090 \text{ kW}$

If it is desired to operate the system at a higher design pressure, the power calculations must be revised.

24-3-12 Electrodialysis

Process Description. Electrodialysis is also a demineralization process. In this process, an electrical potential is used to transfer the ion through ion-selective membranes.[55,56]

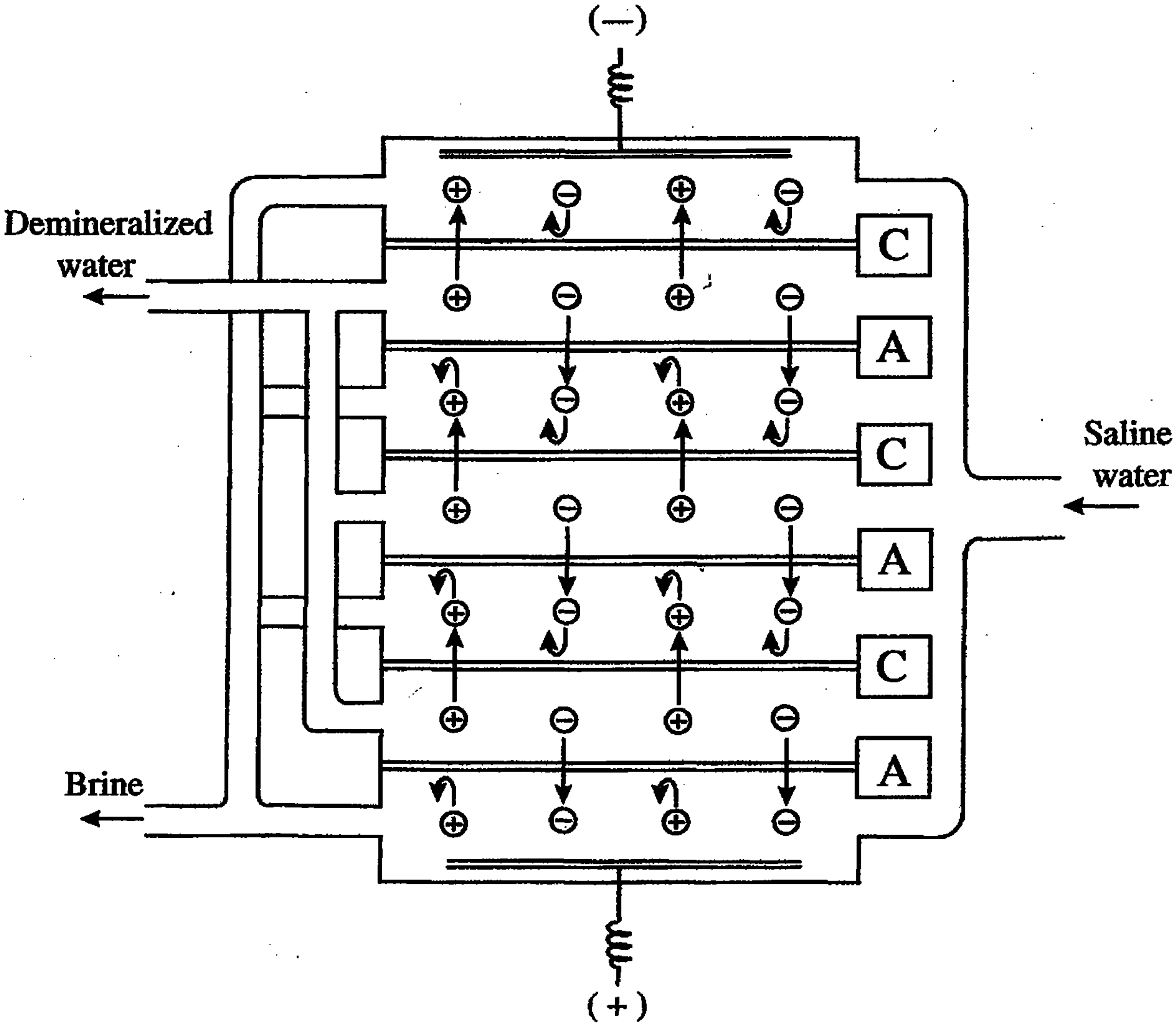


Figure 24-14 Mechanism of the Electrodialytic Process.

This process is most practical and widely used to treat brackish waters. The energy requirement is directly proportional to the concentration of salts in the water being treated. An electrical voltage is applied in cells (also called stacks). The cells contain alternatively arranged cation- and anion-permeable membranes. The cations and anions in feed liquid migrate to the opposite poles. They pass the cation- and anion-selective membranes, thus producing clean water and brine streams. To avoid membrane fouling, the water must be pretreated as is done in reverse osmosis. The schematic flow diagram of an electrodialysis system is given in Figure 24-14.

Equipment. The major equipment for the electrodialysis system includes a membrane stack with support system, pretreatment system, and brine handling and disposal system.

Design Parameters. The design parameters for electrodialysis systems are average design flow, salinity, product recovery rate, and influent quality to the membrane and pretreatment requirements.

24-3-13 Freeze Process

Process Description. If saline solution is cooled to the freezing point and ice crystals are formed, the dissolved salts remain in the solution. Thus, the brine is of a concentration greater than the initial solution, and the ice contains pure water. This process has been used in colder climates where ice is formed in saline water during the winter. The ice is separated from the brine and melted in summer to produce fresh water. It should be noted that the freezing point of a solution and quality of ice produced depends on the salinity of the solution.[57] Freezing temperature is lower for higher salinity water, and ice crystals produced become coated with the brine at an excessively high salt content.

Equipment. The equipment commonly needed for the freeze process includes a cooling system to freeze the ice slab (applicable to cold climate), an ice separation and ice melting system, and the collection and removal of brine and fresh water.

Design Parameter. Common design parameters for the freeze process are design flow, salt concentration, cooling rate, land area, depth of impoundment needed if natural cooling is used, and brine disposal.

24-4 PROBLEMS AND DISCUSSION TOPICS

24-1 Draw a process train of a secondary wastewater treatment plant. Identify the possible locations where suitable coagulant may be added for precipitation of phosphorus. List the advantages and disadvantages of coagulation at each location.

24-2 In a jar test, municipal wastewater was coagulated using aluminum sulfate $Al_2(SO_4)_3$ $18H_2O$. The phosphorus concentration in the wastewater was 8 mg/L as P. At alum dosages of 89, 136, and 216 mg/L, soluble P levels remaining in the supernatants were 2, 1.5, and 0.4 mg/L, respectively. Calculate the molar ratio Al:P for these removals. Also prepare a plot of molar ratio of Al:P versus log of percent P removed.

24-3 What is the purpose of recarbonation after two-stage lime treatment? Give chemical equations to justify your reasoning.

24-4 Why does nitrification not occur concurrently with BOD reduction in an aeration basin?

24-5 Calculate methanol consumption (kg/d) for denitrification of 0.01 m^3/s well-nitrified effluent containing 18 mg/L nitrate nitrogen and 5 mg/L dissolved oxygen. Nitrite nitrogen is 0.3 mg/L (see Chapter 13).

24-6 Discuss the major limiting factors of ammonia stripping.

24-7 Breakpoint chlorination is used for oxidation of 18 mg/L ammonia-nitrogen in an effluent. Calculate chlorine dosage and how many mg/L chloride ions are added in the effluent.

24-8 List the various types of filtration systems commonly used in the polishing of secondary effluent. Discuss the advantages and disadvantages of microscreens.

24-9 Prepare a list of different chemicals that are removed from secondary effluent by carbon adsorption process (consult Sec. 3-4-2, Table 3-4, and Refs. 7 and 46–48).

24-10 Discuss the advantages and disadvantages of various demineralization processes (ion exchange, distillation, and membrane processes) if used for recovery of potable water from a secondary effluent.

24-11 List the environmental concerns of land treatment of municipal wastewater.

24-12 Describe the major design features, site characteristics, and effluent quality from an overland flow system.

24-13 Calculate the total land area required for a slow-rate irrigation system to treat 0.08 m^3/s primary treated municipal wastewater. Assume the buffer land is 300 percent of theoretical land requirement and the area needed for roads and ditches is 25 percent of theoretical land area. The sewage application rate is 1.5 m per year. Also calculate (a) the theoretical field area required (ha for each 43.8 L/s) and (b) BOD_5 loading kg/ha·d if influent has a BOD_5 of 140 mg/L.

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25

Wastewater Treatment Systems for Small Flows

25-1 INTRODUCTION

The Environmental Protection Agency (EPA) estimates that there are approximately 18 million housing units in the United States that dispose of their wastewater using on-site wastewater treatment and disposal systems. This is about 25 percent of all housing units. Additionally, about one-half million new systems are being installed each year.[1]

It has been estimated that only 32 percent of the total land area in the United States has soils suitable for on-site treatment and disposal of wastewater. In many areas, on-site systems have not received public acceptance; feelings are that these systems are second-rate, temporary, or failure-prone. This perspective may partly be true because many systems are poorly designed, poorly constructed, and inadequately maintained.

Small communities in rural areas are generally faced with many situations that make the centralized sewage collection, treatment, and disposal systems very difficult and, in many cases, quite impractical. As a result, the planning, design, construction, and management of on-site systems for individual homes and small subdivisions and establishments have received much interest in recent years. In this chapter the basic elements of planning, design, construction, and management of on-site wastewater treatment systems for individual homes, cluster housing and small communities, and commercial/institutional establishments are presented.

25-2 WASTEWATER PROBLEMS OF SMALL COMMUNITIES

Small communities with populations of 1000 people or less account for about 32 percent of the number of treatment plants, although total flow treated is less than 0.7 percent.

These communities have faced a variety of financial, technical, and strategic problems that make centralized collection and treatment facilities less practical. The major problems are as follows:[1,2]

1. For effluent disposal in surface waters, the same degree of treatment is required as that for large communities.
2. There is a limited financial base because of lower household income, smaller commercial and industrial taxes, poorer bond rating and higher interest rates, and investment uncertainty because of fewer major employers.
3. Population is spread out, which increases the cost of a collection system.
4. The operation and maintenance cost is high to meet the stringent discharge requirements.
5. These communities do not benefit from the economies of scale that are possible with the construction and operation of larger facilities.
6. The per capita costs of wastewater treatment facilities is significantly higher than that for larger communities. Experience has shown that the per capita cost for a community of 1000 persons is two to four times higher than that for a community of 100,000 people.

25-3 STRATEGY FOR SELECTION OF WASTEWATER TREATMENT AND DISPOSAL SYSTEMS FOR SMALL FLOWS

The wastewater treatment and disposal systems for individual homes, small establishments, and small communities in general fall into two broad categories: (1) unsewered areas and (2) sewered areas. The treatment and disposal methods for each category may vary greatly, depending upon wastewater flow, geology, subsurface soil conditions, and surface water disposal options. The unsewered areas do not have a wastewater collection system and therefore must utilize *on-site* treatment and disposal options. The sewered areas typically have collection systems to convey the wastewater to a suitable location for treatment and disposal or reuse. The treatment and disposal or reuse systems in sewered areas utilize technology similar to that used for small municipal wastewater treatment facilities presented in several chapters of this book. Additional information on collection options and package plants is presented in Secs. 25-8-1 and 25-8-2. The strategy for selection of on-site treatment and disposal options are based on three logical steps: (1) wastewater characterization and facility sizing, (2) site evaluation and selection for on-site effluent disposal, and (3) selection of the most efficient treatment technology that is compatible with the wastewater characteristics and disposal options. The primary criterion for selection of such a system combination is protection of public health while preventing environmental degradation.

In unsewered locations, wastewater from individual dwellings and establishments are usually managed by on-site treatment and disposal systems, the most common of which is the septic tank for partial treatment of the wastewater, followed by a subsurface soil disposal field for final treatment and disposal. A properly designed, constructed, and maintained subsurface absorption system performs reliably over a long period of time with little attention. This is because of the large natural capacity of the soil utilized to assimilate or attenuate the pollutants in the wastewaters. In areas where conditions are unsuitable for disposal by conventional subsurface soil absorption fields, a modified approach in subsoil waste application, a higher level of treatment, or other methods may be necessary. Therefore, the on-site system selection strategy presented here is based on the assumption that subsurface soil absorption is the preferred on-site disposal option because of its greater reliability with a minimum of attention. Where the site characteristics are not well suited for conventional subsurface soil absorption systems, other subsurface soil absorption systems are utilized. All three elements of this strategy are presented below.

25-4 WASTEWATER FLOW RATES AND CHARACTERISTICS

25-4-1 Average Flow and Variation

Daily average wastewater flow rates and daily and long-term variations in flow are used to determine the size of various system components. In addition, the characteristics of the wastewater is important in selecting the most effective treatment and disposal option. Average wastewater flow rates typically measured at treatment facilities may vary from about 150–450 Lpcd (liters per capita per day). The higher flow rates may also account for infiltration and inflow. Typical per capita flow rates to be expected from various residential units are presented in Table 25-1.[1-3]

The flow variations in individual homes, small establishments, and small communities may be quite large. As discussed in Chapter 3, the peaking factor generally depends upon the number of people served by the system. Peak-hour flow may be four to eight times the average flow rates. Lower variation is for small communities, while large variations are for individual homes and small establishments. The peaking factors for a maximum day, week, and month may also vary from 1.5 to 4 for small communities to individual homes.[1-3]

TABLE 25-1 Typical Wastewater Flow Rates from Single-Family Residences

	Flow Rate, Lpcd	
Type of Residence	**Range**	**Typical**
Detached homes		
Low income	150–210	170
Medium income	150–300	210
High income	190–380	250
Apartments and condominiums	130–210	150

Source: Adapted in part from Refs. 1–3.

25-4-2 Wastewater Characteristics

The characteristics of the wastewater generated by nonresidential establishments can vary greatly with the type of establishment. Consideration of the raw material used and waste-generating processes at a particular establishment can give a general idea of the character of the wastewater and may serve to indicate if the wastewater will contain any problem constituents.

Generally, the characteristics of wastes generated by a typical residential dwelling can be estimated from the flow rate and the mass loading (grams per capita per day). The mass loadings in terms of BOD, TSS, TN, TP, grease, and coliform produced per day from the human body wastes are known. There may be some influence on waste characteristics because of garbage grinders that may increase BOD_5 and TSS by about 20 percent if used extensively in homes. The typical mass loading and characteristics of residential wastewater quality are provided in Table 25-2.[1-3] For establishments where the waste-generating sources are significantly different than a residential dwelling, the characterization of waste stream can only be made through sampling and field investigations. Particular attention should be paid to problem constituents such as high grease levels from a restaurant or lint fibers from a laundromat. Readers should refer to Secs. 3-2-1 and 6-6-3 to obtain unit waste loadings for commercial and industrial establishments and for flow reduction and water conservation devices in homes and other establishments.

TABLE 25-2 Characteristics of Typical Mass Loading for Residential Wastewater[a]

Parameter	Mass Loading (g/cap/day)	Concentration (mg/L)
Total solids	115–117	680–1000
Volatile solids	65–85	380–500
Suspended solids	35–50	200–290
Volatile suspended solids	25–40	150–240
BOD_5	35–50	200–290
Chemical oxygen demand	115–125	680–730
Total nitrogen	6–17	35–100
Ammonia nitrogen	1–3	6–18
Total phosphorus	3–5	18–29
Phosphate, as P	1–4	6–24
Total coliforms[b]	10^{11}–4.0×10^{12}	10^7–10^9
Fecal coliforms[b]	—	10^8–10^{10}

[a]Typical residential dwellings equipped with standard water-using fixtures and appliances (excluding garbage disposals) generate a flow of approximately 170 Lpcd (45 gpcd).
[b]Number of organisms per capita per day and number of organisms per 100 mL.
[c]Number of organisms per 100 mL.

25-5 SITE EVALUATION AND SELECTION FOR ON-SITE TREATMENT AND EFFLUENT DISPOSAL

The soil environment into which the wastewater is discharged can be a valuable part of an on-site wastewater disposal system. If utilized properly, it can provide excellent treatment at little cost. However, if stressed beyond its assimilative capacity, the system will fail. Therefore, careful site evaluation is a vital part of an on-site system design.

25-5-1 Preliminary Information for Site Investigation

The objective of site investigation is to evaluate the characteristics of an area so that its potential for treatment and disposal of effluent can be established. A good site evaluation is one that provides sufficient information to select the most appropriate treatment and disposal system from a broad range of feasible options. A systematic approach is essential. The first step is to gather any available resource information about the site. Next, a plot plan of the site and the land immediately surrounding the area should be drawn. A scale large enough to display details such as buildings, manmade features, topography, soil type, land use, and others should be used. Table 25-3 is developed to summarize needed information and their sources for site investigation.

TABLE 25-3 Preliminary Information Gathering for Site Investigations.

Item	Information and Sources
Client contact	Obtain location and description of site, owner, current land use, and type of proposed facility.
Wastewater characteristics	Develop flow, and identify unusual constituents such as grease, fat or oil, high organic strength, and hazardous or toxic substances.
Preliminary evaluation	Determine soil types, geology, and topography. Obtain aerial photographs showing soil type, land use survey quadrangles, soil texture, climatological data, and the potential for flooding hazards. Obtain local records of soil tests, system designs, and reported problems with on-site systems installed in the area. Evaluate suitability for subsurface soil absorption.
Information sources	USDA Soil Survey Division (SSD) office,[a] appropriate state agencies, U.S. Geological Survey, U.S. Army Corps of Engineers, local government records, and university research reports and theses

[a]Formerly Soil Conservation Service (SCS).

25-5-2 Field Testing

Field testing of soils in the vicinity of the disposal field will provide valuable information about site characteristics and treatment requirements. Principal factors that should be considered in the field survey are (1) visual survey, (2) slope, (3) identification of soil characteristics, (4) percolation test, and (5) hydrological characteristics. Each factor is presented below.

Visual Survey. A visual survey is made to locate the areas on the lot with the greatest potential for subsurface soil absorption. The location of any depressions, gullies, steep slopes, rocks or rock outcrops, or other obvious land and surface features are noted and marked on the plot plan. Vegetation types are also noted that may indicate some soil characteristics. Locations and distances from a permanent benchmark to lot lines, wells, surface waters, buildings, and other features or structures are also marked on the plot plan. If a suitable area cannot be found for a subsurface absorption system based on this information, other sites may be considered.

Slope. The type and degree of slope of the area indicate the drainage pattern. For large areas, it may be necessary to draw contour lines.

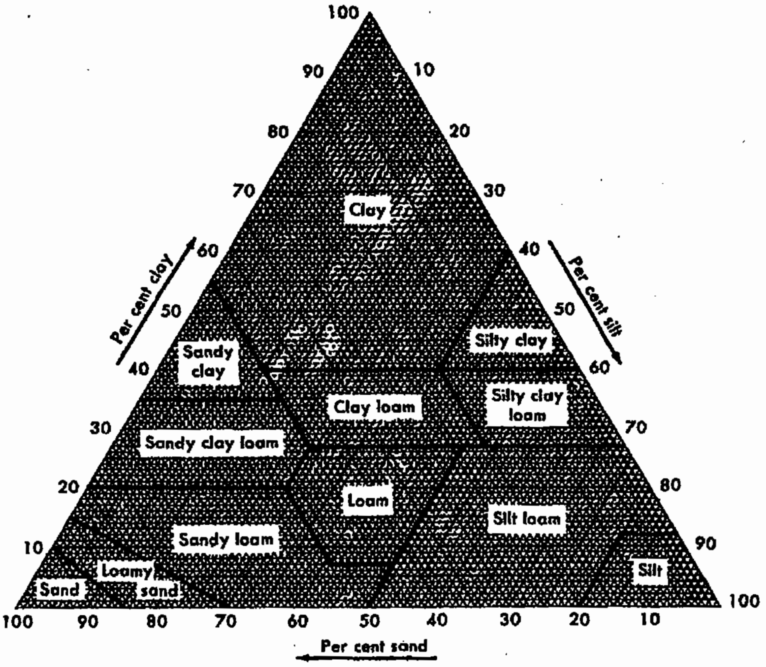
Identification of Soil Characteristics. Identification and evaluation of soil characteristics can best be determined from sufficient borings or pits. The properties of soil needed to assess its hydraulic properties and its ability to treat wastewater are (a) soil texture, (b) soil structure, (c) soil color, (d) seasonally saturated soils, (e) location of impervious layers, (f) pressure of swelling clays, and (g) bulk density.

Soil texture is one of the most important physical properties of soil, which is closely related to pore size, pore distribution, and pore continuity. It refers to the relative proportions of various sizes of solid particles that are smaller than 2 mm in diameter. It is determined by rubbing a moist sample between the thumb and forefinger. Various soil textures for general soil classifications are given in Table 25-4.[1] Characterization of soil classification by USDA is provided in Figure 25-1.[1,4] The soil structure has a significant influence on the soil's acceptance and transmission of water. Well-structured soils will transmit water more rapidly than structureless soils of the same texture.[5-9] The color and color pattern of soil are good indicators of the drainage characteristics of the soil. Bright uniform colors indicate well-drained and well-aerated soils. Seasonally structured soils are detected by the presence of colors (mottling in soil). Location of impervious layers and bedrock indicates the probable influence zone under the percolation field. Impervious layers and bedrocks seriously restrict the movement of water. Soil bulk density is related to porosity and movement of water. High bulk density is an indication of low porosity and restricted flow. Swelling clays can seal off soil pores when wet.[1,2,5] Typical site criteria for use of disposal fields and beds are provided in Table 25-5.[1,2] The physical characteristics of various soils of different textural properties are summarized in Table 25-6.[4]

TABLE 25-4 Textural Properties of Mineral Soils

	Feeling and Appearance	
Soil Class	**Dry Soil**	**Moist Soil**
Sand	Loose, single grains that 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. It 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.	It forms a cast that bears careful handling without breaking. It 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 a 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, flour-like 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 because of 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 and sticky and puddles easily.
Clay	Aggregates are hard; clods are extremely hard and strongly resist crushing by hand. When pulverized, it has a grit-like texture because of the harshness of numerous very small aggregates that persist.	Casts can bear considerable handling without breaking. It forms a flexible ribbon between thumb and forefinger and retains its plasticity when elongated. Rubbed surface has a very smooth, satiny feeling. It is sticky when wet and easily puddled.

Source: Adapted from Ref. 1.



Sand – 2.0 to 0.05 mm. diameter
Silt – 0.05 to 0.002 mm. diameter
Clay – smaller than 0.002 mm. diameter

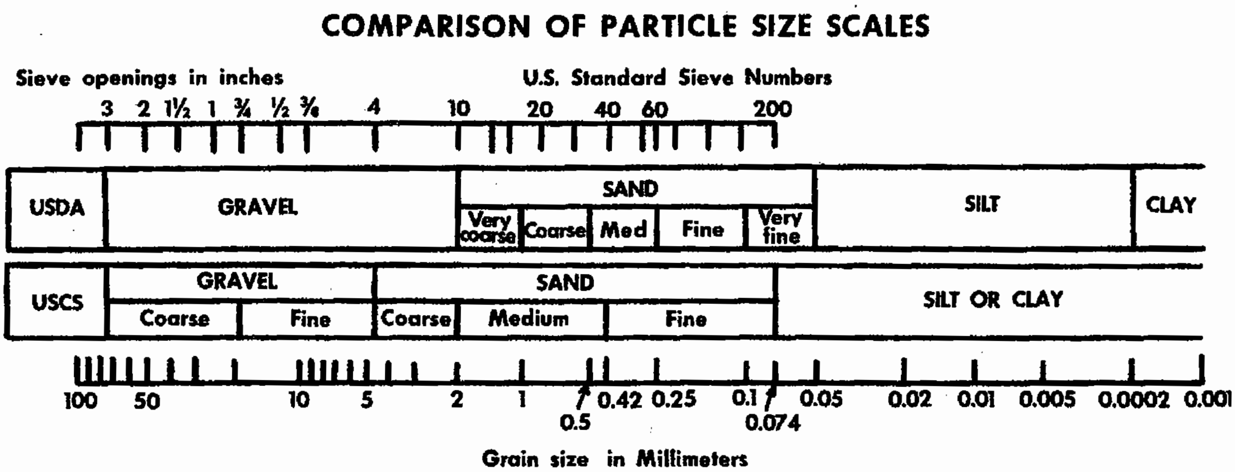


Figure 25-1 Textural Classification Chart (U.S. Department of Agriculture) and Comparison of Particle Size Scales (from Ref. 4).

Percolation Test. The hydraulic conductivity of a soil is important to design the percolation field. Several methods are used to measure the hydraulic conductivity of soils.[10-12] The most commonly used test is the percolation test. When properly run, this test is a good indication of hydraulic conductivity under saturated conditions. If movement of water in the percolation field occurs under unsaturated conditions, then empirical factors must be used to estimate the unsaturated hydraulic conductivity from the results of a saturated condition. The percolation test has some variability in data. Reasons

TABLE 25-5 *Typical Site Criteria for Use of Disposal Fields and Beds*

Item	Criteria
Landscape position[a]	Level, well-drained areas, crests of slopes, convex slopes more desirable. Avoid depressions, bases of slopes, and concave slopes unless suitable surface drainage is provided.
Slope[a]	0 to 25%. Slopes in excess of 25% can be utilized but the use of construction machinery may be limited. Seepage bed systems are limited to 0 to 5%.
Typical horizontal separation distances[b]	
Water supply wells	16–33 m (50–100 ft)
Surface waters, springs	16–33 m (50–100 ft)
Escarpments, manmade cuts	3–6 m (10–20 ft)
Boundary of property	1.5–3 m (5–10 ft)
Building foundations	3–6 m (10–20 ft)
Soil	
Texture	Soils with sandy or loamy textures are best suited. Gravelly and cobbley soils with open pores and slowly permeable clay soils are less desirable.
Structure	Strong granular, blocky, or prismatic structures are desirable. Platey or unstructured massive soils should be avoided.
Color	Bright uniform colors indicate well-drained, well-aerated soils. Dull, gray, or mottled soils indicate continuous or seasonal saturation and are unsuitable.
Layering	Soils exhibiting layers with distinct textural or structural changes should be evaluated carefully to ensure water movement will not be severely restricted.
Unsaturated depth	0.6–1.2 m (2–4 ft) of unsaturated soil should exist between the bottom of the disposal field and the seasonally high water table or bedrock.

[a]Landscape position and slope are more restrictive for seepage beds because of the depths of cut on the upslope side.
[b]Intended only as a guide. Safe distance varies from site to site, based on local codes, topography, soil permeability, groundwater gradients, geology, etc.
Note: ft × 0.3048 = m.
Source: Adapted from Refs. 1 and 2.

TABLE 25-6 *Physical Characteristics of Soils of Various Textural Properties*

Major Divisions		Name	Color	Drainage Characteristics	Permeability (cm per s)
Coarse-Grained Soils	Gravel and gravelly soils	Well-graded gravels or gravel-sand mixtures, little or no fines	Red	Excellent	$k > 10^{-2}$
		Poorly graded gravels or gravel-sand mixtures, little or no fines		Excellent	$k > 10^{-2}$
		Silty gravels, gravel-sand-silt mixtures	Yellow	Fair to poor Poor to practically impervious	$k = 10^{-3}$ to 10^{-6}
		Clayey gravels, gravel-sand-clay mixtures		Poor to practically impervious	$k = 10^{-6}$ to 10^{-8}
	Sand and sandy soils	Well-graded sands or gravelly sands little or no fines	Red	Excellent	$k > 10^{-3}$
		Poorly graded sands or gravelly sands, little or no fines		Excellent	$k > 10^{-3}$
		Silty sands, sand-silt mixtures	Yellow	Fair to poor Poor to practically impervious	$k = 10^{-3}$ to 10^{-6}
		Clayey sands, sand-clay mixtures		Poor to practically impervious	$k = 10^{-6}$ to 10^{-8}

Major Divisions		Name	Color	Drainage Characteristics	Permeability (cm per s)
Fine-Grained Soils	Silts and clays LL[a] is less than 50	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	Green	Fair to poor	$k = 10^{-3}$ to 10^{-6}
		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		Practically impervious	$k = 10^{-6}$ to 10^{-8}
		Organic silts and organic silt-clays of low plasticity		Poor	$k = 10^{-4}$ to 10^{-6}
	Silts and clays LL[a] is greater than 50	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Blue	Fair to poor	$k = 10^{-4}$ to 10^{-6}
		Inorganic clays of high plasticity, fat clays		Practically impervious	$k = 10^{-6}$ to 10^{-8}
		Organic clays of medium to high plasticity, organic silts		Practically impervious	$k = 10^{-6}$ to 10^{-8}
Highly Organic Soils		Peat and other highly organic soils	Orange	Not recommended for disposal field	

[a]Liquid limit.
Source: Adapted from Unified Soil Classification System (Ref. 4).

for large variability may be caused by (1) soil moisture, (2) fractures, and (3) procedure and individuals. The standard procedure for a percolation test is given below:[1,2]

1. At least three tests within the proposed area must be performed.
2. Drill a test hole, 15 cm (6 in.) in diameter, to the depth of the absorption system. Scratch the sides of the hole and loose material is removed.
3. A layer of 5 cm (2 in.) gravel having sizes 1.2–1.8 cm (1/2–3/4 in.) should be placed at the base.
4. Fill the hole with clean water to a depth of 0.3 m (12 in) and maintain this depth for at least 4 h, preferably overnight. Automatic siphon or float valve may be employed to an automatic level control. Thorough soaking is essential for good results.
5. Percolation rate measurement should be started 15 h, but no more than 30 h, after the soaking period began. Any soil that sloughed off into the hole should be removed, and the water level is adjusted to 15 cm (6 in.) above the gravel or 20 cm (8 in.) above the bottom.
6. Immediately after adjusting the liquid level, the measurement should begin at every 30-min interval. The recording accuracy should be within 1.6 mm (1/16 in.). After each measurement, the level is adjusted to the original 15 cm (6 in.) above the gravel layer. The test is continued until two successive water level drops are obtained. The last water level drop is used to calculate the percolation rate. At least three independent measurements are made.
7. In sandy soils in which the first 15-cm (6-in.) water drop is less than 30 min, the water level measurements should be made at 10-min intervals for a 1-h period. The last water level drop is used to calculate the percolation rate.
8. The percolation rate is expressed in min/cm (min/in.).
9. Similarly, the test is conducted at several locations in the area. If the difference in various readings is greater than 7.8 min/cm (20 min/in.), the results are not averaged.

Several other tests are also available that give a saturated coefficient of permeability. Information on these test methods and comparative results may be found in Refs. 1, 10, and 13–17. The estimated percolation rates for various soil textures are summarized in Table 25-7.

Hydrological Characteristics. Hydrological information for site evaluation is essential. This includes depth to groundwater, the hydraulic gradient and direction of flow, coefficient of permeability, and site assimilative capacity. The assimilative capacity of a site is defined as its ability to accept water that may percolate into the ground without developing a groundwater mound and resurfacing during wet weather conditions. The major pathways that govern the assimilative capacity of a soil are percolation of water downward and laterally, taken up by plants, or evapotranspiration.[18] In most situations, the flow that can be transported away from a site can be calculated from Darcy's law [Eq. (25-1)]:[2,19,20]

$$v = -ks \tag{25-1}$$

TABLE 25-7 *The Estimated Percolation Rates for Various Soil Textures*

	Percolation Test		Permeability		Percolation Rate	
Soil Texture	**min/cm**	**min/in.**	**cm/h**	**in./h**	**$L/m^2 \cdot h$**	**$gal/ft^2 \cdot h$**
Sand	<4	<10	>15	>6.0	>150	>3.7
Sand loams, porous silt loams	4–25	10–64	2.5–15	1.0–6.0	25–150	0.6–3.7
Compact silt loams, silty clay loam	25–60	64–150	1–2.5	0.5–1.0	10–25	0.3–0.6

$L/m^2 \cdot h \times 0.0245 = gal/ft^2 \cdot h$.
Source: Adapted from Ref. 17.

where

v = velocity of flow, m/d (ft/d)
k = coefficient of permeability or hydraulic conductivity, m/d (ft/d) or cm/s
s = hydraulic gradient or the slope of water surface, m/m (ft/ft)

The negative sign in the equation is because of negative head loss in the direction of the flow. The flow rate is obtained by multiplying the velocity by the cross-sectional area. This is given by Eq. (25-2):

$$Q = -Aks \qquad (25\text{-}2)$$

where

Q = flow rate, m^3/d (ft^3/d)
A = cross-sectional area perpendicular to the direction of flow, m^2 (ft^2)

25-5-3 Disposal Field Performance

The effluent from a septic tank or from any other treatment system is applied over a disposal field. Many methods of effluent application and disposal are presented in a Section 25-7. Irrespective of treatment and application methods, the liquid travels through the vadose zone, and further treatment continues in the field through a combination of physical, chemical, and biological mechanisms. The porous medium in the disposal field may act as a submerged anaerobic filter under continuous inundation or as an anaerobic-aerobic filter under periodic application. The performance of the disposal field therefore depends upon the hydraulic application rate and the quality of effluent. The treatment of effluent at the soil interface in a disposal field may be divided into two broad conditions: (1) gravity flow from a septic tank and (2) intermittent application.

TABLE 25-8 Hydraulic Loading Rates for Disposal Fields and Seepage Beds on Bottom and Sidewall Area

Soil Texture	Percolation Test (min/cm)	Hydraulic Loading Rate ($L/m^2 \cdot d$)
Sandy soil	<4	>48
Sand loams, porous silt	4–25	12–48
Compact silt loams, silty clay loam	25–60	5–12

Gravity Flow from a Septic Tank. An anaerobic condition generally develops in a disposal field under intermittent gravity-flow application of effluent from a septic tank. A biomat will form on the interface, which will reduce percolation and enhance mineralization. Many minerals such as iron, aluminum, and calcium will precipitate as sulfide and phosphate. These minerals will eventually leach out, and a dynamic balance will be established. The long-term hydraulic capacity of the biomat will finally control the movement of the fluid. This is also called the long-term acceptance rate (LTAR) and has a value in the range of 40–120 $L/m^2 \cdot d$ (0.3–0.9 $gal/ft^2 \cdot d$).[2,21]

Intermittent Application. Periodic dosing of the disposal field by a pump or dosing syphon creates an environmental condition that is quite beneficial. During application, the hydraulic loading is uniform over the entire field. The aerobic environment will prevail after percolation, even after a brief anaerobic condition in many sections of the field. The effluent treatment progresses rapidly, and the biomat layer is not as heavy as that under gravity-flow application. The net result is a higher LTAR. The disposal field is smaller, and the overall performance is remarkably improved. Excellent nitrification and denitrification has been reported with favorable total nitrogen removal (see Chapter 13 for nitrification/denitrification).

Design Hydraulic Loading Rates. The percolation test data indicate soil permeability. There is no direct relationship between the observed short-term percolation test with clean water and the LTAR based on septic tank effluent applied over the percolation field. Many states utilize tables and curves to convert the results of the percolation test into allowable hydraulic loadings. The biomass that builds up on the bottom of the trench reduces the percolation rate, while the sidewalls remain effective. As the liquid level in the trench increases, the driving head over the bottom also increases. The net effect is a variable hydraulic loading along the length and cross section of the trench. In many cases, only the bottom area is used, while in other cases, only the vertical area of the sidewalls is used. In the majority of cases, however, both bottom and sidewall areas are used. The hydraulic loading rates for disposal-field trenches and seepage beds based on bottom and sidewall areas in relation to the percolation test are summarized in Table 25-8.

TABLE 25-9 Basic Requirements of Disposal Sites Falling into Three Major Groups

Requirements	Normal Site	Difficult Site	Adverse Site
USDA texture	Coarse to medium sand, fine sand, sand loams, porous silt loams	Silty clay, loam, porous silt loam, silty clay loam	Clays, colloidal clays, expansive clays
Flooding	None, protected	Rare	Common
Slope, percent	0–8	8–15	>15
Depth to bedrock, m	>2	1–2	<1
Depth of high water table below bottom of disposal field, m	>2	1–2	<1
Permeability			
cm/h	5–150	0.5–5	<0.5
in./h	2–60	0.2–2	<0.2
Other features such as water supply wells, surface water springs, escarpments, manmade cuts, building foundations, boundary property, and lot size	Desirable	Moderate	Undesirable

25-5-4 Evaluation and Classification of Disposal Site

The ability of a disposal site to function properly for an extended period of time is dependent upon many natural and designed conditions. Common factors are (1) effluent quality and quantity applied and mode of application; (2) ground surface slope; (3) property size, distance from water supply well, building foundation, and others; (4) depth of groundwater table and assimilative capacity; (5) depth of stratum; (6) fractured bedrock; and (7) climate. Although all these factors and many more are critically reviewed for site evaluation and design, little is known quantitatively for the collective impact of these factors. Commonly, most disposal sites will fall broadly into three major groups:

1. Normal site conditions for conventional septic tank and effluent application over the disposal field
2. Difficult site conditions for improved treatment and effluent application systems over the disposal field
3. Adverse site conditions requiring special systems

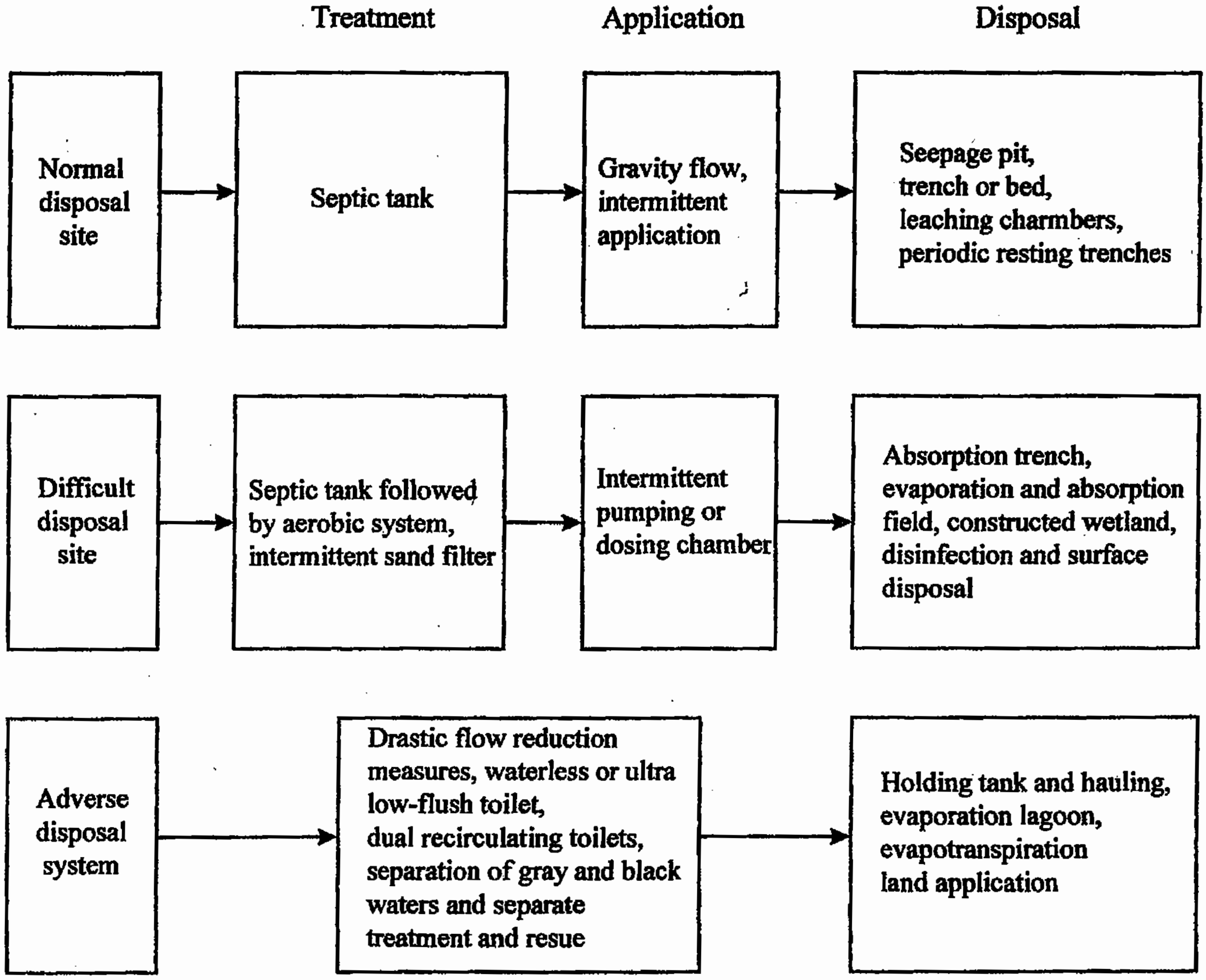


Figure 25-2 Overall Site Rating System Compatibility with Respect to Treatment Methods, Mode of Effluent Application, and Effluent Disposal.

The basic requirements of disposal sites that broadly fall into these three major groups are summarized in Table 25-9. The treatment requirement and mode of effluent application and disposal systems are shown in Figure 25-2. Design information, treatment methods, effluent disposal methods, overall system design, and configuration are covered later in this chapter.

25-6 ON-SITE WASTEWATER TREATMENT SYSTEMS

Principal components of on-site wastewater treatment systems for individual residences, cluster homes, and other establishments are presented in this section. Before presenting the treatment systems, it is essential to describe the grease trap that is commonly used with many establishments.

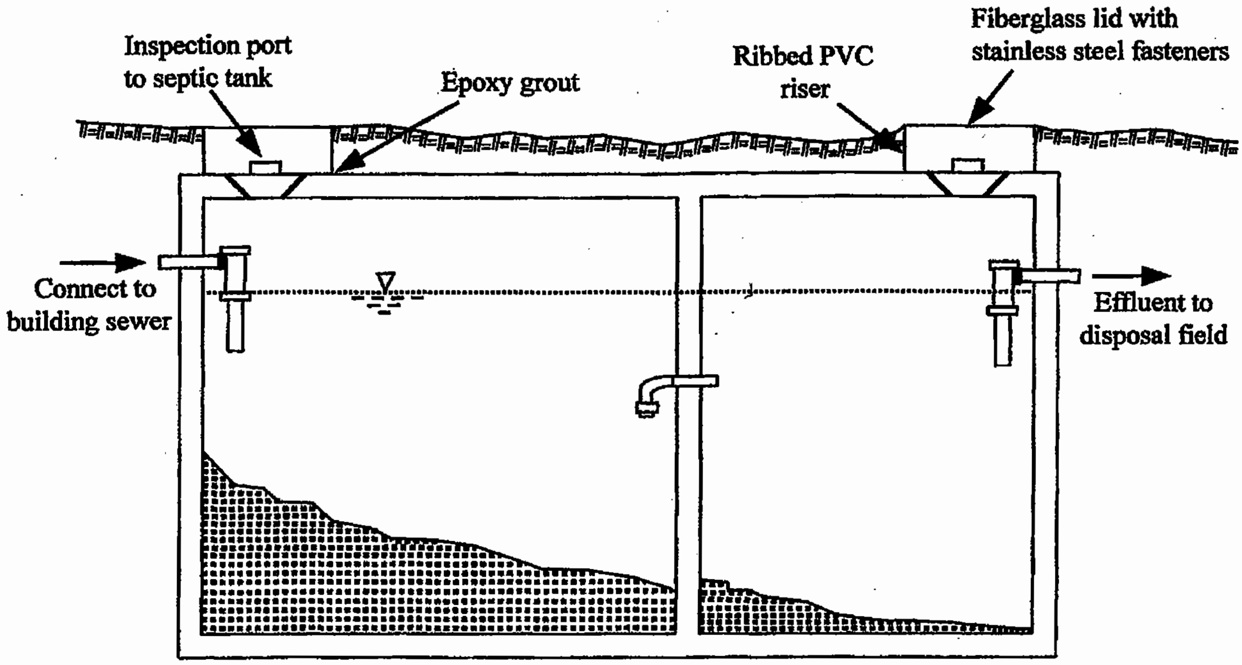


Figure 25-3 Typical Septic Tank Design.

25-6-1 Grease Trap

A grease trap is an appurtenance that is used to remove excessive amounts of grease that may interfere with subsequent treatment and disposal. In many commercial and institutional establishments where large volumes of kitchen wastes are generated (cafeterias, restaurants, schools, hospitals), the grease can clog the sewer lines and inlet and outlet structures of treatment devices. The purpose of a grease trap is simply to remove the grease.

Grease traps are interceptor or flotation tanks similar to a septic tank located at the heavy oil- and grease-containing streams. Typically, they help to cool and solidify the grease. A detention time of 30 min is recommended for effective flotation of grease. A number of commercial grease and oil traps are also commercially available. Details on grease traps for different establishments may be found in Refs. 22 and 23.

25-6-2 Septic Tanks

The septic tank is the most widely used on-site wastewater treatment option in the United States. Currently, about 25 percent of the new homes being constructed use septic tanks for treatment prior to on-site disposal. A section of a typical septic tank is shown in Figure 25-3.

Septic tanks are buried, watertight receptacles designed and constructed to receive wastewater, to separate solids from the liquid, to provide limited digestion of organic matter, to store solids, and to allow the clarified liquid to discharge for further treatment and disposal. The settleable solids and partially decomposed sludge settle to the bottom of the tank and accumulate. The complex organic matter is liquified under anaerobic conditions. A portion is converted into carbon dioxide, methane, and hydrogen sulfide. The minimum acceptable detention time at average flow is 0.5 d. A scum of lightweight material (including fats and greases) rises to the top. The partially clarified liquid is allowed to flow through an outlet structure below the floating scum layer. Proper use of baffles and tees protect against scum outflow.

Basic Design Considerations. The septic tank must be watertight. Excessive leakage from septic tanks can cause the level of the scum layer to drop below the outlet baffle. This would allow excessive scum to discharge into the absorption field. Scum will dry and compact and greatly diminish percolation into the absorption field. In these cases, normal tank-cleaning practices will not remove the dried scum. Another problem can occur if the tank is not watertight. Infiltration may enter the tank, which can cause hydraulic overloading of the tank and the percolation system.

To ensure that the flow out of the septic tank carries a minimal concentration of settleable solids, proper inlet and outlet devices must be installed. These devices serve three general functions: (1) to minimize turbulence caused by the inlet flow to prevent mixing and poor settling of sludge, (2) to prevent resuspension of solids at the outlet of the tank, and (3) to prevent rising gases from the anaerobic digestion process from interfering with settling and resuspension of particles. Many types of influent and effluent structures have been utilized. These include outlet tee, effluent vault, scum baffle, and various types of gas deflection baffles. The details may be found in several references.[1,2,18,21,22]

The septic tanks also utilize one, two, or three compartments, although two-compartment systems are most common. Larger septic tanks are also used for small communities and subdivisions. The basic design criteria for septic tanks are provided in Table 25-9.

A number of septic tank manufacturers market and claim improved designs over conventional systems. These are made of polyethylene and fiberglass and utilize a single-compartment tank with an effluent filter vault. Various types of screens are provided to filter solids. These screens are removable for cleaning and can be installed in existing septic tanks. Other systems utilize a submersible effluent pump inside a vault. A high-level alarm float is also provided in many cases. Some of these features are shown in Figure 25-4.[2]

Performance. The settleable solids deposit on the bottom of the septic tank or in the first compartment. These solids undergo solubilization and partial stabilization under the anaerobic environment. As a result, NH_3-N may be higher in the effluent than in the influent. Coliform numbers in the effluent are not changed significantly. The effluent quality from a septic tank is provided in Table 25-10.

Septage Quality and Disposal. Solids accumulate in a septic tank, and the entire volume must be pumped out periodically. The content pumped out is called septage and contains scum, sludge, and a large volume of water. The typical concentrations of various key constituents are as follows (in mg/L): BOD_5 = 6000, TSS = 15,000, TN = 700, NH_3-N = 400, TP = 250, and oil and grease = 8000.[2,13]

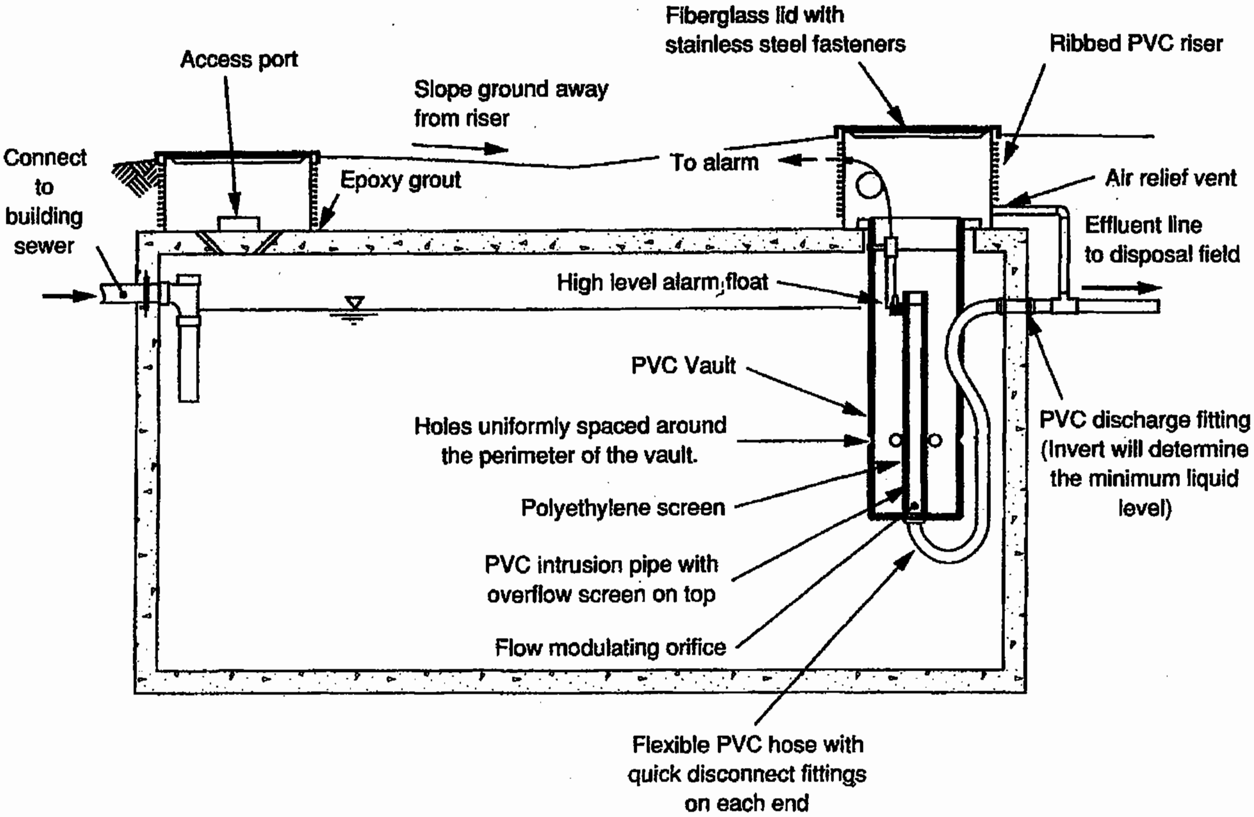


Figure 25-4 A Single-Compartment Septic Tank with Filter Vault and High-Level Alarm (from Ref. 2; used with permission of The McGraw-Hill Companies).

The most common and effective method of septage disposal is by discharge into a local wastewater treatment plant. The requirements for co-treatment with wastewater are (1) sufficient plant capacity and (2) septage receiving station.[2,13] The receiving station is generally ahead of the grit removal facility and contains a wet well and a grinder pump that releases the contents gradually. Other methods of septage disposal are co-disposal with solid wastes, land application, and processing at a treatment facility designed and built to handle septage. A facility designed to treat septage utilizes stabilization ponds, aerated lagoons, constructed wetlands, and a complete primary and secondary treatment plant with chemical and biological treatment capacity. Readers may refer to several references for in-depth coverage on this topic.[1,2,13]

25-6-3 Imhoff Tank

Imhoff tanks are two-story tanks in which sedimentation occurs in the upper compartment and solids accumulate in the lower compartment. The overhanging lip in the bottom of the sedimentation basin helps to reduce the entry of gases and gas-buoyed solids into the sedimentation chamber. The Imhoff tanks are suitable for smaller communities, and they function similar to a septic tank. They are rarely used for single- or multiple-family residences. Design details may be found in Refs. 2 and 24.

TABLE 25-10 Typical Design Criteria for Septic Tanks

Design Parameter	Unit	Range	Typical
Liquid volume			
Minimum	m^3	2.5–4.0	2.5[a]
1–2 bedrooms	m^3	2.5–4.0	2.5[a]
3 bedrooms	m^3	3.5–5.7	4.5[a]
4 bedroom	m^3	3.5–7.6	5.7[a]
5 bedrooms	m^3	4.5–7.6	5.7[a]
Additional bedrooms	m^3	0.6–1.0	1.0[a]
Compartments			
Number	No.	1–3	2[b]
Volume distribution in multicompartment tanks			
Two-compartment tank	% (total) 1st, 2nd	67,33	67,33
Three-compartment tank	% (total) 1st, 2nd, 3rd	33,33,33	33,33,33
Length to width	ratio	2:1–4:1	3:1
Depth	m	0.5–2.0	1.5
Clear space above liquid	cm	25–30	25
Depth of water surface below inlet	cm	7–10	8
Inspection ports (see Fig. 25-3)	No	2–3	2
Inlet and outlet devices	No	1 each	1 each
Effluent quality[c]			
BOD_5	mg/L	120–140	130
TSS	mg/L	50–100	90
NH_3-N	mg/L	20–50	40
NO_3-N	mg/L	<1	<1
TN	mg/L	25–80	50
TP	mg/L	10–20	15
Total coliform	MPN/100 mL	10^3–10^6	10^5
Virus	PFU/mL	10^5–10^7	10^6

[a]Most regulatory agencies have minimum size requirements for septic tanks (typically 4500 L). If septic tank size is not specified, the values given in this table can be used as a guide.
[b]Two- or three-compartment tanks are used when a screen vault is not used.
[c]Influent quality is given in Table 25-2.
Source: Adapted in part from Refs. 1, 2, and 18.

25-6-4 Suspended Growth Aerobic System

A wide variety of suspended growth aerobic systems are marketed that are essentially miniature batch or continuous flow activated sludge systems. The raw wastewater or effluent from a septic tank is mixed with active biomass. Diffused air maintains the aerobic condition. The final clarifier may be inside or outside a conical aeration basin or on

TABLE 25-11 Typical Operating Parameters for On-site Extended Aeration Systems

Parameter[a]	Average	Maximum
MLSS, mg/L	2,000–6,000	8000
F/M, g BOD/g MLVSS·d	0.05–0.1	—
Solids retention time, d	20–100	—
Hydraulic retention time, d	2–5	—
Dissolved oxygen, mg/L	>2.0	—
Mixing, kW/m^3	0.01–0.03	—
Clarifier overflow rate, $m^3/m^2{\cdot}d$	8–16	33
Clarifier solid loading, $kg/m^2{\cdot}d$	4–6	10
Clarifier weir loading $m^3/m{\cdot}d$	124–370	370
Sludge wasting interval, months	3–6	8–12

[a]Pretreatment: trash trap or septic tank.
$kW/m^3 \times 5.20 = hp/10^3{\cdot}gal$; $m^3/m^2{\cdot}d \times 24.55 = gpd/ft^2$; $kg/m^2{\cdot}d \times 4.878 = lb/ft^2{\cdot}d$;
$m^3/m{\cdot}d \times 80.52 = gpd/ft$.
Source: Adapted in part from Refs. 1 and 2.

the side of a rectangular basin. Some of these arrangements are shown in Figure 25-5. Most designs utilize an extended aeration mode. Typical design parameters for an extended aeration facility are provided in Table 25-11.

25-6-5 Fixed-Film Aerobic Systems

A number of small fixed-film systems for individual homes and small establishments have been developed. Among these are packed-media, plates or tubes, and biodiscs. These systems are designed in conjunction with an anaerobic chamber or septic tank. The upflow packed-media system uses diffused aeration. The clarifier may be separate or on the top of the media. Examples of several fixed-film treatment systems are shown in Figure 25-6.

25-6-6 Intermittent Sand Filter

Intermittent sand filter is a bed of granular material. Septic tank effluent is applied intermittently, and treated liquid is collected at the bottom of the filter. The effluent is of high quality, which can be discharged into a disposal field or disinfected and discharged to surface waters. The major components are filter housing, underdrain pipe, gravel and sand bed, and flow distribution piping buried in gravel. The filter may be buried under natural soil (buried filter) or have free access (open filter). Another variation of an intermittent sand filter is a recirculation type, generally called a recirculating filter. Each of these variations is described later in this section.

The treatment mechanisms in intermittent sand filters are complex combinations of physical, chemical, and biological transformations. They provide physical straining and

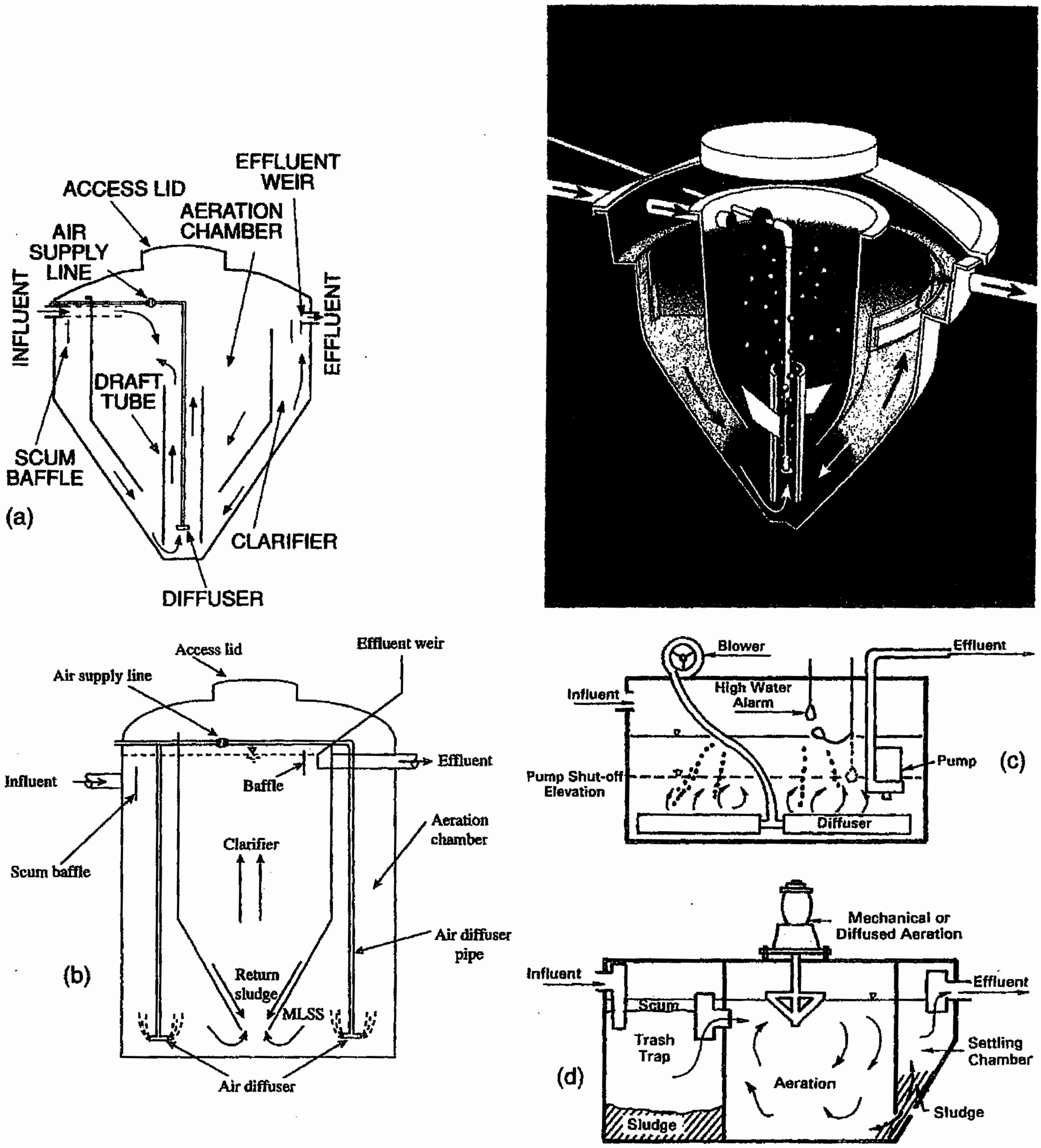


Figure 25-5 Suspended Growth Aerobic System for Individual Homes: (a) draft tube and aeration chamber in the center (courtesy Consolidated Treatment Systems); (b) clarifier in the center of aeration basin; (c) batch extended aeration system; and (d) flow through extended aeration system (from Ref. 1).

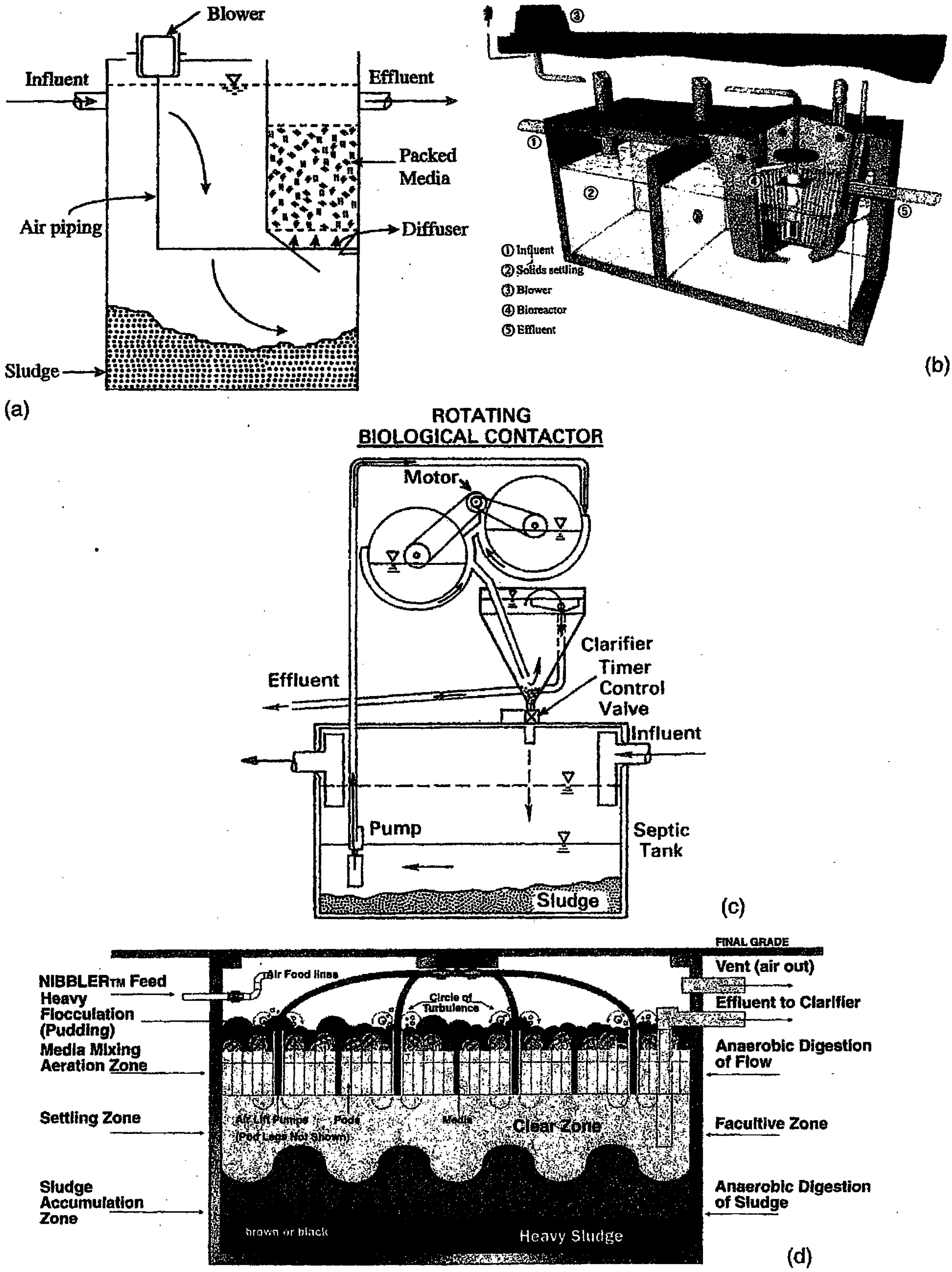


Figure 25-6 Fixed-Film Aerobic Systems: (a) packed-bed upflow system with diffused aeration; (b) fixed-film reactor with diffused aeration (courtesy Bio-Microbics, Inc.); (c) bio-discs (rotating biological contactor, RBC) with external clarifier (from Ref. 1); and (d) the suspended growth, The Nibbler™ Bio-Mat system (courtesy NCS, a Division of Northwest Cascade).

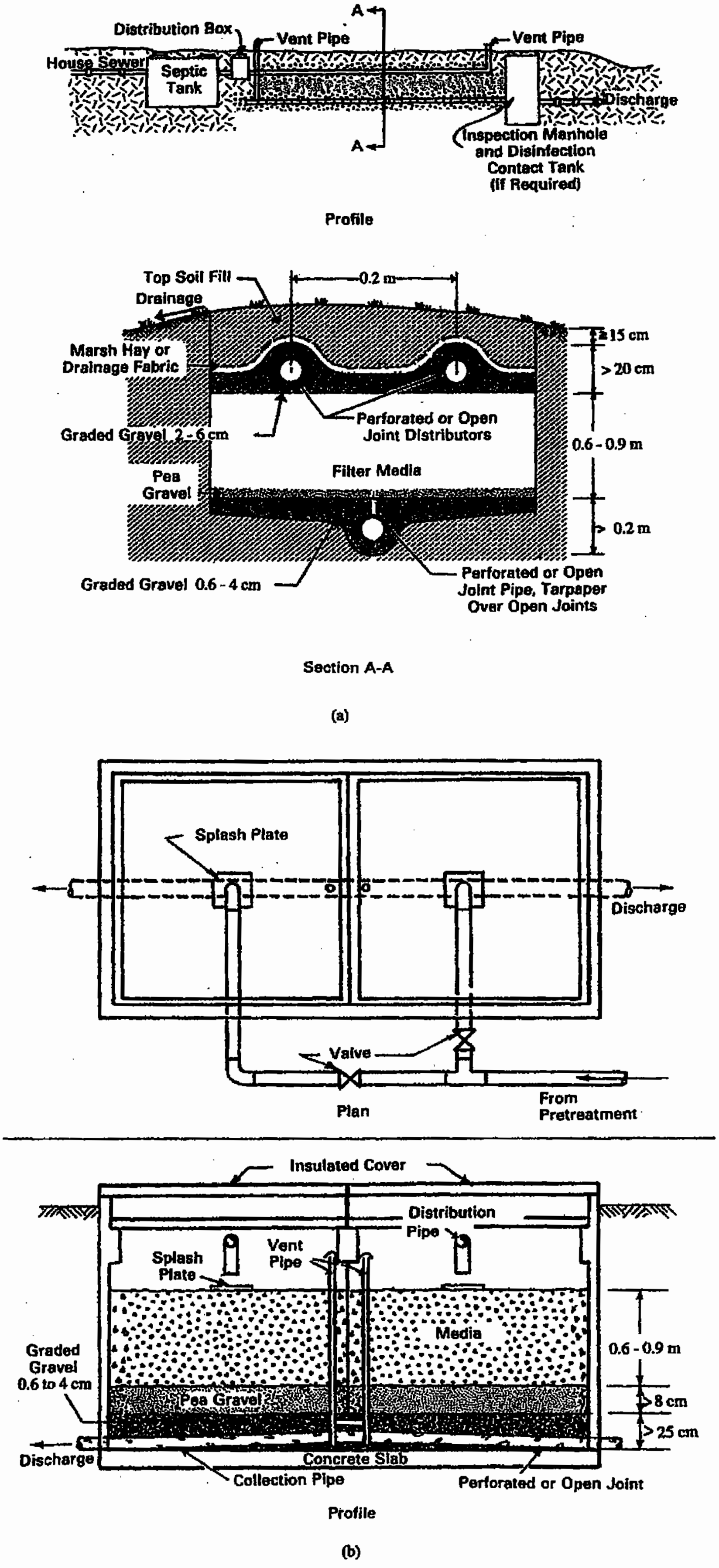


Figure 25-7 Intermittent Sand Filter Installations: (a) typical buried intermittent sand filter and (b) typical free access intermittent sand filter.

sedimentation within the media grains. Chemical coagulation and sorption and biochemical transformations are the predominant mechanisms. There are broad trophic levels ranging from bacteria to annelid worm that operate within a filter. Media clogging does occur. Major factors contributing to clogging are (1) wastewater characteristics, (2) method and rate of wastewater application, (3) characteristics of the media, and (4) filter environment.[1] High organic loading, absence of dissolved oxygen, low hydraulic loading, and low temperatures are the main causes of clogging.

Proper filter dosing is critical for proper performance. The key elements are (1) proper flow distribution and (2) dosing frequency that may range from 30 min to once a day.[1] Proper flow distribution over the bed can be achieved by ridge and furrow application, drain tile distribution, surface flooding, and spray distribution. The common methods to maintain an intermittent sand filter are (1) resting the bed, (2) raking the surface to break the inhabited crust, and (3) replacing the top surface layer.

As mentioned previously, the intermittent sand filters are divided into three types: (1) buried, (2) open, and (3) recirculating. The basic design features, operation, and performance vary slightly. These are discussed below.

Buried Intermittent Sand Filter. The buried filters are constructed below the grade. A geotextile or suitable drainage fabric is placed over the top of the filter for separation of the filter surface with the native soil. Air vents are provided for ventilation. A typical filter installation is shown in Figure 25-7(a). The design details and filter performance data are summarized in Table 25-12.[1,2]

Open Intermittent Sand Filter. The open filters are essentially the same as buried filters, with the exception that they do not have soil cover. They may be open or covered to improve maintenance and keep them warm during winter months. The installation details are shown in Figure 25-7(b), and design and performance data are provided in Table 25-12.[25-29]

Recirculating Filter. The recirculating filters are generally open and have a similar design to the other two. The basic differences are (1) the effective size of the coarse media is slightly large, (2) the loading rate based on the influent flow is larger, and (3) a recirculation tank receives the effluent from a septic tank or other treatment device, as well

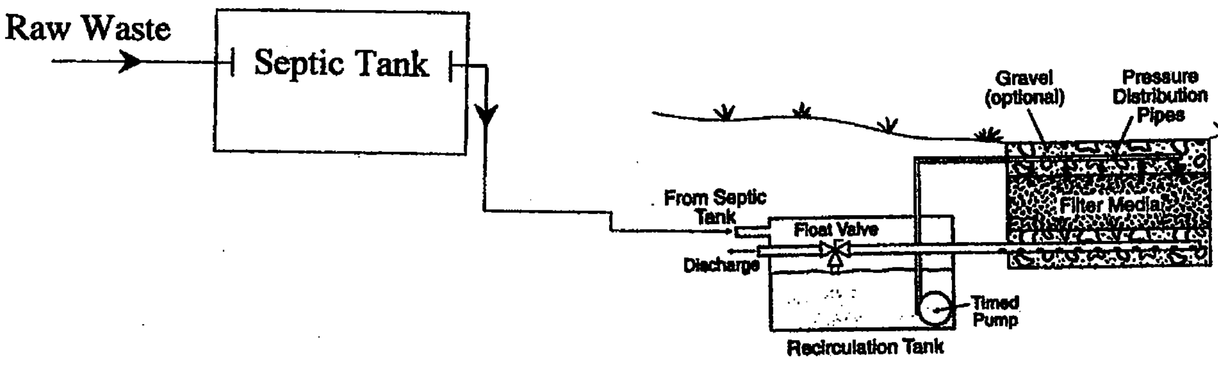


Figure 25-8 Typical Recirculating Intermittent Filter System.

TABLE 25-12 *Design Features and Performance of Intermittent Sand Filters (Buried, Open, and Recirculating Types)*

Item	Burried Filter	Open Filter	Recirculating Filter
Design Criteria			
Pretreatment	Minimum level-sedimentation (septic tank or equivalent)	Minimum level-sedimentation (septic tank or equivalent)	Minimum level-sedimentation (septic tank or equivalent)
Hydraulic loading			
All year	<17 L/m^2·d (<1.0 gpd/ft^2)	34–85 L/m^2·d (2.0–5.0 gpd/ft^2)	50–85 L/m^2·d (3.0–5.0 gpd/ft^2) forward flow
Seasonal	<34 L/m^2·d (<2.0 gpd/ft^2)	85–170 L/m^2·d (5.0–10.0 gpd/ft^2)	
Media			
Material	Washed durable granular material (less than 1 percent organic matter by weight	Washed durable granular material (less than 1 percent organic matter by weight)	Washed durable granular material (less than 1 percent organic matter by weight)
Effective size	0.50–1.00 mm)	0.35–1.00 mm	0.3–1.5 mm
Uniformity coeff.	<4.0 (<3.5 preferable)	<4.0 (<3.5 preferable)	<4.0 (<3.5 preferable)
Depth	0.6–0.9 m (24–36 in.)	0.6–0.9 m (24–36 in.)	0.6–0.9 m (24–36 in.)
Underdrains			
Material	Open joint or perforated pipe	Open joint or perforated pipe	Open joint or perforated pipe
Slope	0.5–1.0 percent	0.5–1.0 percent	0.5–1.0 percent
Bedding	Washed durable gravel or crushed stone 0.6–3.8 cm (1/4–1-1/2 in.)	Washed durable gravel or crushed stone 0.6–3.8 cm (1/4–1-1/2 in.)	Washed durable gravel or crushed stone 0.6–3.8 cm (1/4–1-1/2 in.)
Venting	Upstream end	Upstream end	Upstream end

Distribution			
Material	Open joint or perforated pipe	Troughs on surface; splash plates at center or corners; sprinkler distribution	Troughs on surface; splash plates at center or corner; sprinkler distribution
Bedding	Washed durable gravel or stone 2–6.5 cm (3/4–2-1/2 in.)		
Venting	Downstream end		
Dosing	Flood filter; frequency greater than 2 per day	Flood filter to 5 cm (2 in.) depth; frequency greater than 2 per day	Flood filter to approximately 5 cm (2 in.) depth; pump 5–10 min per 30 min; empty recirculation tank in less than 20 min
Recirculating ratio	None	None	3:1–5:1 (5:1 preferable)
Recirculation Tank	None	None	Volume equivalent to at least one day's raw wastewater flow
Effluent Quality			
BOD_5, mg/L	4	10	16
TSS, mg/L	4	10	10
NH_4^+-N, mg/L	3	3	8
NO_3^--N, mg/L	27	32	—

Note: (1) Nitrogen is lost due to denitrification. (2) Open and recirculating filter surface is raked and weeds are removed weekly.
Source: Adapted in part from Refs. 1, 2, and 25–29.

as a portion of sand filter effluent. The recirculating tank is normally one-fourth to one-half the size of the septic tank. The recirculation ratio is controlled by a splitter box, movable gate, check valve, or float. The schematic flow diagram is shown in Figure 25-8. The design information and performance data are summarized in Table 25-12. Filter clogging is controlled by surface racking and a resting period.[1,2]

25-6-7 Nutrient Removal

Nitrogen and phosphorus may be removed from wastewater by biological methods. A discussion on this subject may be found in Chapter 13. A large portion of nitrogen removal is achieved by nitrification/denitrification in intermittent sand filters that receive low organic loading. Nitrogen removal also occurs in the absorption field if the wastewater loading is intermittent and environmental conditions develop for nitrification/denitrification. Other applicable methods for ammonia removal is by ion exchange through naturally occurring clinoptololite resin (see Chapter 24).

Although biological phosphorus removal is applied in large municipal wastewater treatment plants, its application for an on-site disposal system is not practicable. Chemical precipitation of phosphorus using alum or iron coagulants is possible in suspended or attached growth biological systems, which is presented in Sec. 24-3-2. Chemical dosage requirements may be found in Chapters 12, 13, and 24. These systems are complex and are beyond the capability of individual homes and small establishments to properly operate and maintain them.

25-7 FINAL DISPOSAL METHODS

The effluent from a septic tank, aerobic system, or intermittent sand filter can be safely disposed of onto the land or into surface waters or evaporated into the atmosphere by a variety of methods. The most accepted method is subsurface soil absorption if conditions permit. Under unfavorable conditions, other options are considered. In Sec. 25-5, site evaluation and selection information was presented. Based on performance criteria, the potential disposal sites were divided into three major groups: (1) normal site conditions, (2) difficult site conditions, and (3) adverse site conditions (see Tables 25-4 to 25-9 and Figures 25-1 and 25-2). These disposal site conditions are further reviewed here so that the discussion on disposal methods can be tied in more closely with the site conditions.

25-7-1 Normal Site Conditions for Common Treatment and Disposal Systems

Normal site conditions are characterized by suitable subsoils for percolation of septic tank effluent, a deep water table and unfractured bedrock, favorable ground surface slope, no flooding, and other features such as a water supply well and other water sources; buildings, escarpments, and so forth sufficiently far away; and large lots. The septic tank effluent can be disposed off on-site by gravity flow over a conventional percolation trench or bed.

25-7-2 Difficult Site Conditions for Improved Treatment and Disposal Systems

The subsoil conditions for percolation of septic tank effluent are not well suited because of low permeability, a high water table, high and fractured bedrocks, unfavorable ground slope, occasional flooding, and other features such as proximity to water supply well and other water sources, buildings, escarpments, and small lots (Table 25-4 to 25-9 and Figures 25-1 and Figure 25-2). The effluent quality from septic is not sufficient for satisfactory subsoil disposal. Further treatment with aerobic processes or intermittent sand filters may be needed. Additionally, periodic dosing of a disposal field by a pump or dosing syphon may be necessary.

25-7-3 Adverse Site Conditions Requiring Special Systems

Adverse site conditions make on-site subsoil disposal impractical. This may be because of impervious subsoils, a very high water table, high and fractured bedrocks, steep slopes, frequent flooding, and close proximity to a water supply well and other water resources, buildings, and the like. A drastic change in water conservation, treatment and recycling, and disposal technology may be necessary. The characteristics of adverse sites for subsoil disposal have been presented previously.

25-7-4 Description of On-site Disposal Options for Normal Soil Conditions

The on-site final disposal options for a normal site condition include (1) seepage pits, (2) gravel filled absorption trenches or a bed with different dosing arrangements, and (3) leaching infiltrator or chambers. The normal soil conditions for on-site disposal are presented in Sec. 25-7-1. A description of each option in the overall context of treatment, effluent application, and overall system configuration is presented below.[1,2,30,31]

Seepage Pit. These are deep wells or excavations. The high water table must be very deep. The dominant seepage surface is sidewalls. Pits are commonly 2–4 m in diameter and 3–6 m deep and often filled with rocks. Septic tank effluent is discharged into the pit, which percolates into the surrounding soil. The design hydraulic loading is 8–16 $L/m^2 \cdot d$. The construction details of seepage pits are shown in Figure 25-9(a). See page pits are generally discouraged by local regulatory agencies in favor of absorption trenches or beds.

Absorption Trenches or Beds. The gravel-filled absorption trenches and beds are the most commonly used methods for on-site disposal of septic tank effluent. A trench is generally 0.3–1 m wide and 0.3–1.5 m deep. The length and number may depend on the design hydraulic loading, which may be 10–20 $L/m^2 \cdot d$. The bottom is filled with crushed rocks or gravel to about a 15-cm depth. The perforated pipe is placed over the gravel layer and then covered by gravel. A semipermeable barrier is placed over the gravel, and then the trench is covered by backfill. Both bottom and sidewalls are infiltrative surfaces.

The intermittent or alternating dosing, serial distribution, or resting trenches enhance the operation. The trenches are generally narrow and have one percolation pipe. The absorption beds are wider and have more than one percolation pipe. The details of trench and bed are shown in Figure 25-9(b) and (c). Five arrangements of system components using gravel-filled absorption trenches of different dosing and resting alternatives are given below.[31]

Septic Tank Effluent Discharge by Gravity. The effluent from the septic tank receives intermittent gravity flow. A biomat will develop at the bottom, which will control the movement of fluid. This arrangement is shown in Figure 25-9(d).

Septic Tank with Serial Distribution. The effluent floods all soil surfaces. The drop box enables inspection of the system and controls the discharge into each trench. Caping the pipe outlets in the upper trench forces resting. The serial distribution automatically loads the upper trenches and minimizes the loading on lower trenches. It is used on sites that have gentle to steep slopes. This arrangement is shown in Figure 25-9(e).

Septic Tank with Alternating Trenches. This arrangement is such that one set of trenches rests while the other set treats the septic tank effluent. The valve box is used to control the flow. The arrangement is shown in Figure 25-9(f).

Septic Tank with Pressure Dosed Distribution. A pump or siphon doses a pressure distribution manifold that disperses the effluent evenly to each trench. The pressure manifold may include valves or plugs that permit control over trench loading or trench resting. This arrangement is shown in Figure 25-9(g).

Septic Tank with Shallow Trench Low-Pressure Pipe Distribution. A small diameter pipe located at a shallow depth receives pumped effluent. Effluent moves under pressure through small holes in the pipe and soaks the entire trench network area. Used in areas with high groundwater or shallow soils or on steep slopes that require hand excavation. The arrangement is shown in Figure 25-9(h).

Leaching or Infiltrator Chamber. These are open bottom, concrete chambers or arched plastic chambers that replace the trench or bed. Many are proprietary systems made of high-strength polyolefin chambers that sit directly in a 1-m-wide trench. The chambers interlock end to end along the length. The louvered sidewall allows effluent to pass laterally into the soil and prevents backfill intrusion into the chamber. When constructed, the chambers create an underground cavern that stores septic tank effluent and floods the soil surface prior to seeping downwards and from the sides. The arrangement of leaching field components is shown in Figure 25-9(i). An installation an Infiltrator® leaching chamber is shown in Figure 25-9(j).

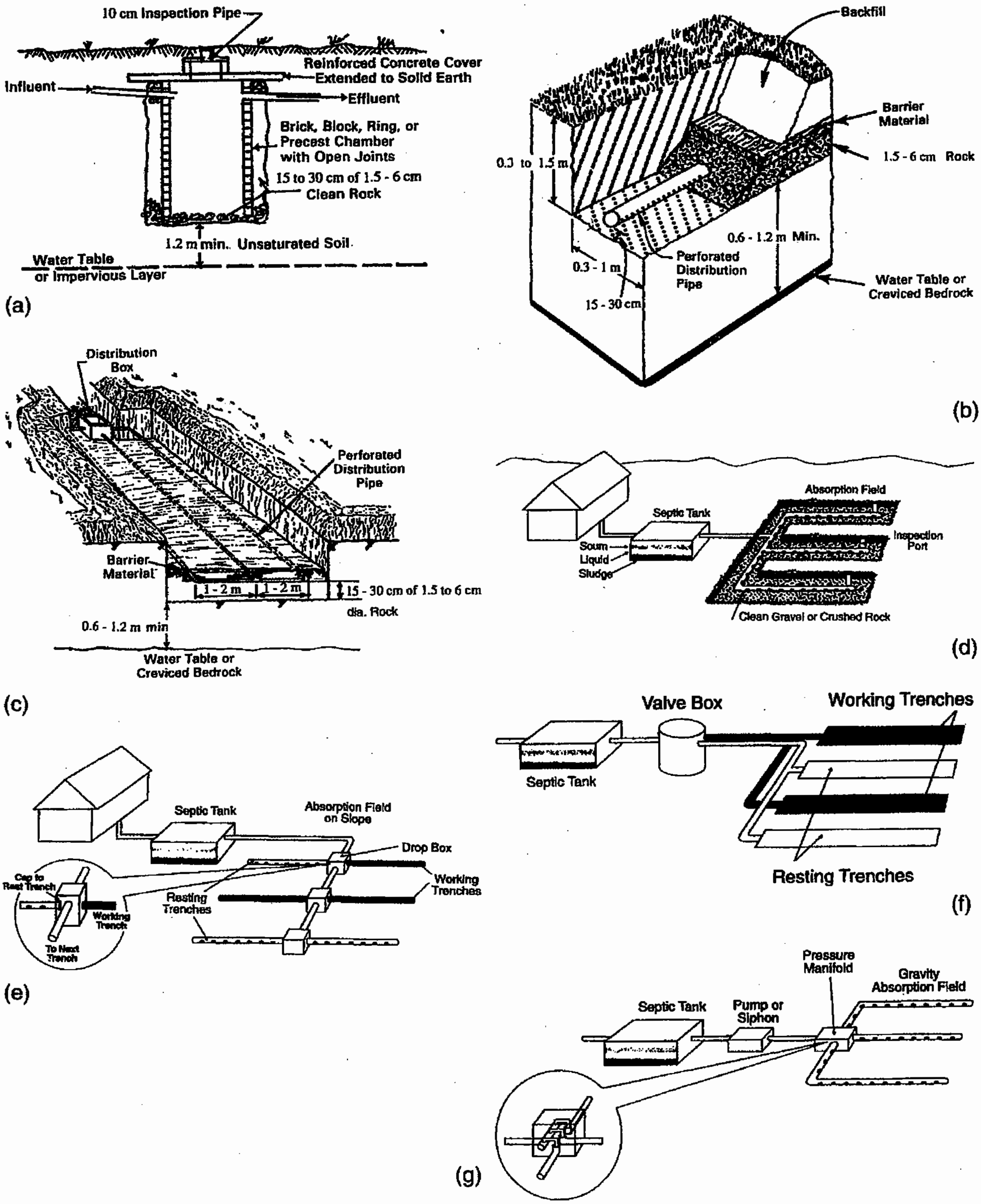


Figure 25-9 On-Site Disposal Field: (a) seepage pit; (b) details of a seepage trench; (c) details of seepage bed; (d) septic tank and gravel absorption trench with gravity flow; (e) septic tank with serial distribution; (f) septic tank with alternate trench; (g) septic tank with pressure dosed distribution.

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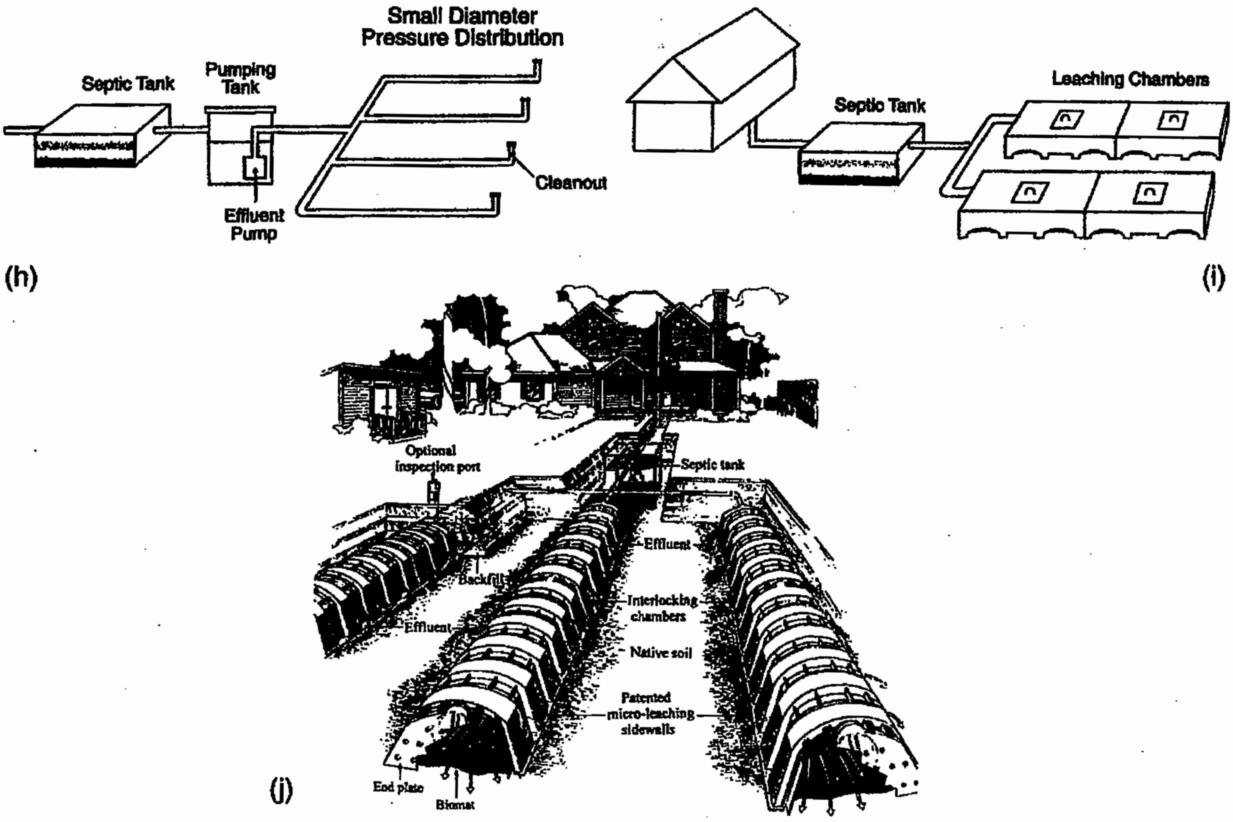

Figure 25-9—cont'd (h) septic tank with shallow trench, low-pressure pipe distribution, (i) arrangement of leaching chamber (from Refs. 1 and 31); and (j) installation of Infiltrator® leaching chamber (courtesy Infiltration System, Inc.).

25-7-5 Description of On-site Disposal Options for Difficult Site Conditions

The on-site final disposal options for difficult site conditions require higher quality effluent because subsoil has poor permeability and disposal conditions are not very favorable. A general discussion on difficult site conditions is given in Sec. 25-7-2. The on-site final disposal options for difficult site conditions include (1) trench and bed or leaching chambers with improved treatment; (2) shallow sand-filled, pressure-dosed disposal fields; (3) a mound system; (4) evaporation and absorption beds; (5) recirculating intermittent sand filter, disinfection, and surface discharge; and (6) constructed wetlands. These systems are briefly presented below.

Trench, Bed or Leaching Chamber. The system arrangements are similar to those discussed under normal site conditions. The septic tank effluent is further treated aerobically by suspended or attached growth or by recirculating sand filters, as shown in Figure 25-10(a). Additionally, the intermittent or alternating gravity or pressure dosing, serial distribution, resting trenches or beds, or leaching chambers are utilized to enhance the percolation rate. The design hydraulic loading is in the range of 10–15 $L/m^2 \cdot d$.

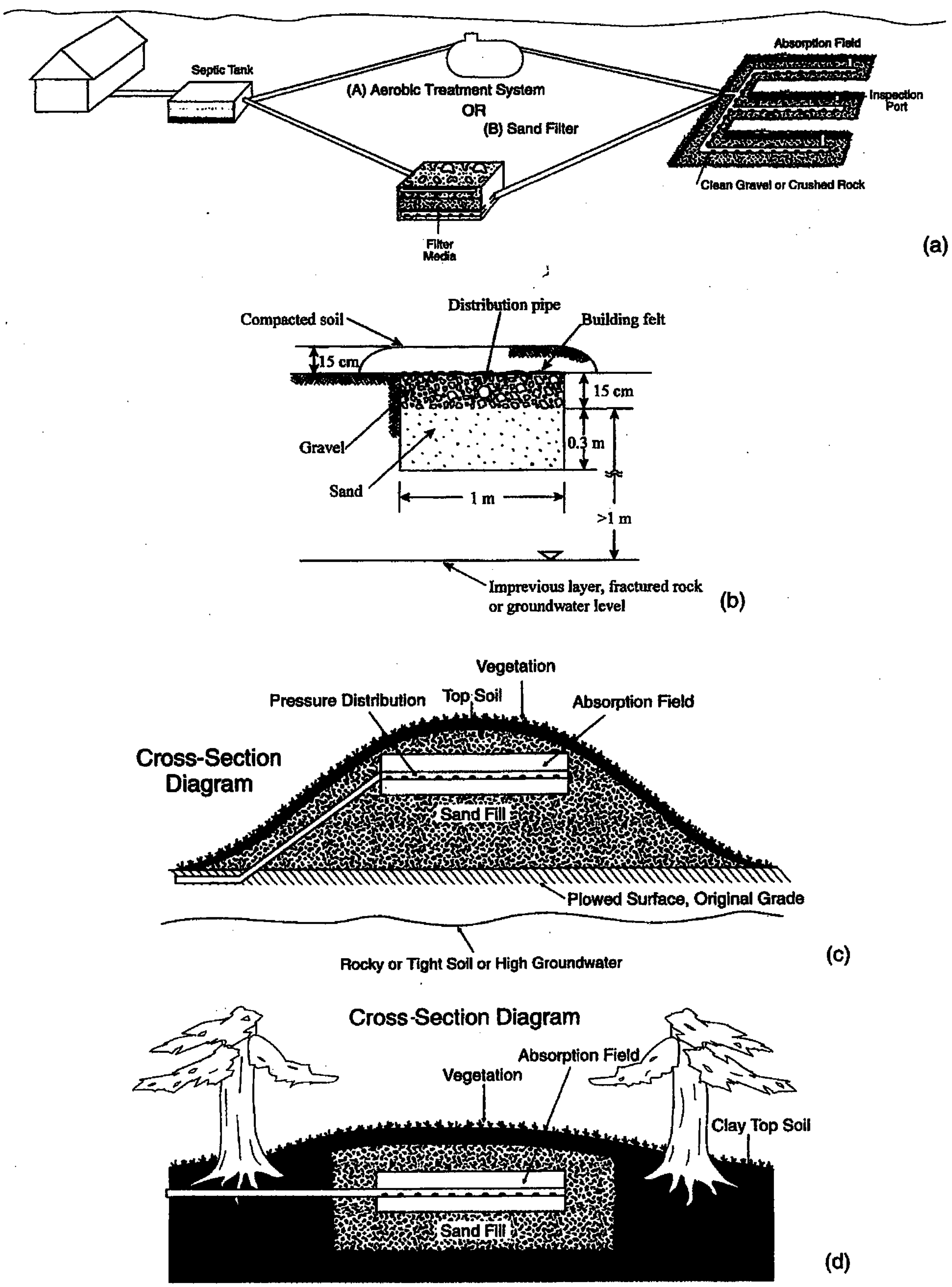

Figure 25-10 Subsurface Disposal Systems in Difficult Site Conditions: (a) septic tank in combination with aerobic or intermittent sand filter using percolation trench; (b) shallow sand-filled, pressure-dosed disposal field; (c) septic tank and mound; (d) evaporation and absorption bed.

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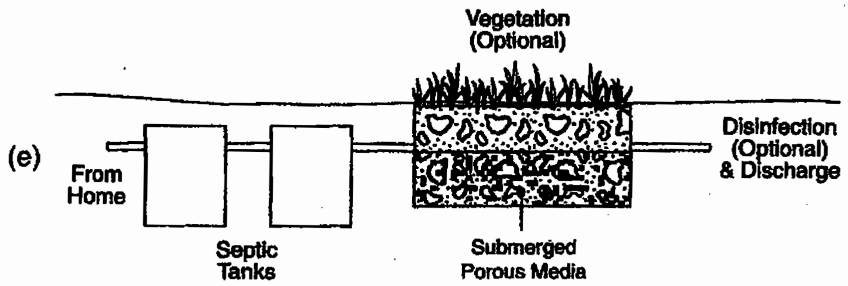


Figure 25-10—cont'd (e) constructed wetland (from Refs. 1 and 31).

Shallow Sand-Filled, Pressure-Dosed Disposal Field. This is a combination of intermittent sand filter and disposal field. The septic tank effluent is applied over the top of gravel; beneath is the sand bed. The bed is covered by compacted soil. The quality of effluent as it passes through the sand layer is high. Figure 25-10(b) shows a cross section of such a disposal field.

Septic Tank and Mound System. The mound system is a soil absorption system that is evaluated above the natural soil. The system consists of a gravel bed or trenches on top of a bed of sand placed above the plowed ground surface. The septic tank effluent is applied, which is treated before it moves into the natural soil. Good vegetation cover enhances evapotranspiration. The system works well in areas with high groundwater, high bedrock, or tighter clay soils. The details of the mound system are shown in Figure 25-10(c).

Evaporation and Absorption Bed. The evaporation and absorption beds maximize evapotranspiration. The effluent from a septic tank or aerobic system flows into gravel trenches or chambers in a mound of sandy soil. The trees that grow around the system and plants on top of the system pull liquid from the sand and transpire the water into the air. Some effluent may seep into the soil. This system is well suited for arid regions. The overall system components are shown in Figure 25-10(d).

Recirculating Intermittent Sand Filter, Disinfection, and Surface Discharge. The effluent from a septic tank is run through an open or buried recirculating sand bed for a single or repeated application of effluent. Effluent may be discharged into the water surface after disinfection or may be discharged into a percolation field. Significant nitrogen reduction may also be achieved. The design conditions and system layout are provided in Table 25-12 and Figure 25-8.

Constructed Wetlands. Effluent from a series of septic tank passes through a bed of rocks planted with reeds. Liquid evaporates and drains into a soil absorption system or discharges into a creek. Surface discharge requires disinfection. The system configuration is shown in Figure 25-10(e). Discussion on constructed wetlands may be found in Chapter 24.

25-7-6 Description of On-site Disposal Options for Adverse Site Conditions

Adverse site conditions may make subsurface disposal of effluent difficult or even impractical. The characteristics of adverse site conditions are covered in Sec. 25-7-3. No-discharge technology is often applied to meet the basic needs of on-site disposal. Among these, serious consideration is given to flow reduction, treatment and reuse of gray water for toilet flushing, and effluent reuse within the system or around the site. The on-site treatment and disposal options for adverse site situations presented in this section include (1) a holding tank, (2) evaporation lagoons, (3) waterless or ultra-low-flow toilet systems, (4) dual systems, and (5) land application systems.[1,2,32-35] Each of these options is briefly discussed below.

Holding Tank. Sewage flows from low-flush toilets and water-saving fixtures into a large watertight storage tank. The alarm in the tank signals the owner to have the sewage hauled away. Mostly recreational housings utilize holding tanks because of seasonal occupancy. The cost of pumping and hauling wastes is quite high. For an entire community, management by a public agency is frequently required, although contracting helps to reduce costs. The system details are shown in Figure 25-11(a).

Evaporative Lagoon. A series of septic tanks or other pretreatment systems discharge septic tank effluent or residential wastes into a lagoon. Sunlight and long storage times support the natural breakdown of the waste and kill off harmful organisms. The effluent evaporates, slowly seeps into the soil, or is given further treatment through land application. On-site lagoons require large lots and may be fenced. The basic arrangement is shown in Figure 25-11(b).

Waterless or Ultra-Low-Flow Toilets. Many types of waterless or low-flush toilets are utilized.[30,32,33] Among these are (1) composting toilets, (2) incinerating toilets, (3) water conservation toilets, and (4) recirculating toilets that recycle chemically treated wastes or treated wastewater or gray water to flush the toilets. A brief description of some of these systems may be found in Table 3-7. References 30–35 provide in-depth coverage on this subject. In recent years, dual-system and total recycle systems have received much interest in areas where site conditions are adverse for on-site waste disposal. These systems are briefly described below.

Dual-System. Two systems are used to treat the wastes. The wastewater from showers, sinks, kitchen, and laundry (gray water) is separated and treated. A small portion of treated gray water is used for flushing the low-flush (6 L or less) toilets. The remaining treated water is used over the landscape, disposed of in an absorption system, or discharged into surface water. The toilet waste, which is called black water, is stored in a holding tank similar to a septic tank with no overflow. The holding tank is periodically pumped out. Both systems are shown in Figure 25-11(c).

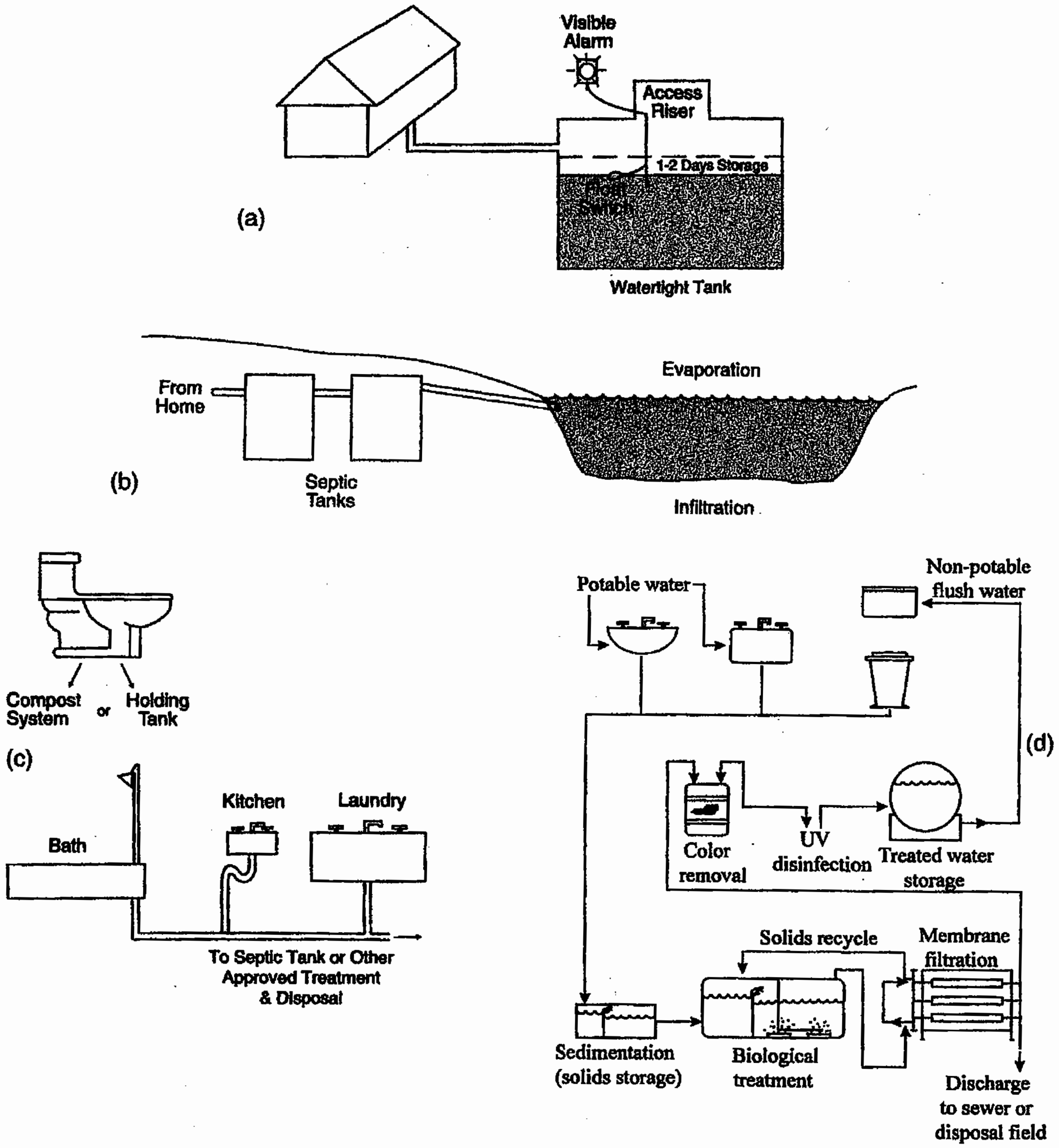


Figure 25-11 Small-Flow, On-site Wastewater Treatment, Reuse, and Disposal Systems: (a) holding tank; (b) evaporative lagoon; (c) dual systems: (i) black water, (ii) gray water; (d) complete system.

Total Recycle. The wastewater from a cluster of homes or small establishments is given total treatment. The effluent is reused for toilet flushing, and the remaining is reused over the landscape or discharged into surface water. The treatment scheme may include preliminary treatment, biological treatment, chemical precipitation, filtration, carbon adsorption, or membrane processes. A process diagram, which is expensive to build and operate, is shown in Figure 25-11(d).

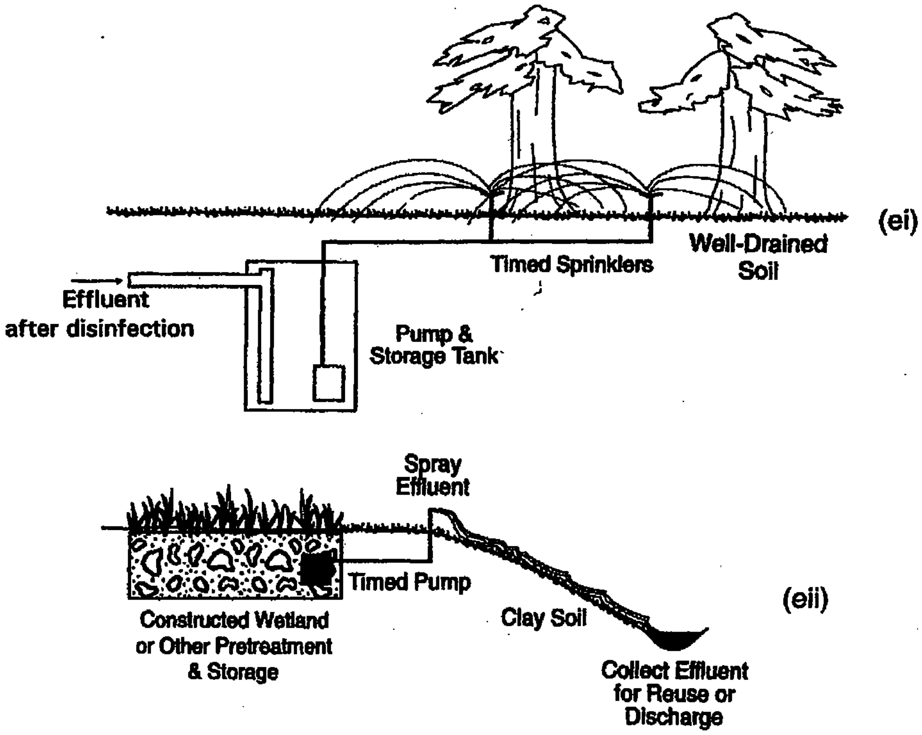


Figure 25-11—cont'd (e) land application: (i) slow-rate land treatment, (ii) overland flow (from Refs. 1 and 31).

Land Application. Common land application methods for small flows are slow-rate irrigation and overland flow. Effluent from a septic tank is further treated to a secondary level and stored. In slow-rate irrigation, timed sprinklers apply the effluent at night or below the soil surface to plants and trees in the surrounding area. The plants take up nutrients, and a portion of applied water is lost by evapotranspiration. This protects high groundwater in more permeable soils. Disinfection of effluent and fencing of the area may be required for individual home use. This system is more common in warm climates but is not widely permitted by health authorities. The system arrangement is shown in Figure 25-11(e)(i), and more information may be found in Chapter 24. Treated effluent from a lagoon or wetland is sprayed on the surface of a gentle, grass-covered slope. In an overland flow system, the effluent flows over the soil through the grass and collects at the base where it is disinfected before being discharged. It is best for tighter soils where absorption systems are not possible. Care is needed for the grassy land and disinfection system. This system is shown in Figure 25-11(e)(ii). Additional information may be found in Chapter 24.

25-8 WASTEWATER COLLECTION, TREATMENT, AND DISPOSAL SYSTEMS FOR SEWERED AREAS

Many small rural communities may require a centralized wastewater collection and disposal system because of high housing density, poor soil conditions, small lots, or expected rapid growth in the future. These systems typically consist of (1) a collection system to convey the wastewater away from each residence or establishment, (2) a treatment plant, and (3) an effluent disposal system. In this section the wastewater collection, treatment, and disposal systems for such communities and clusters of homes are provided.

25-8-1 Wastewater Collection Systems

To convey wastewater from individual homes or establishments to a central location requires a collection system. There are four types of wastewater collection systems that have been used in small communities.[2,3] These are (1) conventional gravity systems; (2) small-diameter, variable-slope sewers; (3) pressure sewers; and (4) vacuum sewers. The selection of one system over the other may depend upon the size, density, topography, climate, and financial resources. Each of these systems in the context of small communities is presented below.

Conventional Gravity Systems. Conventional gravity systems have been historically used in municipal systems. The design, construction, and operation techniques are well developed. Where topography permits, the gravity system will continue to be used in small and large communities. Chapter 7 is exclusively devoted to this subject.

Small-Diameter, Variable-Slope Sewers. Small-diameter, variable-slope (SDVS) gravity sewers collect effluent from septic tanks at each service connection and transport it by gravity to a treatment plant or gravity sewer. Such systems are also known as small-diameter effluent sewers, effluent drains, and small bore sewers.[35] The alignment of the pipes in both the horizontal and vertical directions can be curvilinear. The pipe network includes no closed loops. Uphill sections can be used, provided enough hydraulic grade exists to maintain flow in the desired direction and service connections are above hydraulic grade line. Minimum diameters are usually 10 cm (4 in.), but smaller sizes have also been used. Plastic pipe is typically used since it is economical in small sizes and resists corrosion by the septic wastewater. Cleanouts are provided for flushing the line. Manholes are used only at the major junctions of main lines. Air release risers are required at or slightly downstream of extreme summits in the sewer profile. Excavation depths are shallow because of the small diameter and flexible slope and alignment, and excavation volumes are typically much smaller than with conventional sewers.

Two types of SDVS gravity sewer systems are normally used in the United States: variable grade and minimum grade. Variable grade systems follow surface contours, taking the advantage of available gradient to maintain flow in the desired direction as long as there is no backflow into any service connection at peak design flow. Minimum grade systems are similar to conventional systems in which minimum downward slopes are needed. Recent designs blend both of these approaches, allowing variable grade but minimizing the number of flooded sections.

The service connections are generally equipped with a septic tank effluent pump (STEP) unit, creating a hybrid of the STEP pressure sewer. Design information on these sewers may be found in Refs. 2 and 36–39.

Pressure Sewers. Pressure sewer systems are operated by pumping and generally use smaller pipe diameters than conventional sewers. In sparsely populated areas, they usually result in lower construction costs as compared to conventional sewer systems. Depending upon the application, the pressure sewers are independent of slope and utilize a number of pressurizing inlet points and an outlet to a treatment facility, or to a downstream gravity sewer.

There are two major types of pressure sewer systems: the grinder pump (GP) system and the STEP system. The major difference between the two systems is in the on-site equipment and layout. Neither system requires any modification of household plumbing. In both designs the household wastes are collected in the sanitary sewer and conveyed by gravity to the pressurization facility. The discharge piping arrangement on individual connection includes at least one check valve and one gate valve to permit isolation of each pressurization system from the main sewer. GPs can be installed in the basement of a home to provide easier access for maintenance and greater protection from vandalism. Design information on pressure sewers may be found in Refs. 2, 36, 39, and 40.

Vacuum Sewer. A vacuum sewer has three major subsystems: (1) the central collection station, (2) the collection network, and (3) the on-site facilities. At a central collection station; a vacuum is generated that is transmitted by the collection network throughout the area to be served. Wastewater from conventional plumbing fixtures flows by gravity to an on-site holding tank. As soon as a required quantity of wastewater is collected into the holding tank, the vacuum interface valve opens for a few seconds, allowing the wastewater and a volume of air to be sucked through the service pipe and into the main line. The difference between the atmospheric pressure behind the wastewater and the vacuum ahead provides the primary propulsive force. The fact that both air and wastewater flow simultaneously produces high velocities that prevent blockages under normal operating conditions. Following the valve closure, the system returns to equilibrium, and the wastewater reaches the central collection tank. A conventional nonclog wastewater pump conveys the wastewater through a force main to a treatment plant or through a gravity interceptor. The design details on the vacuum system may be found in Refs. 2, 36, and 39.

25-8-2 Wastewater Treatment Systems

The wastewater treatment systems for small communities utilize preengineered package plants or individually designed and built facilities. The design and operation of most of these facilities in general may require some special considerations: (1) very large fluctuations occur in both flow and organic loading; (2) very large flows may occur because of oversized pumps in the collection system; (3) very small flows may create solids settling and self-cleaning problems in channels, pipes, and return sludge lines; (4) denitrification in the clarifier may cause solids carryover; (5) special care is required for scum, grease and foaming control, maintaining proper MLSS, and air supply; (6) there is rapid temperature change; and (7) sludge wasting and disposal must be considered. The treatment facilities in general utilize many physical, chemical, and biological treatment pro-

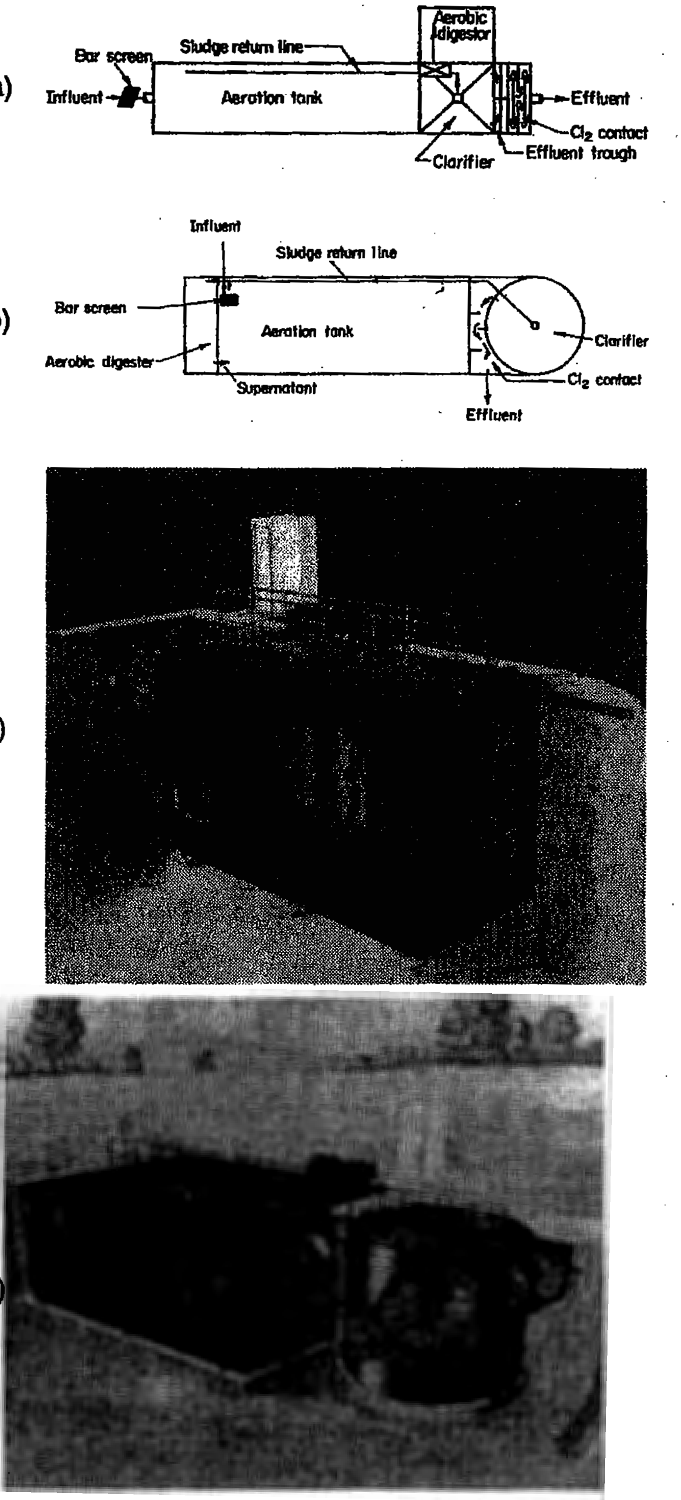
(a)
(b)
(c)
(d)


Figure 25-12 Preengineered Package Plant Process Arrangement: (a) layout of rectangular extended aeration plant; (b) extended aeration plant with circular clarifier; (c) rectangular plant installation; (d) rectangular plant with circular clarifier (courtesy Smith Pump Co., Inc).

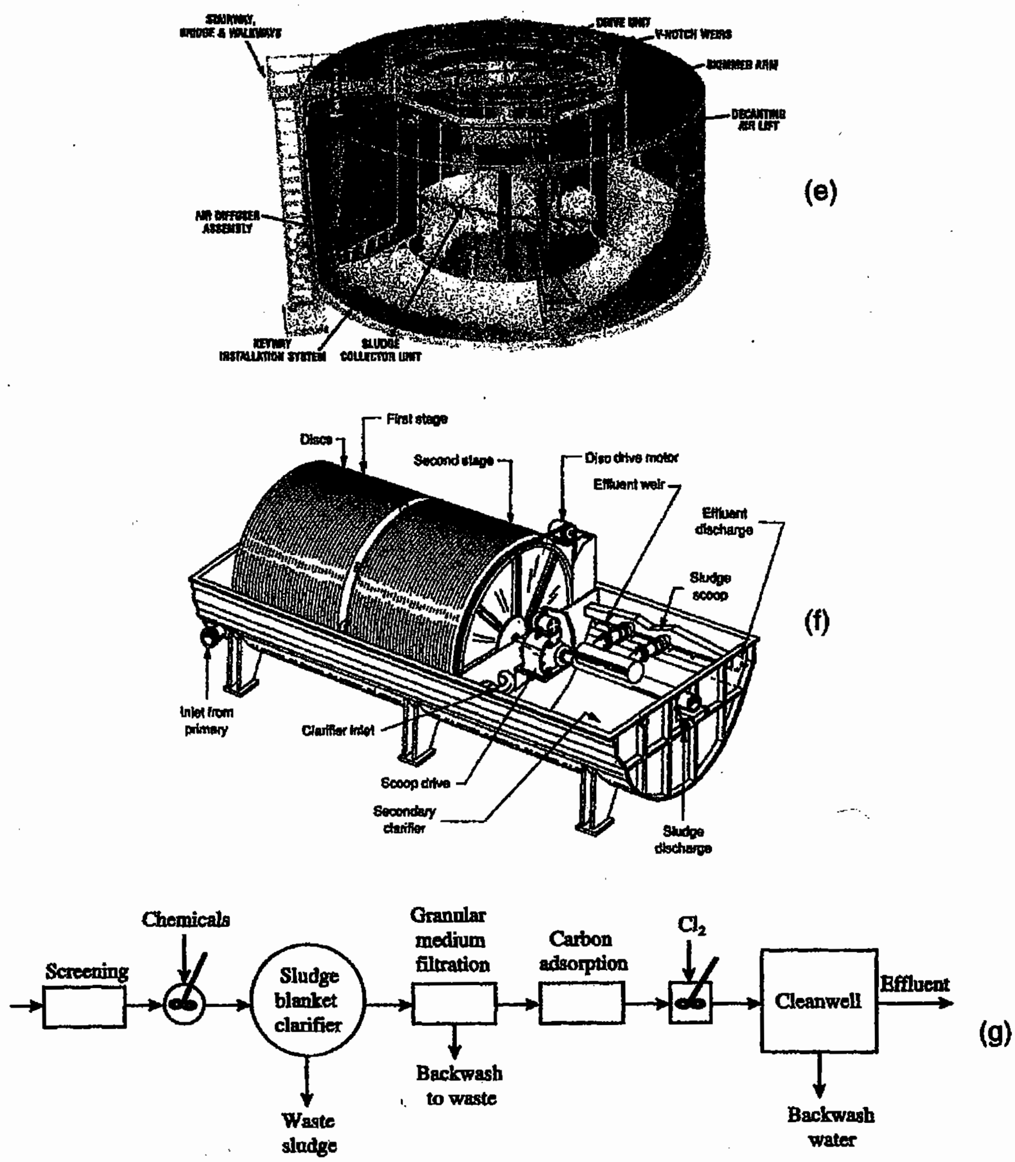


Figure 25-12—cont'd (e) circular contact stabilization package plant (courtesy Smith and Loveless, Inc.); (f) typical example of a rotating biological contactor package plant (from Ref. 2, used with permission of The McGraw-Hill Companies); and (g) physical/chemical treatment process arrangement.

cesses discussed throughout this book. A brief description of both package plants and individually designed facilities is given below with reference to many sections in the book.

Preengineered Package Plants. For small communities, subdivisions, clusters of homes, and small establishments, the package plants have received extensive popularity and rapid commercial growth in this country. The size of these plants may vary from 10–400 m^3/d. While there is a considerable variation in process design and hardware assembly among individual package plants built by various manufacturers, all can be classified as either biological or chemical.[41-43] Common biological treatment plants utilize extended aeration and contact stabilization modes of activated sludge, sequencing batch reactor, and rotating biological contactor. The sludge is aerobically digested and pumped

TABLE 25-13 Processes Commonly Used for Design and Construction of Wastewater Treatment Facilities for Sewered Communities

Treatment Processes	Reference
Package Plants	
Extended aeration	Table 13-3
Contact stabilization	Table 13-3
Sequencing batch reactor (SBR)	Table 13-3
Rotating biological contactor (RBC)	Sec. 13-4-2
Chemical processes	Sec. 12-5-2
Individually designed and built systems	
Septic tank	Sec 25-6-2
Imhoff tank	Sec 25-6-3
Septic tank and recirculating sand filter	Sec 25-6-6
Stabilization pond	Sec. 13-3-2
Aeration lagoon	Sec. 13-3-2
Oxidation ditch	Table 13-3
Sequencing batch reactor (SBR)	Table 13-3
Rotating biological contactor (RBC)	Sec. 13-4-2
Trickling filter	Sec. 13-4-1
Combine attached and suspend growth dual process	Sec. 13-4-4

out periodically. The chemical treatment plants utilize coagulation/precipitation, filtration, and membrane processes. While most of these processes have been presented throughout this book, a summary table is given (Table 25-13) to provide a ready reference of different sections where the process design information on these systems may be found.

Most package plants are complete factory-built, prefabricated systems shipped for installation. The site preparation may require excavation and a concrete foundation slab. The slab is generally below grade (to minimize freezing or rapid temperature change) and anchored to prevent flotation. Several subassemblies are shipped for field assembly where shipping height limitations may not permit transportation of a complete unit. Many package plant layouts, assembled units, and physical-chemical process trains are shown in Figure 25-12. The names and addresses of many manufacturers of package plants are provided in Appendix D.

Individually Designed and Built Facilities. These facilities are designed and built on the site. The treatment systems commonly utilized are stabilization ponds, aerated lagoons, and various modifications of suspended and attached growth reactors. Most of these processes have been presented in this book and are listed in Table 25-13 along with reference to various sections where additional information may be found.

TABLE 25-14 Effluent Disposal or Reuse Options for Small Communities

Disposal or Reuse	Minimum Treatment Required	Reference
Subsurface absorption	Primary	Sec. 25-5
Land application, slow-rate irrigation	Secondary	Secs. 15-2-4 and 24-2-1
Groundwater recharge	Advanced	Sec. 15-2-7
Wetlands, natural and constructed	Secondary	Sec. 24-2-2
Surface water discharge	Secondary	Sec. 15-2-4
Indirect reuse	Advanced	Sec. 15-2-8

25-8-3 Effluent Disposal and Reuse

The effluent disposal methods for small communities depend on the degree of treatment provided. Most common methods of disposal are subsurface disposal, land application, constructed wetlands, surface water discharge, and indirect reuse. All these methods have been discussed in different sections of this book. Many of these systems are listed in Table 25-14 along with a reference to various sections where additional information may be found.

25-9 PROBLEMS AND DISCUSSION TOPICS

25-1 A single family has three occupants. The water usage is 200 Lpcd. Calculate the concentration (mg/L) of BOD_5, COD, TSS, TN, Org.-N, NH_4^+-N, and TP from the midpoint mass loading given in Table 25-2.

25-2 In a single-family residence, the gray water constitutes 40 percent of total water usage. If the gray water is collected and treated outside the residence and used for toilet flushing, calculate the concentration of BOD_5, COD, TSS, TN, Org.-N, NH_4^+-N, and TP in the wastewater using midpoint mass loadings given in Table 25-2. Assume that the load of contaminants in the gray water is small. The total water usage before recycling was 250 Lpcd. Also determine the water usage rate after reuse.

25-3 Draw a process train to treat the gray water for reuse to flush the toilet. The toilets are equipped with a flush valve (Table 3-7).

25-4 Total coliform discharged per person per day is 10^{12} organisms. If wastewater flow is 200 Lpcd, calculate the concentration of coliform organisms per 100 mL of the wastewater.

25-5 A single-family residence has four occupants. The average wastewater flow is 180 Lpcd. The septic tank has an effective volume of 2.5 m^3. The percolation test result indicates an average percolation rate of 40 min/cm. Obtain the corresponding design hydraulic loading rate from Table 25-8. The percolation trench is 1 m deep and 0.5 m wide, and soil cover is 0.3 m. Calculate the following:

(a) average detention time in the septic tank

(b) the vertical permeability of the proposed disposal site ($m^3/m^2 \cdot d$)

(c) total length of the percolation trench. Assume that percolation occurs from the trench bottom and sidewalls. Provide two percolation trenches.

25-6 A single-family residence lot has a percolation trench for septic tank effluent disposal. The groundwater table is sloping in the northwest direction. Draw the plan to show the residence, septic tank, and four disposal trenches that are arranged in parallel.

25-7 A typical recirculating intermittent filter is used to polish the effluent from a septic tank. Draw the unit arrangement, influent, effluent, and recirculation lines and estimate the effluent quality from the filter.

25-8 The basic design features of a recirculating intermittent filter are given in Figure 25-8. Design a recirculating intermittent sand filter for a residential subdivision that has a total wastewater flow of 3500 L per day. The filter is designed to treat the effluent from a septic tank. Your design should include the following:
(a) filter dimensions
(b) cross section of the filter showing the gravel, filter sand and underdrains
(c) specifications of the filter media
(d) influent and effluent arrangement

25-9 A factory-built, extended aeration plant is designed to serve a population of 50 residents. The wastewater flow is 200 Lpcd. The extended aeration plant has the following components: aeration basin, final clarifier, return sludge pumps, chlorination and dechlorination facility, and aerobic digester. Design each component and sketch the facility. The peak flow is three times the average flow. Summarize the following design information: MLVSS, SRT, *F/M*, HRT, Q_r/Q, WAS, and effluent quality.

25-10 Design an aerated lagoon to treat 150 m^3/d septage. The characteristics of the septage are given in Sec. 25-6-2. The aerated lagoon has floating aerators. The bottom and sidewalls are double lined. The sidewalls are at a slope of 1:1, and the liquid depth is 3.3 m. Determine the dimensions of the lagoon, number of aerators, and power requirement. Draw the plan and cross section of the lagoon.

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26

Management of Stormwater and Combined Sewer Overflows

26-1 INTRODUCTION

The discharge of stormwater runoff and combined sewer overflows (CSOs) has resulted in contamination problems that have often prevented the attainment of water quality standards in many receiving waters. The contaminants found in stormwater and CSOs include bacteria, solids, BOD, nutrients, metals, pesticides, and other potentially toxic constituents. Yet contaminants are discharged during rainfall in many older towns in the United States. This problem is more serious in other countries where separate collection systems are not provided.

The purpose of this chapter is to provide information on stormwater runoff and combined sewer overflows. The topics presented in this chapter include (1) current status; (2) characteristics, impacts, and management of stormwater runoffs; and (3) characteristics, impacts, and management of CSOs.

26-2 CURRENT STATUS

26-2-1 Stormwater Runoff

The 1992 Needs Survey Report to Congress indicated that stormwater runoff from urban areas is a significant contributor to the surface water quality impairment of the nation's waters.[1] Stormwater runoff from urban and industrial areas typically contains significant quantities of pollutants that are similar to those found in wastewater and industrial discharges and, consequently, have been found to cause similar impacts on water quality. Pollutants commonly found in stormwater runoff include nitrogen, phosphorus, sediments, heavy metals, biochemical oxygen demand (BOD), and synthetic organic compounds such as pesticides and herbicides. In addition to pollutants, the increased quantity of stormwater discharged from rapidly urbanizing areas also poses a threat of significant impact on aquatic ecosystems because of physical alterations.

To help improve the quality of stormwater discharges, Congress amended the Clean Water Act in 1987, which directs the EPA to develop National Pollutant Discharge Elimination System (NPDES) permit application requirements for the following classes (types) of stormwater discharges:[2] (1) discharges from municipal separate storm sewer systems serving a population of 100,000 or more, (2) stormwater discharges associated with industrial activity, and (3) stormwater discharges that contributed to a violation of a water quality standard or are significant contributors of pollutants to surface waters. Thus, the discharge from municipal separate storm sewer systems will not necessarily require traditional end-of-pipe controls; rather, municipalities will be required to develop and implement site-specific stormwater management programs.

Under NPDES regulations, municipalities submit a two-part permit application for discharges from their stormwater systems. The permitting requirements will include a stormwater system inventory and baseline condition and description, followed by monitoring and planning for stormwater management. The monitoring will involve (1) measures to reduce pollutants in runoff from commercial and residential areas, (2) measures to direct and remove illicit connections and improper disposal into storm sewers, (3) measures to reduce pollutants in runoff from industrial sites, and (4) measures to reduce pollutants in runoff from construction sites.[3] Municipalities must develop stormwater management plans that include controls to reduce the discharge of pollutants to the maximum extent practicable. The management plans must also address the legal, administrative, and financial aspects of the municipality's stormwater control program.[4]

26-2-2 Combined Sewer Overflows

On April 11, 1994, EPA signed a final regulation that outlines the National Combined Sewer Overflow (CSO) Policy. Like all point sources of pollution discharges, CSOs are covered under the Clean Water Act through NPDES permitting. But until recently, limited guidance was developed for issuing permits that are specifically aimed at CSO discharges.[5,6]

The 1992 Needs Survey Report to Congress indicates that currently about 1100 communities served by 1303 CSO facilities nationwide use combined sewer systems, which are designed to carry sanitary and industrial wastewater and stormwater. These facilities are mainly located in older cities in the Northeast, the mid-central states, and along the West Coast. Combined sewer overflows occur when the capacity of the combined sewer system is exceeded during a storm event. During these storm events, part of the combined flow in the collection system is discharged untreated into the receiving waters.[1]

The CSO policy establishes a framework and lays out clear expectations for municipalities, NPDES permitting and enforcement authorities, and state water quality standard authorities for controlling CSOs. The municipalities were expected to implement the nine minimum controls and to submit documentation of their implementation no later than January 1, 1997. The nine minimum controls are (1) proper operation and maintenance, (2) maximum use of the collection system for storage, (3) review and modification of the pretreatment program, (4) maximization of flow to the POTW for treatment, (5) prohibition of CSOs during dry weather, (6) control of solid and flotable materials, (7) public notification, (8) monitoring of CSO impacts and efficiency of controls, and (9) pollution prevention.[7]

The municipalities also are expected to undertake immediately the development of a long-term CSO control plan. The long-term plan should include the following key elements: (1) a comprehensive characterization of the combined sewer system, CSOs, and impacts on the receiving water bodies; (2) special consideration for sensitive environmental areas; (3) an evaluation of a range of CSO control alternatives; (4) coordination with the NPDES permitting authority and state water quality enforcing authorities when selecting control measures; (5) development of a public participation plan; (6) a schedule for implementing the selected CSO control measures that considers the financial resources of the municipalities; and (7) implementation of a postconstruction water quality monitoring program.[5] The policy provides the municipality with two approaches for showing that its selected CSO control will be sufficient to meet the water quality standards: (1) the presumption approach and (2) the demonstration approach. Under the presumption approach, the municipality can provide a specified level of control that is presumed to meet water quality standards unless there is data showing otherwise. For example, one of the specified levels of control is no more than an average of four to six overflow events per year. Under the demonstration approach, the municipality can provide information and data showing that the selected CSO control actually meets water quality standards.[5]

The permitting authority (EPA or an approved state agency) must (1) review and revise, as appropriate, the state CSO permitting strategy developed in response to the 1989 EPA strategy; (2) develop and issue Phase I permits requiring implementation of nine minimum controls, submit documentation of their implementation, and develop and submit a long-term CSO control plan; and (3) develop and issue Phase II permits requiring continued implementation of the nine minimum controls, implement the CSO control measures selected from the long-term CSOs control plan, and take appropriate enforcement measures.

26-3 CHARACTERISTICS, IMPACTS, AND MANAGEMENT OF STORMWATER

26-3-1 Causes and Effects of Stormwater Runoff

Stormwater runoffs from urban areas create flooding and water quality degradation, As more and more land becomes covered with buildings, roads, parking lots, sidewalks, and other impervious surfaces, stormwater is prevented from percolating into the soils. Instead, it runs off those impervious surfaces and drain rapidly into the nearest water body, causing[8]

1. Flooding because of the increased peak flow
2. Accelerated erosion of stream channels that may deposit silt into lakes and reservoirs

3. Reduced groundwater recharge, which may reduce base flow of streams during periods of dry weather
4. A large quantity of debris and contaminants such as organics, salt, sand and silt, heavy metals, oils, and pesticides to be washed off into receiving waters. It has been estimated that runoff from the first hour of a moderate-to-heavy storm in a typical U.S. city will contribute a greater pollution load than would the city's untreated sanitary wastewater during the same period of time.
5. In older urban areas, a combined sanitary and stormwater sewer system to overflow during storm events, which may cause gross contamination of natural waters
6. Coastal waters to be adversely impacted by pollutants from urban runoffs. Beach closures, contaminated shellfish, and loss of commercial and recreational fisheries are common in coastal waters of many large metropolitan areas.

The hydrological changes resulting from urbanization is shown in Figure 26-1. A comparison of pollutant load from a hypothetical city runoff and raw sanitary wastewater is provided in Table 26-1.

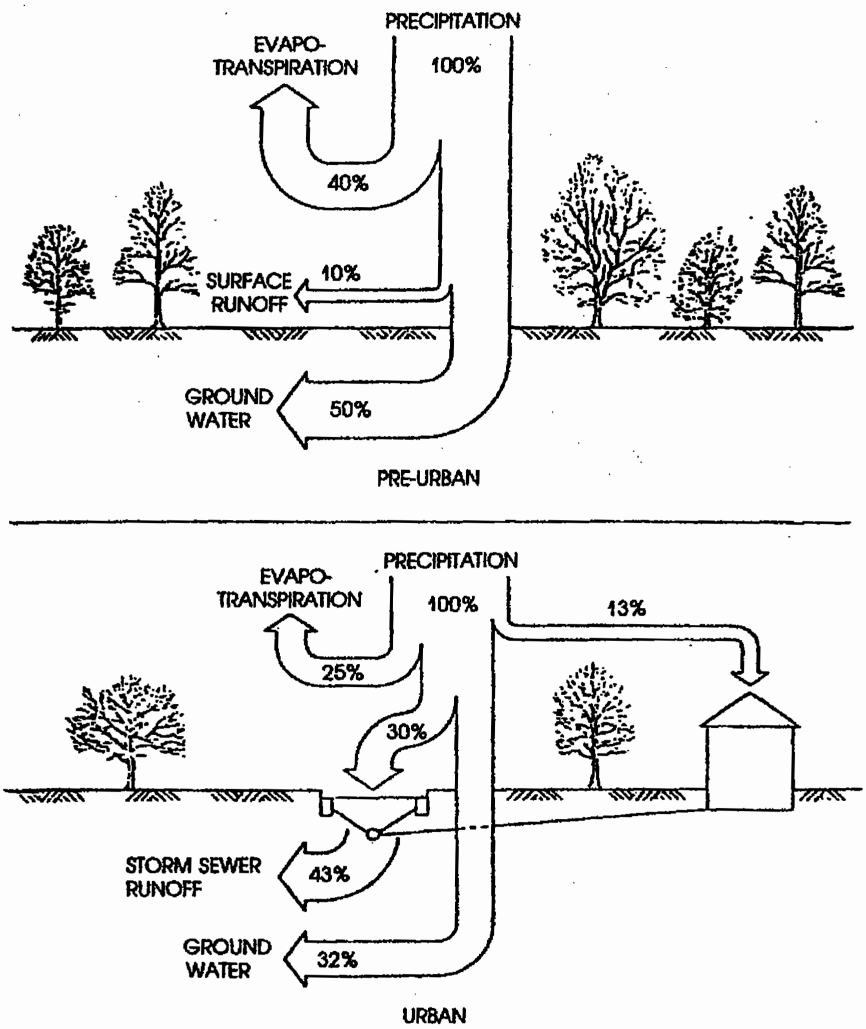


Figure 26-1 Comparison of Water Budget before and after Urbanization (from Ref. 8).

TABLE 26-1 Comparison of Pollutant Loads in Hypothetical City Street Runoff with That of Raw Sanitary Sewage Resulting from a Hypothetical City

	Contaminant Loads from Street Surface Runoff (kg/h)	Raw Sanitary Sewage[a] (kg/h)	Ratio of Street Runoff to Sewage	Secondary Plant Effluent (kg/h)
Settleable and Suspended Solids	280,000	650	430	65
BOD	2800	550	5	55
COD	6500	850	8	60
Total coliform organisms	4000×10^{10} organisms per h	$46{,}000 \times 10^{10}$ organisms per h	0.0087	4.6×10^{10} organisms per h
Kjeldahl nitrogen	440	105	4.2	10
Phosphates	220	25	8.8	1.3
Zinc	130	0.42	310	
Copper	40	0.09	470	
Lead	115	0.07	1,800	
Nickel	10	0.02	480	
Mercury	15	0.14	110	
Chromium	22	0.09	260	

[a]Loadings discharged to the receiving waters (average hourly rate).
Source: Adapted from Refs. 8 and 9.

26-3-2 Stormwater Flow Estimation

The quantity of stormwater runoff from urban areas is determined for design of storm sewers. The runoff is dependent upon the size of the area, topography, and rainfall. Unfortunately, the rainfall events are predictable only in a statistical sense, and whatever return period may be selected as a design, some storms of higher intensity will occur, which will overtax the system. The actual quality of runoff therefore will be a function of the statistical frequency of the design storm, distribution of storm in time (the hyetograph), the antecedent conditions, the season of the year, and the physical layout of the service area. Several methods are used for the estimation of flow. These are (1) the rational method, (2) the Soil Conservation Service (SCS) technique, (3) empirical formula or rainfall–runoff correlation equations, (4) hydrograph method, (5) the inlet method, and (6) computer simulation techniques. The rational method has been most common in the past, but now computer simulation techniques are extensively used. Therefore, the rational method and computer simulation techniques are presented below. The other methods are briefly described in Table 26-2.[10-15]

TABLE 26-2 Summary of Other Methods for Runoff Estimation

Method	Description	Reference
The SCS technique	The Soil Conservation Service (SCS)[a] of the U.S. Department of Agriculture developed a method that has been further adapted to a computerized simulation technique. This technique hinges on determination of a runoff curve number (CN).	10, 11, 15
Empirical formula or rainfall–runoff correlation equations	A number of runoff formulations have been empirically developed for various localities. These equations are specific to the area and utilize extensive runoff measurements in times of heavy rainfall.	11, 14, 15
Hydrographic techniques	Use the unit hydrograph,[b] which may be constructed from the existing records of rainfall and runoff or synthetic methods. Peak flow rates from urban and small watersheds can be determined for a storm of given duration by lagging unit hydrographs.	10, 11, 13, 14
Inlet method	The inlet method is based on rainfall measurements and inlet and sewer gaugings in urban areas. The method comprises three parts: (1) flow to each inlet, (2) attenuating the peakflow, and (3) summing the attenuated peaks to determine the total peak at the desired point.	10, 13, 16, 17

[a]The Soil Conservation Service (SCS) is now Soil Survey Division (SSD) of the Department of Agriculture.
[b]The unit hydrograph is defined as the hydrograph of surface runoff resulting from an effective rainfall over a unit of time (less than the time of concentration).

Rational Method. The rational formula for estimating peak runoff was introduced in the United States in 1889. Since then, it has been the most widely used method for designing drainage facilities for small urban and rural areas. This relationship is expressed by Eqs. (26-1)–(26-3):

$$Q = 0.00278\, CIA \text{ (SI units)} \tag{26-1}$$
$$Q = CIA \text{ (U.S. customary units)} \tag{26-2}$$
$$Q = C_a CIA \text{ (U.S. customary units)} \tag{26-3}$$

where

Q = maximum rate of runoff, m^3/s (ft^3/s)
A = drainage area contributing flow to the point of study, ha (acre)

C = runoff coefficient assumed dimensionless. Typical values of C for storms of 5–10 years' return periods are summarized in Table 26-3.

C_a = correlation factor to account for return period or antecendent conditions. The value of C_a varies from 1 to 1.25, and there is no need to adjust the C coefficient for hydrological conditions.[18]

I = uniform rainfall intensity lasting for a critical period of time t_c, mm/h (in./h); t_c is time of concentration in min

TABLE 26-3 Typical Runoff Coefficients C *for 5- to 10-Year Rainfall Frequency*

Description of Area	Runoff Coefficients
Business	
Downtown areas	0.70–0.95
Neighborhood areas	0.50–0.70
Residential	
Single-family areas	0.30–0.50
Multifamily, detached	0.40–0.60
Multifamily, attached	0.60–0.75
Residential (suburban)	0.25–0.40
Apartment dwelling areas	0.50–0.70
Industrial	
Light areas	0.50–0.80
Heavy areas	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yard areas	0.20–0.40
Unimproved areas	0.10–0.30
Streets	
Asphaltic	0.70–0.95
Concrete	0.80–0.95
Brick	0.70–0.85
Drives and walks	0.75–0.85
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

Source: Adapted in part from Refs. 10–13.

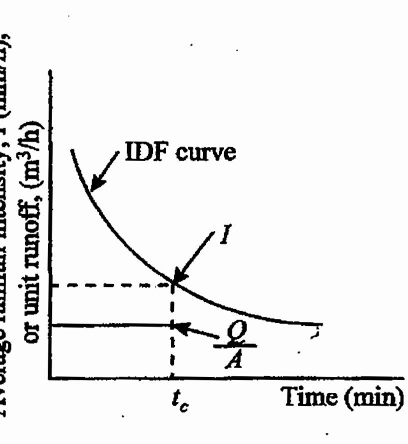


Figure 26-2 Rainfall–Runoff Relation for the Rational Method.

The rationale for the method is based on the concept that application of a steady, uniform rainfall intensity will cause runoff to reach its maximum rate when all parts of the watershed are contributing to the outflow at the point of design. That condition is met after the elapsed time t_c, the time of concentration, which usually is taken as the time for a wave to flow from the most remote part of the watershed. At this time, the runoff rate matches the net inflow rate. This concept is shown graphically by the intensity-duration-frequency (IDF) curve (Figure 26-2). The peak intensity of unit runoff Q/A is proportional to the value of I defined at t_c. The constant of the proportionality is thus runoff coefficient (Q/IA). It may be noted that Q/A is a point value and that the relation, as it stands, yields nothing of the rest of the hydrograph.

Many relationships that express the intensity of rainfall and time of concentration as a function of physical characteristics of the drainage area have been developed for many parts of the country. Additionally, flow estimation and design examples of storm sewers are also available in the literature. Readers may find in-depth coverage on these subjects in Refs. 10–14.

Computer Simulation Techniques. Over the past 50 years, efforts have been made to utilize many computational techniques to solve complex problems of urban hydrology. Many computer-based models have been developed that are capable of producing (when calibrated properly) a great deal of information concerning the response of a drainage system to any selected meteorological conditions. These models vary a great deal in the level of details required and sophistication with which the hydraulic and hydrological factors are modeled. The most useful readily available technique for accurate routing in an urban drainage system involves solution of the continuity equations and momentum equations. A number of approximate routing techniques using kinematic wave equations are also used to minimize computer time. Although there are a large number of computer models available, currently, the most commonly used models in urban hydrology are HEC-1, STORM, and SWMM. A brief summary of these models is presented below.

HEC-1. The U.S. Army Corps of Engineers developed this computer simulation model. It is one of a family of hydrological/hydraulics computer models. It has the ability to model many components of urban drainage systems; it has internal options to generate hydrographs for routing flows. The data required include a hyetograph, infiltration/ detention functions and sufficient hydraulic inputs. The program utilizes a number of storage routing techniques and can model pumping from one basin to another.[10,11,19]

STORM. The U.S. Army Corps of Engineers developed the Storage, Treatment, Overflow, and Runoff Model (STORM) to assist in estimating pollutant loading resulting from urban runoffs. STORM is a continuous simulation model that permits analysis of multiple storm events. The hydraulic routing procedures are far simpler than those in HEC-1 and therefore require less data and yield less definitive results.[10,11,20]

SWMM. The U.S. Environmental Protection Agency developed the Stormwater Management Model (SWMM) over an extended period of time. This model has substantial capabilities beyond simple stormwater modeling. The information is introduced in the form of unit hyetographs of different subareas and characteristics of subareas. The overland flow is routed, which considers local storage and keeps track of flows in excess of conduit capacity. The excess flows are stored at the inlet until the system can accommodate them. The program prints messages at each time step and location at which surcharging occurs and reports the volume of water stored.[10,11,21]

SWMM has the capability of modeling quality, as well as quantity, of flow in storm sewers and combined sewerage systems. The stormwater quality is predicted based on land use and sanitary and industrial wastes if introduced into the system. The quality of combined sewer overflows as predicted by SWMM is discussed in Sec. 26-4.

26-3-3 Stormwater Management

The traditional approach of stormwater management has been aimed at removing the water from the site as quickly and as efficiently as possible. Unfortunately, this approach has two serious effects: (1) It increases the rate and volume of runoff thereby contributing to flooding, scouring, and erosion of streambanks and landscape, and (2) pollutants accumulated over the builtup areas are transported efficiently, and the drainage may cause a greater pollution load to the receiving waters. An ideal stormwater management system should allow absorption and retainment of water on the site where the rainfall occurs over a given time. The quantity and quality of water leaving the site would not be significantly different than that if the site had remained undeveloped.[8]

Objectives. The stormwater management program in urban areas typically should have the following objectives:

1. Prevent increased runoff from new land development to reduce potential flooding and flood damage.
2. Minimize the erosion potential from a development or construction project.
3. Increase water recharge into the ground.
4. Enhance the quality of stormwater runoff to prevent water quality degradation in receiving waters.

5. Reduce stream bank erosion to maintain stream channels for their biological functions, as well as for drainage.
6. Assure the adequacy of existing and proposed culverts and bridges.
7. Prevent reductions in stream base flow by new land developments.

Control Methods. These objectives can be achieved by installing and maintaining properly designed stormwater management and erosion control systems. These systems in the broadest sense consist of controlling runoff from developed areas and land areas that are undergoing development.

Stormwater runoff can be controlled from newly developing areas or by retrofitting practices to control runoff from areas of existing development. In general, stormwater can be managed by employing one or more of the control techniques for on-site management, regional management, and watershed-wide management. The most common control facilities utilize infiltration practices that consist of basin, pit, trench, or impoundment. These facilities help to attenuate peak discharges, reduce runoff volume, provide groundwater recharge, and remove many contaminants. The recommended separation distance between the bottom of these facilities to a high groundwater table is 1 m. These facilities should be sufficiently far from wastewater percolation fields. The estimated long-term pollutant removal rates for these facilities are provided in Table 26-4. Many of these facilities and basic design information are summarized in Table 26-5. Other facilities intercept the runoff in sheet flow, and vegetative land retards the flow and remove contaminants. Such facilities are also presented in Table 26-5. In-depth coverage on this subject may be found in Refs. 8 and 22–25.

Treatment Facility. Approaches for reducing pollution from separate stormwater discharges are now in the early stages of development and evaluation. It is anticipated, however, that in many cases the benefits obtained by construction of treatment works for abatement purposes will be small compared with the costs. On the other hand, the techniques mentioned in Table 26-5 for control and prevention of stormwater impact can be more cost-effective. The policy of the EPA is, therefore, not to use construction grants

TABLE 26-4 Estimated Long-Term Pollutant Removal Rate for Infiltration Facilities

Urban Pollutant	Removal Rate	Limiting Factor
Sediment	90%	Should actually be trapped before reaching the facility
Total phosphorus	65–75%	Leaching of remineralized organic P
Total nitrogen	60–70%	Leaching of soluble nitrate
Trace metals	95–99%	Behavior similar to sediment
BOD	90%	Leaching of dissolved organic matter
Bacteria	98%	Straining

Source: Adapted from Ref. 8.

TABLE 26-5 Stormwater Control Facilities Description Design Criteria and Performance

Control Facility	Description	Design Information
Infiltration basin	These are constructed by excavating a depression in the ground down to relatively permeable soil or by constructing an embankment or dam on a permeable base. They exfiltrate stormwater runoff into the underlying soil. The details of an infiltration basin are shown in Figure 26-3(a).	Reduce 60–90 percent of post-development annual runoff volume. They can serve drainage areas up to 20 ha and function best if soil infiltration rate is 1.5 cm/h. Maximum allowable ponding is 40 h.
Infiltration trench and dry well	These are subsurface trench or well backfilled with gravel or coarse stone aggregate. These provide large storage capacity and more infiltration surface than infiltration basin. It is important that sediment is prevented from entering the infiltration trench. There are many types of infiltration trenches in use. One common type is shown in Figure 26-3(b).	These are 0.6–3 m in depth. They are seldom used to control areas larger than 4 ha.
Porous pavement	Porous asphaltic paving material consisting of graded aggregate cemented together by asphalt into a coherent mass that has sufficient interconnected voids to provide a high rate of permeability to water. These are used for small areas with low traffic volume such as parking lots. A typical porous pavement design is shown in Figure 26-3(c).	Design is based on 2-year design storm. The stone reservoir underneath should drain in 72 hours or less.
Constructed wetlands	Both surface water and subsurface constructed wetlands may provide impoundment in a low land area. They provide infiltration, removal of pollutant, and improvement in water quality.	Shallow depth of 0.3–1 m performs better. Average area is about 30 percent of the subwatershed area draining into it. Design details may be found in Chapter 24.

Continued

TABLE 26-5 Stormwater Control Facilities Description Design Criteria and Performance—cont'd

Control Facility	Description	Design Information
Grassed swales	These are designed to intercept sheet flow from surrounding lands and to provide for as much detention and/or infiltration as possible. They have limited capacity to accept runoff. Figure 26-3(d) shows an arrangement of grassed swales.	Side slope should be no greater than 3:1 (h:v). Design percolation rate 1.5 is cm/h.
Filter strip	These strips consist of close-growing grasses or older densely planted vegetation established at the perimeter of disturbed or impervious areas to intercept runoff in sheet flow and remove particulate contaminants. A shallow stone trench that follows the contour can be used as a level spreader at the top of the strip to distribute flow evenly. Figure 26-3(e) shows a schematic of filter strip.	Not used on slopes steeper than 15 percent. A minimum length of 6 m is necessary. Additional length is 1.2 m for each 1 percent slope as a safety factor.

Source: Adapted in part from Refs. 8 and 21–25.

for treatment works to control pollution from separate discharges of stormwater, except under unusual conditions where the project clearly has been demonstrated to meet the planning requirements and criteria developed for CSOs.

Protecting Sensitive Environmental Resources. As presented above, many techniques are used to control and treat stormwater runoffs. It may be noted that none of these techniques is fully effective all the time. Therefore, for protection of a sensitive environment such as a high-quality trout stream, water supply reservoir, lake, natural wetlands, or coastal estuary, the most reliable and perhaps most appropriate approach may be restricting the growth and development in the contributory drainage through proper planning and zoning. If development is absolutely necessary, several stormwater runoff abatement techniques (Table 25-5) should be designed as a prudent course of action to protect the sensitive water resources.

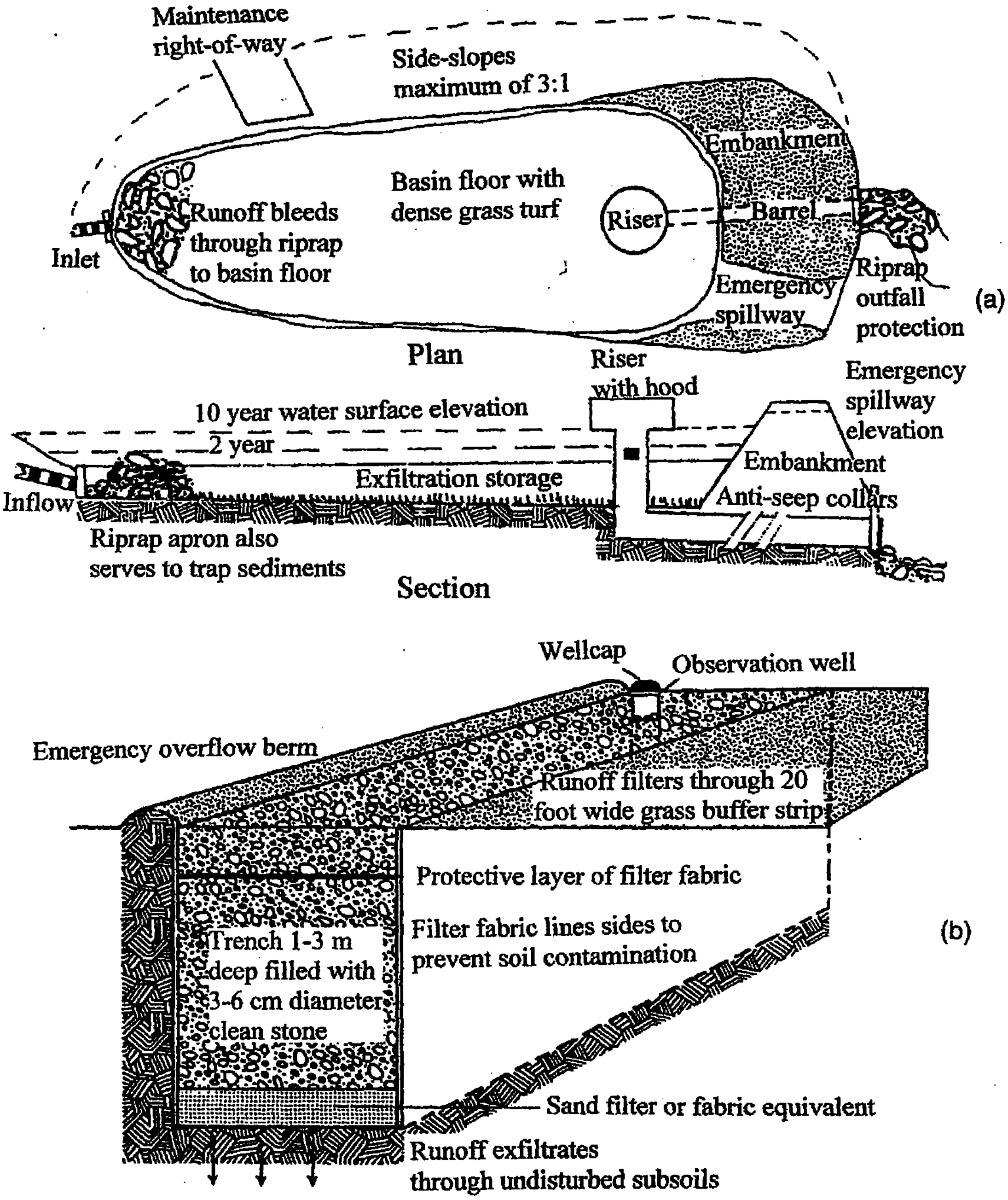


Figure 26-3 Infiltration Facilities for Control of Stormwater Runoff: (a) infiltration basin that has a vertical riser to control the depth of runoff (riser height designed to control 2-year and 10-year storms); (b) basin infiltration trench design.

Continued

26-4 CHARACTERISTICS, IMPACTS, AND MANAGEMENT OF COMBINED SEWER OVERFLOWS

A combined sewer carries both sanitary wastewater and stormwater runoff. The characteristics of wastewater in a combined sewer will vary, depending upon dry and wet weather conditions and characteristics of surface runoff. Its impact upon wastewater treatment facilities and receiving waters may be significant during wet weather conditions.

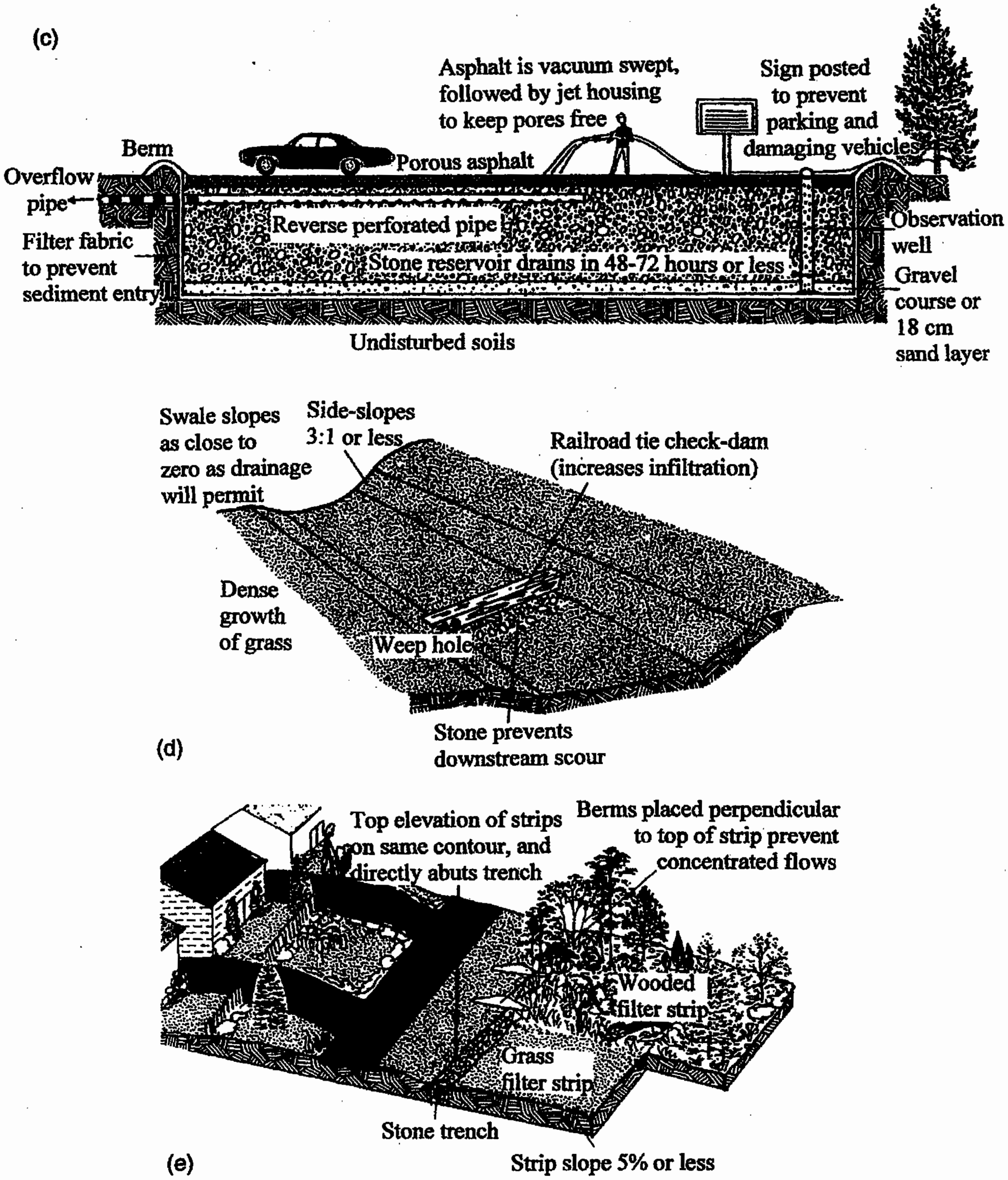


Figure 26-3—cont'd (c) porous pavement design features; (d) schematic of grassed swale; and (e) location and details of filter strip (from Ref. 8).

26-4-1 Characteristics and Impacts of CSOs

Numerous studies have concluded that CSOs are major contributors to water pollution in some municipalities. The amount of toxic priority pollutants found in the influent to POTWs served by combined sewers increases significantly during and immediately following a storm event. Recreational beach closings and shellfish bed closings have been attributed to CSOs. It has been reported that between 10 and 20 percent of harvest-limited shellfish beds may be directly attributable to combined sewer discharges.[5]

Factors Influencing Characteristics of CSOs. The combined sewers receive sanitary sewage during dry weather, and combined sanitary and stormwater flows during wet weather conditions. The ratio of wet-to-dry weather flow may exceed an 8:1 ratio.[26]

Many factors influence the characteristics of combined sewage. Among these are (1) precipitation, intensity, and duration; (2) predominant land use in each catchment: commercial, industrial, residential; (3) subcatchment condition: street-sweeping frequency, buildup of surface contaminant and debris, number of dry weather days before a storm; (4) drainage basin: size, time of concentration, surface characteristics; (5) sewer system: size and slope; and (6) sanitary wastewater contributing sources. Because of the variability of precipitation events and contributing factors, the combined wastewater characteristics tend to be highly variable from location to location and from time to time. Actual measurements and computer simulation is generally needed to characterize the combined flows.

Computer Modeling. As discussed in Sec. 26-3-2, many computer models have been developed to simulate the characteristics of urban stormwater runoff and combined sewer overflows. Of the many computer models available, the most widely used model is the Stormwater Management Model (SWMM). This model can simulate most of the influencing factors indicated above as linear, exponential, and power function equations. Both hydraulic and contaminant routing are performed by SWMM for individual storm events or for long-term multiyear rainfall records. The model keeps track of flows in excess of conduit capacity for different subareas.

As with any model, extensive field data are needed for model calibration. For using SWMM, the calibration and verification is essential. The field data (both flow and contaminants) collected from a storm are compared with estimated results from the model. The estimated inputs are adjusted within the reasonable bounds to obtain the best fit of measured and predicted values. Model verification is done by comparing measured and predicted values without any adjustments of parameters. A comparison of measured combined wastewater flow rates and predicted flow rates from SWMM calculations at a CSO outlet and at a wastewater treatment plant is shown in Figure 26-4.[26]

Quantity and Quality Characterization. The characteristics of combined wastewater are highly variable and difficult to predict. The following generalizations, however, can be made:

1. The wet-to-dry weather capacity ratio for combined interceptor sewer design may range from 1:1 to 8:1, with a median ratio of 4:1.
2. Initially, the concentrations of many contaminants such as TSS, oxygen-demanding wastes, heavy metals, nutrients, toxic organics, and microorganisms are high in stormwater runoff because of surface washoff.
3. The concentrations of these contaminants in combined sewers initially also increase in spite of mixing with raw municipal wastewater. This is often termed the *first flush*

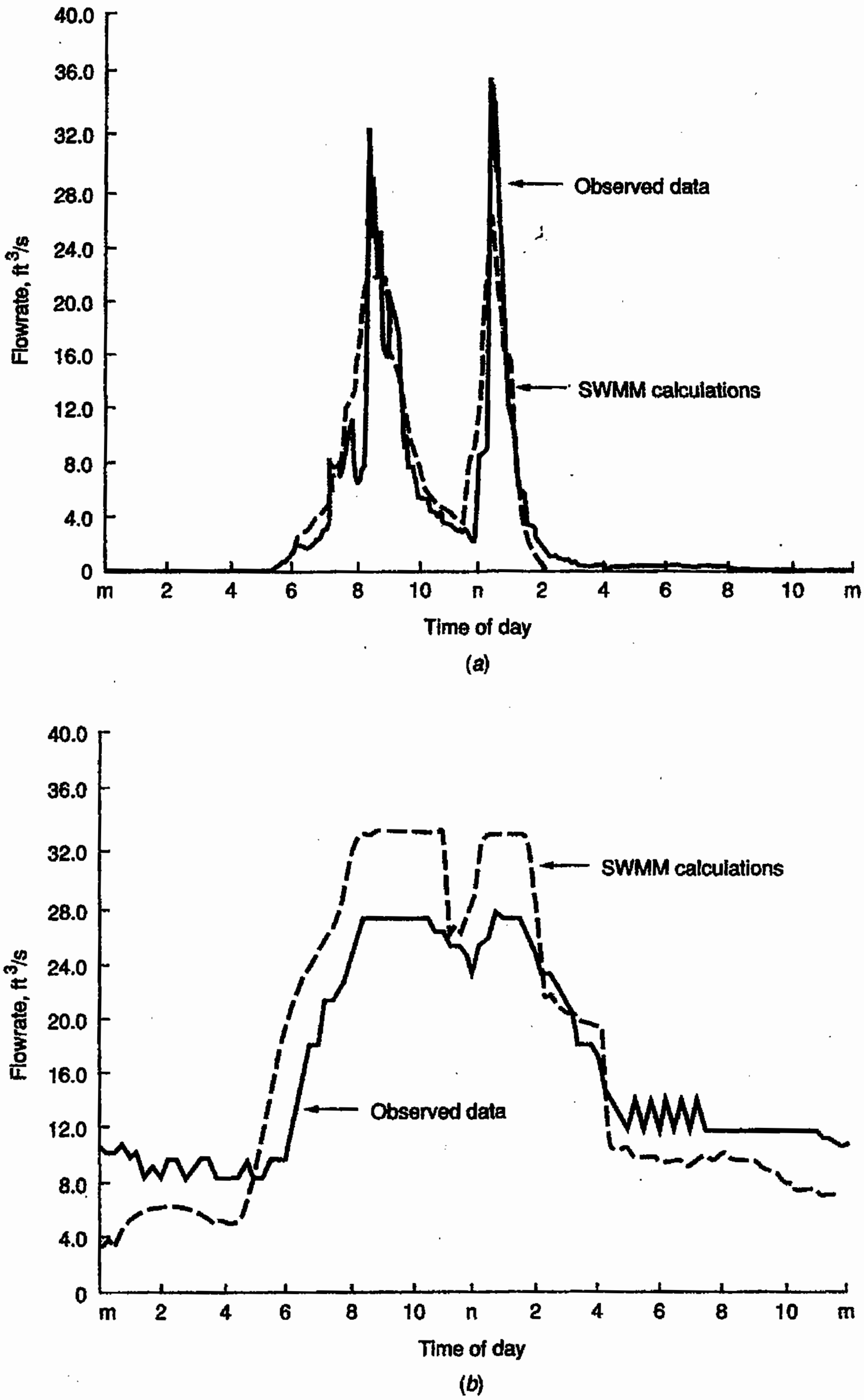


Figure 26-4 Comparison of Measured Wastewater Flow Rates versus Model-Predicted Flow Rates Using SWMM: (a) at CSO outlet and (b) at a wastewater treatment plant (adapted from Ref. 26; used with permission of The McGraw-Hill Companies).

because of resuspension of settled matter in the sewers and manholes caused by excess stormwater flow that already contains high concentrations of contaminants from surface washoff.

4. After the first flush, if flow continues to be high because of long duration or subsequent storm events, the contaminant level drops in spite of the raw municipal wastewater.
5. After the stormwater runoff subsides, the original municipal wastewater characteristics resume.
6. The typical range of concentrations of various constituents reported are TSS = 270–550 mg/L, BOD_5 = 60–220 mg/L, COD = 260–480 mg/L, fecal coliform organisms 2×10^5 to 1.1×10^6/100 mL, total nitrogen = 4–17 mg/L, total phosphorus = 1.2–2.8 mg/L, and lead = 140–600 mg/L.

26-4-2 Flow Regulators

A combined sewer carries both sanitary wastewater and stormwater runoff. Wastewater treatment facilities are hydraulically designed to handle peak dry weather flow plus an infiltration and inflow allowance. Any flow exceeding the design capacity of the plant must be diverted away from the plant.

Overflow and diversion structures divert the excess flow into a companion facility that may be a temporary storage basin, a relief or bypass sewer, or an outfall structure. Examples of overflow and diversion structures are side weir, baffled side weir, transverse weir, leaping weir, orifice, and relief syphon. The flow-regulating devices are used to prevent flooding of downstream facilities, such as surcharging an existing interceptor. Such devices are flow regulators. Examples of such flow regulators are mechanical, tipping plate, and Hydro-Brake regulators. Most of these devices are automatic mechanical regulators, either a float-operated type or a flap-gate type. These are generally impracticable for small flows because of clogging. All such devices may need periodic inspection, skillful adjustment, and careful maintenance. The design details of side weir, leaping weir, and Hydro-Brake or vortex flow-type regulators are shown in Figure 26-5. Readers may consult Refs. 12 and 26–29 for discussions on each device.

26-4-3 Outlet Structure

The outlet structure of a combined sewer is at the end of the sewer. It could be at the outfall of receiving water or where a combined sewer intersects the interceptor sewer. These gates consist of a flap hung against an inclined seat or an elastomeric check valve. Discussion on these outlet structures may be found in Refs. 12 and 26.

26-4-4 Management of Combined Sewer Overflow

Management of combined sewer overflows is important, and much effort has been devoted for assessment of the magnitude of the problem. EPA's estimate of the national CSO correction cost is \$41.2 billion.[1] A variety of CSO management practices has been studied, and some of them have been tested on a limited basis. Broadly, the CSO management practices fall into three major categories: (1) source control, (2) collection system control, and (3) storage and treatment. Each of these management practices is discussed below.

Source Control by Best Management Practices. Source controls for CSOs aim at reducing the stormwater flow, reducing pollution-causing sources, and maintaining aesthetic conditions. Reduction of stormwater flow can be achieved by constructing flow detention basins, porous pavements, infiltration trenches, roof drain storage, and utilization of pervious areas for recharge. All these techniques will retard the runoff or prevent runoff from entering the combined sewers.

Reduction of pollution sources will decrease contaminant loading of storm runoff, thereby decreasing contaminant loading of CSOs. Major sources are air pollutants, solid

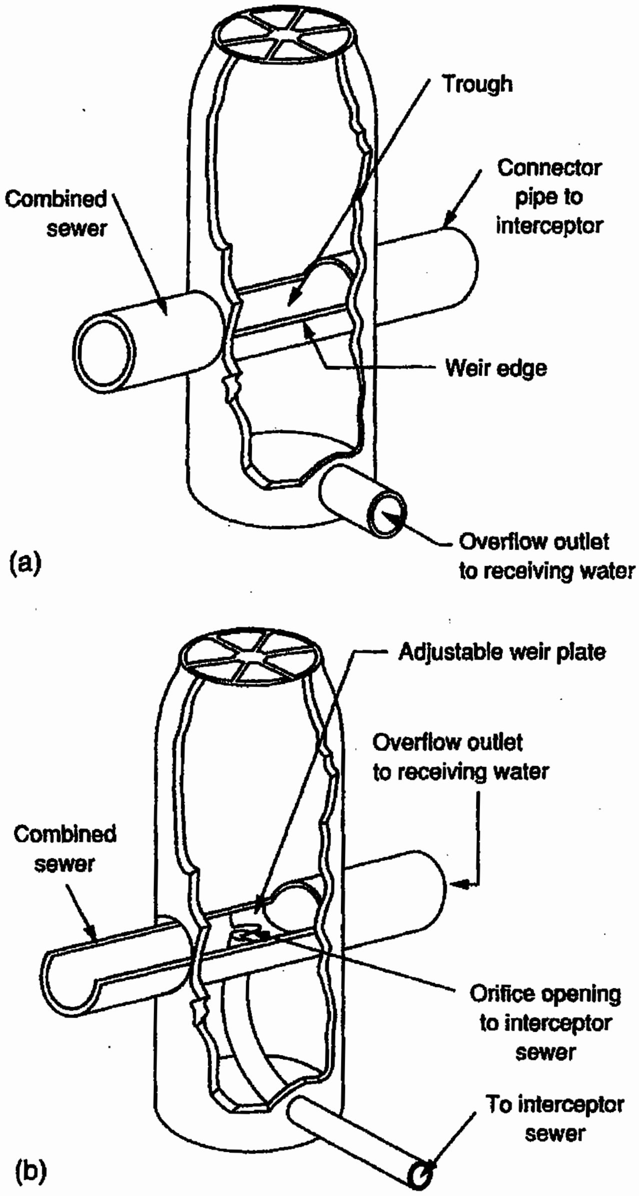
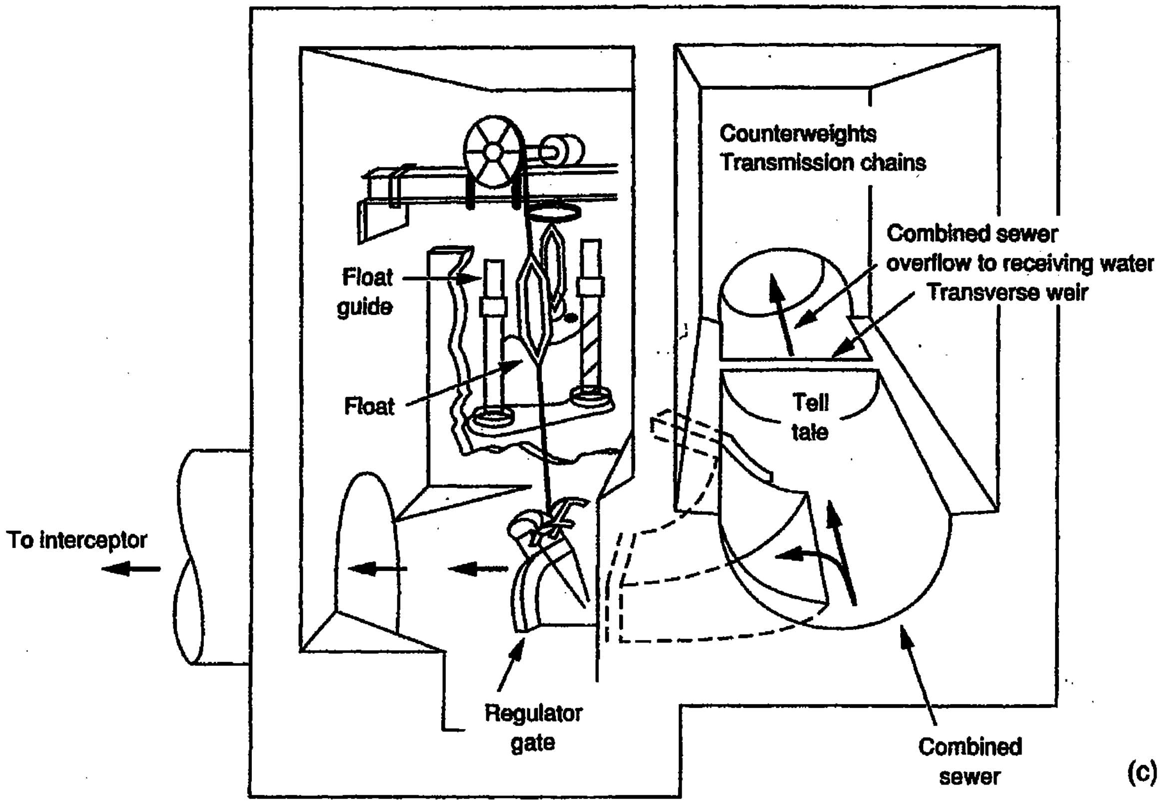
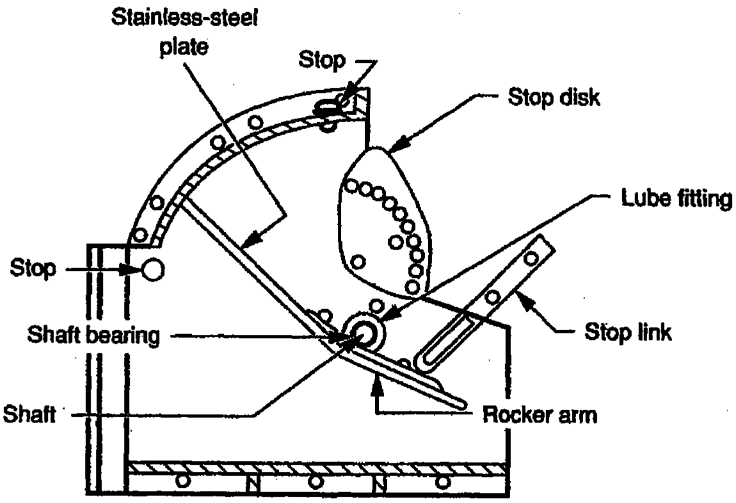


Figure 26-5 CSO Diversion and Regulation Devices: (a) side weir; (b) transverse weir with orifice.





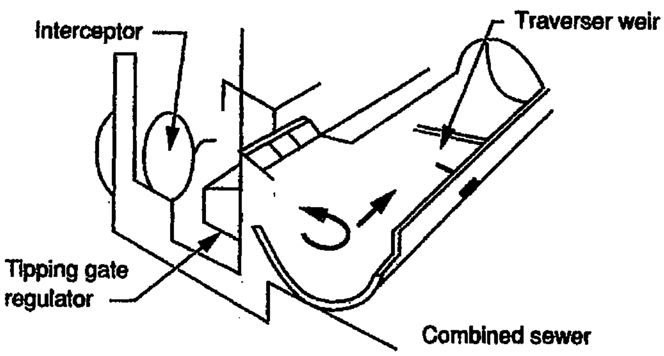
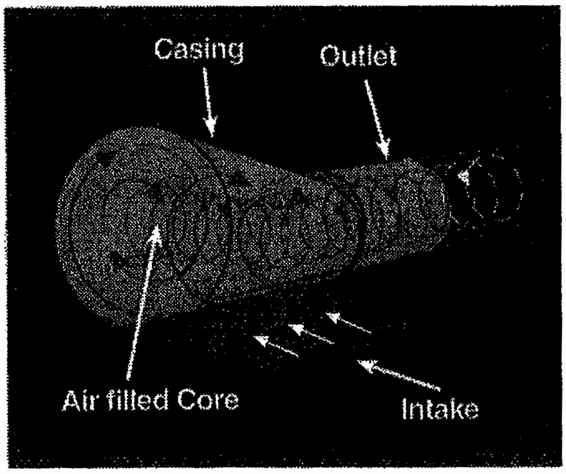
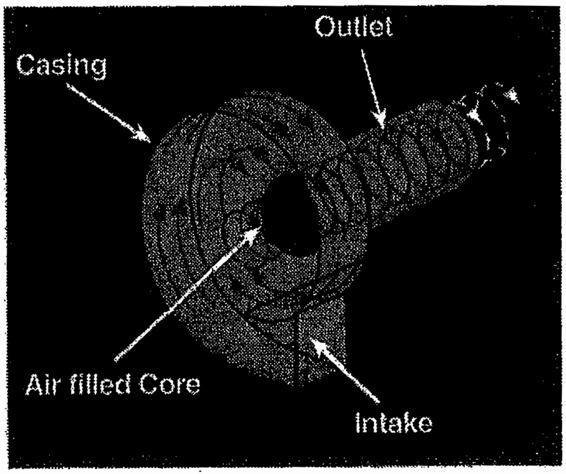


Figure 26-5—cont'd (c) mechanized regulator; (d) tipping plate regulator (from Ref. 26; used with permission of The McGraw-Hill Companies). *Continued*

(e)
(i)




(ii)

Figure 26-5—cont'd (e) Reg-U-Flo.®, two vortex-type regulators (courtesy H.I.L. Technology, Inc.).

wastes and litter, snow removal and deicing agents, fertilizer and pesticides, soil erosion, commercial and industrial runoffs, and animal wastes.

Most of the routine cleaning operations enhance aesthetics and also reduce pollutant loadings. Among these are street sweepings, sewer line flushing, and catchbasin cleaning. The overall improvement in the quality of CSOs may not be very significant but will certainly have a positive impact in the overall context of improving the urban environment.

Collection System Control. Improvements in existing collection facilities can be affected in many ways. Among the common methods are (1) separation of combined sewers into storm and sanitary sewers, (2) infiltration/inflow control, (3) properly designed and operated flow regulation and diversion devices, and (4) modifications in facilities to reduce CSOs. Separation of combined sewers is very expensive. In spite of the cost, this option will not eliminate the impact of stormwater that may result from the discharge of stormwater. The most cost-effective activity under collection system control therefore is improvement in the infiltration/inflow situation to reduce the volume of water that enters the system and must be treated at a POTW. Proper design and operation of flow regulation and diversion facilities will regulate CSOs and will fully utilize the existing capacity of the POTW. Various types of polymers have been used to increase the discharge through the interceptor, relief, and combined sewers that are under capacity and often surcharged. Such systems do enhance the flow capacity but have little effect upon the wastewater quality improvement. Overall quality improvement can only be expected from improved flow management.

Storage and Treatment. Temporary storage of CSOs for later treatment at an existing facility during dry weather conditions is the most cost-effective option because the existing treatment facility is designed to provide full treatment of dry weather flow in addition to the infiltration/inflow allowance. It has the capability to provide full treatment of stored CSO if routed through the plant during dry weather conditions. Additionally, this option has many other benefits: (1) full treatment of CSO and thus no discharge of contaminants associated with stormwater runoff, (2) rapid response to the CSO issue, (3) simplicity because the treatment facility is on-line and fully maintained, and (4) economically feasible because no additional treatment facility is needed. The difficult issue, however, is design and operation of the storage system. The common methods of storage are (1) in-system storage and (2) off-line storage. Both types are briefly described below.

In-System Storage. The in-system storage utilizes the unused capacity in the collection system and is best suited for large-diameter pipes at flat slopes or underground existing tunnels. Inflatable dams or automatic sluice and other types of gates, valves, and regulators (discussed above) are used to capture fully the small storms. All these systems clearly regulate the flow, whereby dry weather flow is diverted into the interceptor while the flows in excess of the interceptor capacity are stored on the downstream side. The inflatable dams provide flexibility of changing the overflow height to increase or decrease the storage to keep a balance with upstream flooding or surcharging and storage needs.

Off-Line Storage. The off-line storage facility is used to store the CSOs for a period long enough to be rerouted through the treatment plant. These storage facilities may be abandoned storage tanks, deep tunnels, pipelines, and channels. In many instances, surface earthen basins and underground tanks are constructed for this purpose. Many exotic inflatable bags, floating pontoons, and flexible curtains have also been built and tried in receiving streams for temporary storage of CSOs.[26,27] Many of these facilities are still in the experimental stages. All these facilities have high operation costs associated with aeration, mixing, dewatering pumps, and general cleanup after dewatering. Surface storage for CSO may raise public health and safety concerns in urban areas.

Instead of building a large centralized storage facility for CSO near the treatment plant, a more practical approach under consideration is to build smaller storage facilities at various strategic locations within the collection systems. Combined flows in excess of the predetermined flow in the interceptor may be diverted to a local storage facility. This flow is stored locally until the surge has passed and the existing wastewater treatment facility is capable of handling the diverted flows. At that time, these stored flows can be routed through the existing interceptors to the operating wastewater treatment facility for complete treatment. It may be noted that SWMM has the capability to keep track of flows in excess of the conduit capacity. The excess flows are hypothetically stored at the inlet until the system has the capability to accommodate these flows.

Treatment Facilities for CSO. As mentioned earlier in this section, the most desirable treatment option for CSO is the existing wastewater treatment facility. This should be the first choice because complete treatment can be provided under dry weather conditions at little or no extra cost when the plant has the excess capacity to handle the rerouted CSO. Only increased grit load in the grit removal facility may be expected. In case such an option is not possible, then a separate treatment facility may be needed. The treatment facilities designed and built just for CSO are often difficult to justify since they will operate only a few times a year. Biological and physical-chemical treatment options have serious limitations. Biological facilities must be operated all year to maintain biological

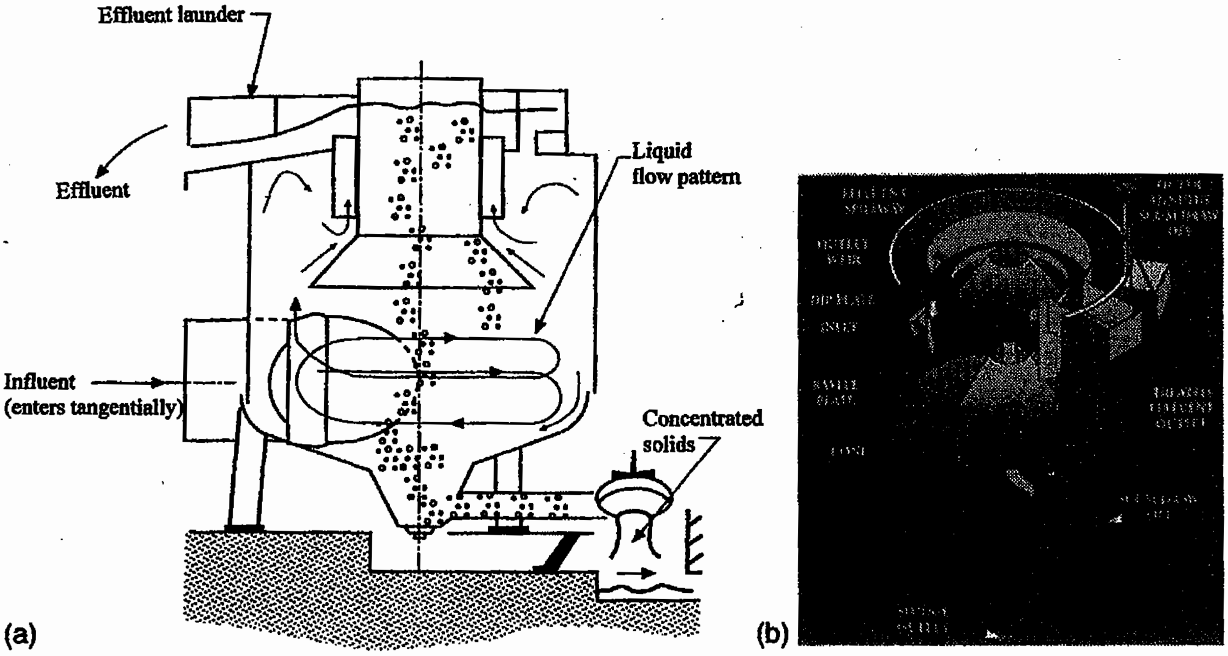


Figure 26-6 Solids Separation Equipment Used for Treatment of Stormwater and CSOs: (a) vortex-type solids separator (from Ref. 29) and (b) Swirl-Flo concentrator/regulator (courtesy H.I.L. Technology, Inc.).

seed. Physical-chemical treatment processes will need large storage facilities for chemical storage and handling and may require some lead-time for chemical mixing and facility operation. In both cases skilled operation is essential.

Physical treatment facilities followed by disinfection have been utilized with some success because they have the flexibility to operate over a wide range of flows. They can also stand idle for extended periods without affecting performance. Among the physical treatment facilities, the swirl concentrator and vortex separators have been tried for removal of settleable solids from CSOs. In Sec. 11-4-3, the operation of a vortex-type grit chamber and gravitational or swirl-flow degritters are presented. The typical vortex-type solids separation device and swirl concentrator/regulator for CSO application are shown in Figure 26-6. In addition to these devices, coarse screens, fine screens, microscreens, and sedimentation facilities have been used with some success. Dissolved-air flotation has also been attempted with little satisfaction.

Disinfection with chlorine, followed by dechlorination, has been used with success. Since handling of pressurized chlorine and sulfur dioxide present safety hazards in urban areas, hypochlorite and sodium metabisulfite or sodium bisulfite have been preferred chemicals for chlorination and dechlorination. Ozone and UV have limited application for CSO disinfection. Ozone generation at the CSO site is difficult because it is needed infrequently. The UV system is often not very effective because of high suspended solids. Readers may refer to Chapter 14 for in-depth coverage on the subject of disinfection.

Goals for CSO Control Measures. As mentioned in Sec. 26-2-2, the final regulations that outline the CSO policy provide municipalities with two approaches showing that the selected CSO control measures will be sufficient to meet the water quality standards.

Under the presumption approach, the municipality can provide a specified level of control that is presumed to meet the water quality standards unless there is data showing otherwise. One of the specified levels of control is assurance that no more than an average of four to six overflow events per year. Under the demonstration approach, the municipality can provide information and data showing that the selected CSO controls actually meet water quality standards. To meet this requirement, the municipalities will continue to utilize the existing technologies and future developments in this field. The use of an existing plant for treatment of CSO will be maximized; this may mean treatment of the entire CSO or options for split treatment. As a result, a portion of CSO will receive full secondary treatment while the remaining may be discharged after primary treatment and followed by disinfection. A great deal of modeling effort will be utilized to route CSOs for local storage for treatment when systems can handle them.

26-5 PROBLEMS AND DISCUSSION TOPICS

26-1 A residential subdivision is developed over a 30-ha land. The current land use is pasture land. The estimated value of C for storms of 5–10 years' return period is 0.15. The uniform rainfall intensity I lasting for a critical period of time of concentration t_c is 65 mm/h. Estimate the runoff from the area.

26-2 After complete residential site development in Problem 26-1, the estimated weighted average value of C is 0.41. Estimate the surface runoff after full development has occurred. The rainfall intensity and land area developed is the same as in Problem 26-1.

26-3 Stormwater runoff can be controlled from newly developing areas. Describe one or more of the control techniques for on-site management in Problem 26-2 to attenuate peak discharges to the predevelopment state.

26-4 Review Ref. 20 and describe basic features of a SWMM computer simulation program to predict stormwater quantity and quality. Indicate how hydraulic and contaminant routing are performed by SWMM for individual storm events and how the flows in excess of conduit capacity are stored at the individual nodes.

26-5 Review Ref. 26 and summarize in tabular form the information about various overflow and diversion structures and flow regulation devices.

26-6 The management techniques for combined sewer overflows are briefly presented in Sec. 26-4-3. Describe the advantages and disadvantages of storage and treatment of CSOs. Why has this option received so much interest in recent years under the NPDES permit application requirement for stormwater discharges and the National Combined Sewer Overflow (CSO) policy?

26-7 A flow-monitoring float was installed in a manhole of an interceptor sewer. The diameter, slope, and n of the interceptor, respectively, are 30 cm, 0.005 m/m, and 0.013. At average flow, the depth in the manhole is 12 cm, and at peak flow the float reads 26 cm. Assume that the depth in the manhole represents the depth of flow in the interceptor. Calculate the peak wet weather and average dry weather flows.

26-8 A combined sewer is 100 cm in diameter. A mechanical regulator is installed that bypasses the excess flow into a diversion sewer. The maximum flow to the treatment plant is reached at a flow depth of 48 cm. As the depth of flow increases above 48 cm, the regulator gate opens in proportion to the depth in the sewer. The gate dimensions are 20 cm wide and 15 cm high. The differential elevation from the center of the gate to the 48-cm depth in the combined sewer is 1 m. Calculate the discharge into the bypass sewer if depth of flow in the combined sewer is 85 cm, the gate opening is 75 percent, and the coefficient of discharge is 0.8.

26-9 In Problem 26-8, the discharge in the bypass sewer is calculated from the gate opening. Calculate the discharge that is bypassed from the combined sewer when the depth in the sewer is 85 cm. The sewer slope and *n* value are 0.0008 m/m and 0.013. Compare the result with that obtained in Problem 26-8.

26-10 A combined sewer is 60 cm in diameter. The slope and *n* values of the sewer are 0.0001 m/m and 0.013. An inflatable dam is provided in the sewer to store excess flow within the system. Under the peak wet weather condition, the maximum depth of flow is 35 cm. If the dam is inflated such that a 55-cm maximum depth of flow is reached in the sewer, normal flow condition is still maintained, and no surcharge occurs in the main sewer and tributaries, calculate the maximum storage obtained per km of the sewer length.

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27

Avoiding Design Errors

27-1 INTRODUCTION

We often hear about the horror stories or the spectacular blunders that designers have made. No matter how humorous or silly these blunders may sound, it should be recognized that somehow they passed through undetected by the project team and the review process. In fact, in the midst of complex details in plans and specifications, little things can be easily overlooked, and any designer can become the victim. As the saying goes, mistakes once committed are often difficult to detect in spite of a regress and careful review of plans and specifications.

The purpose of this chapter is to present many design errors and design deficiencies that have been discovered during construction, operation, and maintenance phases of various projects. Also in this chapter, procedures have been presented that can be implemented by the design and review teams to reduce or eliminate many types of design errors.

27-2 EXAMPLES OF DESIGN ERRORS AND DEFICIENCIES

Some blunders are senseless, and their occurrence is perhaps caused by an oversight or omission while details of the drawings are being worked out. Others may happen because a designer simply forgot to include some items in the design calculations, made a poor judgment or assumption, or perhaps overlooked the implications of the design criteria. Many other poor design provisions may not be errors in the real sense but may be least desirable from the point of view of plant construction or expansion or may not provide the desired flexibility and convenience in plant operation and routine maintenance. Following is a list of many types of the design errors or blunders that have happened in many designs.

(1) The process diagram of a wastewater treatment facility showed the side streams from the sludge-processing area being returned to the primary sedimentation basins. Somehow, the BOD_5 and total suspended solids contributed by these flows were over-

looked in the design of the primary and secondary facilities. As a result these units and the sludge-handling facilities were underdesigned.

(2) In one design the final clarifier was designed without the consideration of the returned sludge. As a result, the actual detention time and overflow rate for the clarifier did not meet the design criteria under the average design flow conditions with recirculation.

(3) In one design of a clarifier, Wright reported that the detailer neglected to show the steel bars from a cantilevered effluent launder hooked and embedded into the basin wall. No one detected this, and when the facility was first put into service, the launder dropped off.[1]

(4) In another settling basin incidence, Wright cited that the designer correctly provided the corbels to support the weight of the effluent launder full of water. This condition could occur when the side gates on the effluent lines were closed and the basin drained. However, the designer inadvertently did not anchor the launder to the corbel against uplift. When the basin was first filled, the launders floated off the support because of the buoyant force.[1]

(5) In one case, Wright reported that a package-pumping station was installed in a flood plane. The station was provided with a submarine-type hatch, and all electrical connections and leads were sealed against flooding. The pumps were controlled through a pneumatic system employing a bubbler tube for level sensing in the wet well. The bubbler tube was not extended above the flood stage. As a result, the station was flooded through the bubbler tube.[1]

(6) Flooding of wastewater treatment facilities is a common occurrence in spite of the flood protection levees. Reported reasons for flooding have included the following: check valves or gate valves were not provided in the stormwater or effluent pipes so the water backed into the plant area; stormwater or effluent pumps were not provided; the storm drainage within the plant site was inadequately designed because of incorrect assumptions of rainfall intensity or runoff coefficient, causing flash flooding; and flood protection levees were not high enough at some sections.

(7) While preparing the unit layout, a design engineer lowered the foundation elevation of one treatment unit to avoid construction above the ground. However, he forgot to make proper changes in the hydraulic profile and relative elevations of the other units. As a result, the free water surface dropped below that of the other units that followed it. Many similar incidences of design blunders can be cited where a change in the design assumption criteria or critical elevation was made, but the resulting effects were not carried throughout the entire design.

(8) While preparing a drawing, an inexperienced draftman can use a wrong symbol. For example, the symbol for a gate valve has two diagonals, while that for a check valve has only one. If the symbols are reversed in a pump drawing, a pump station could be constructed with check valves in the suction line. Wright gave an example of a detailer's error where 24 joists, 16-in. (41-cm) centers, were incorrectly shown as 16 joists, 24-in. (61-cm) centers.[1]

(9) In one design, a chemical feed pump was connected to the chemical solution tank. The tank was not vented to the atmosphere. When the chemical feed pump withdrew the liquid, the tank collapsed because of atmospheric pressure.[1]

(10) Many times the designed units flooded because the designer did not check the hydraulics at peak design flow under emergency conditions when the largest unit was out of service.

(11) One rectangular primary sedimentation basin was constructed with endless conveyor chains and the scraper mechanism running over sprockets. The scraper mechanism was not extended below the effluent launders to the far end wall. As a result, solids accumulated under the effluent structure and created serious odors and an effluent quality problem. The equipment needed redesigned.

(12) Two screen chambers were designed for operation in parallel. Each chamber had gates in the front to stop the flow and remove one chamber from service for routine maintenance. Both channels discharged into a common box that had a common control weir. Stop gates were not provided in the downstream box. As a result, the flow backed up from the effluent side, and neither chamber could be completely drained. Similarly, in many designs, bypass pipes or gates were overlooked, and the unit could not be drained for maintenance purposes.

(13) In one instance, the designer misinterpreted the design criteria or inadvertently oversized the variable-speed pumps at a rate of 150 percent of the peak design flow. Naturally, the plant got flooded whenever the pumps operated at that flow.

(14) Wright reported that, once, a flume was constructed to carry a raw water supply across a coulee. The piers were tall and unstable, but the structure would have been quite stable once the flume was in place and tied to the piers. During the construction phase, however, several piers fell over.[1]

(15) In many designs, the common wall between two units failed when one unit was drained. On many other occasions, the bottom slab heaved upward because of hydrostatic pressure or the sidewall collapsed because of the earth pressure when the units were drained. In all cases, the designer forgot to check the structural safety under the most critical loading conditions.

(16) In one instance, Wright reported that the discharge piping of a pump included a harnessed dresser coupling arrangement. The harnessed connection was to take the hydraulic thrust; therefore, the bolts were designed and specified to be high-tensile bolts. Instead, mild steel harness bolts were installed. When the pumps were tested, the harness bolts failed, resulting in the pump base bending and the anchor bolts failing, with consequent movement of the entire pumping unit away from the header.[1]

(17) Many designs have been prepared with incorrect design criteria or design assumptions. As a result, extensive revisions had to be made later to meet the specific criteria supplied by the concerned regulatory agency. Examples of such criteria are overflow rates, detention time, solids loading, standby units, maximum capacity, chemical dosages, and so on.

(18) In one plant, Sanks reported that several fabricated vertical filters were spaced so closely that the operator had to squeeze sideways to get between them. In another plant, he reported that the control panel required removal of the face plate to replace the desiccant.[2]

(19) In one instance, the suction piping was not properly anchored and secured. Because of vibrations, the opening around the wall cracked open. As a result, the dry pit was flooded because of the high water level in the wet well.

(20) In a chlorinator room, the blower switch and gas masks were located inside the room, which could not be reached in case of an accident.

(21) Many designs have been prepared with little consideration of plant construction and future expansions. Other designs have overlooked the basic operation and maintenance needs of the facility. Common examples of such design deficiencies reported by Wright, Sanks, and others include:[1-4]

1. Not enough space was left for future expansion, or another unit blocked the expansion.
2. A tee of a blind flange was not provided in a pipe at the appropriate location for future connection.
3. Provisions were not made for future expansions of a building.
4. Enough work space was not left around a piece of machinery for servicing or putting a small hoist for lifting a heavy piece.
5. Bolted pipe fittings were casted into walls without enough clearance to install the bolt or put in a wrench.
6. Valves were provided in lines without sufficient clearance for the handwheels or for the operators to reach them.
7. Ceiling hooks were not casted into concrete at appropriate places for a chain hoist.
8. Dimensions of the equipment did not fit the unit size.
9. Floor drains were not provided, or the floor was not properly sloped.
10. A door in the building was too small to bring in the equipment or take it out if the equipment was installed before the structure was completed.
11. Enough space for chemical storage was not provided or chemical feedings equipment was not properly designed.
12. Isolation of chemicals (chlorine feeders, hydrogen peroxide, oxygen, etc.) or isolation of noisy or heat-producing equipment was not considered.
13. Freezing of the pipes, valves, and equipment in cold climates was disregarded, with disastrous consequences.
14. Safety aspects are critical because the bulk chemical storage area offers potential chemical spills. Consideration was not given to items such as a chemical spills or gas leaks. Gas lighter than air (methane, ammonia) travels upward through open stairways or other floor penetrations, and gas heavier than air (chlorine) would disperse horizontally and affect the rooms and tunnels on lower levels.[5]
15. Adequate safety measures to protect not only the plant operators, but also the support personnel working in the administration-operations building were not given. Among these are[5] leak detector, deluge systems, fume scrubbers, minimum distances from storage tanks to buildings, opening in buildings, air intakes, and the like. Most Uniform Fire Codes (UFC) contain provisions relating to such provisions.[6]
16. Electrical conduits, chemical pipes, heating ventilating air conditioning (HVAC), and plumbing may be concentrated at different floors of the buildings. Often, careful planning to avoid conflicts between disciplines in both vertical and horizontal directions was not given.

17. The room layout in the operations building was not properly planned. As a result, the laboratory and restrooms with numerous drains were located directly over electrical and computer rooms. A water leak caused disastrous results.
18. Greater concentration of electrical conduits and chemical piping and plumbing resulted in excessive slab penetrations and openings. Proper allowances for structural safety of the slab were not given.
19. The location of isolation-expansion joints were not coordinated with all disciplines for potential movement and location of embedded piping across isolation joints.

There is a long list of such small design deficiencies. The designer should be conscious of them, and proper considerations should be given to them during early stages of the design.

27-3 PROCEDURE TO AVOID OR REDUCE COMMON DESIGN ERRORS AND DEFICIENCIES

Once an error is made in plans and specifications, there is a good chance that it will escape detection. To prevent such incidents from happening, many design engineering firms have developed a comprehensive design review process. Although such review processes may vary from job to job based on availability of personnel within the organization, there are some basic items that must be considered under all conditions, some of which are listed below:

1. Reviews of the design and checks of the plans and specifications are generally necessary at various stages of the design to eliminate errors and inconsistencies.
2. The preliminary design data of the wastewater treatment facility is developed as part of the facility-planning process (see Chapter 6). This includes design and initial flows, evaluation of alternatives and process selection, detailed process train, and preliminary unit sizing. Many states require justification and approval if major changes in the facility plan are expected. In the author's opinion, the facility plan must be used as a guide during the entire design calculation phase and during preparation of plans and specifications. However, a predesign review of the facility plan should be made by the designers, checkers, project engineer, and project manager. At this review the work plan, responsibilities, time schedules, budgets, and so on should be discussed. Any changes in design data developed in the facility plan (including process train) must be approved by everyone concerned. Afterward, the design data should not be changed.
3. The second review should be made after the design of the individual treatment units is complete. At this review the basic design criteria of the concerned regulatory agency, material mass balance analyses, flows and loadings under normal and emergency conditions (when largest unit is out of service), unit details, and the mechanical equipment for each unit should be checked. Considerations should be given to equipment compatibility, minimum and maximum chemical feed rates, operating pressures, ratings of the pumps, chemical feeders, and so on. Any changes in meeting the nec-

essary space, flexibility requirements, and equipment compatibility and performance should be made at this time.

4. The third review should be made after the layout plan, yard pipings, and hydraulic profile are complete. At this stage the review team should include the design engineer and checker, project manager or principles, and perhaps one or two experienced designers who have not been involved in the preliminary reviews. Small consulting engineering firms may not have enough in-house expertise. In such circumstances a competent consultant may be used to check the design.

This review should be made with a limited number of design items at a time. These items include (1) plant schematics and unit layout, (2) future expansion, (3) hydraulic profile and elevations of the treatment units, (4) space around equipment, (5) flexibility, (6) freezing, and (7) plant operation and maintenance. Each of these items and review considerations is presented below.

1. The plant schematics and unit layout should be thoroughly checked. Special consideration should be given to the following items:
   - forward and return flow lines
   - material mass balance
   - basins and their volumes
   - pumps, piping, valves, gates, bypasses, collection and splitter boxes, etc.
2. Space for future expansion of the plants should be identified on the layout, including future basins, buildings, appurtenances, and the like. Most designers prefer to mark these on the layout plan in dotted lines. This way, any unit or building obstructing future expansion can be easily detected. Pipe connections for future expansion should also be checked. Many designers prefer to provide tees and crosses at appropriate places; the blocked end is used for future connections.
3. Hydraulic calculations and profile should be thoroughly checked. Unit elevations, ground elevations, and water surface elevations at critical flow conditions should be marked on the hydraulic profile sheet. Omissions and errors in the hydraulic profiles can have disastrous consequences.
4. Adequate space around the equipment should be provided to go around, and place small hoist and other tools for equipment maintenance. Headrooms, doors and accesses, ceiling hooks for chain hoists, and the like should be carefully checked.
5. Plant flexibility is important to operate the plant properly under high and low flow conditions. Pipe channels, valves, and structures should be sized for peak design flow when the largest unit is out of operation. The chemical feeders should be sized for extreme conditions. Space for chemical storage and future feeders should be properly planned.
6. Freeze protection of equipment and pipes should be checked if freezing is expected. Channels, pipes, sludge lines, and drains that flow intermittently should have enough earth cover or be designed to drain completely. Moving parts, such as sprockets and chains should be kept below freeze depth in the basin.
7. Operation and maintenance considerations should not be overlooked by the reviewing team. Some designers prefer writing an O&M manual during the design phase to

include the general operational flexibility of the plant.[3] Following is a list of basic operation and maintenance items that should be checked by the review team:

- Arrangements to bypass a unit should be made for routine maintenance.
- Floor drains and proper slope should be provided for draining and flushing the unit.
- Chlorine solution lines should be provided to wash the basin walls, weirs, and channels for routine operational needs.
- Sampling points for each process should be clearly identified in the O&M manual. In large plants, samples from different units are piped directly to the laboratory. This way the representative fresh samples can be obtained by the laboratory staff directly.
- The review team should evaluate the equipment and controls for complexity and maintenance needs. A simple system should always be preferred over a complex system.
- Hand wheels and valves should have operating clearance.
- The operating panels should be accessible, and strip charts, recording pens, and desiccant containers should be easily replaceable.
- Hazardous chemicals (chlorine, ozone, etc.) should be isolated. Noisy and heat-producing equipment should also be isolated. Arrangement should be made for dry feeder equipment for easy filling. All storage tanks must have secondary containment.
- Proper clearance between bulk storage chemicals and administration and control building and air intakes and scrubbers must be made. UFC[6] must be followed throughout the checking process.
- All stairs and walkways should have proper handrails.
- Laboratory and workshops should have adequate space, equipment and tools, lavatories, and showers.

As mentioned earlier, each review process may vary from project to project and the availability of resources of the design firm. Each firm should develop the review procedure that best fits its resources and the needs of the project. The review process given above should be used only as a guide since it is neither complete nor definitive.

27-4 DO'S AND DON'TS DESIGN CHECKLIST

Many design engineers have developed a comprehensive checklist of Do's and Don'ts, which is followed carefully as a routine check after completion of the preliminary design plans and specifications. A few items of such a checklist are provided in Table 27-1. It is a very worthwhile effort, and often, many design deficiencies are discovered under such review.

27-5 DISCUSSION TOPICS

27-1 Information checklists have been presented in Chapters 7–19 for the design of various treatment processes. These checklists were highly specialized for individual processes. Using this information, develop a generalized design checklist for an entire wastewater

TABLE 27-1 Design Checklist for Review of Design Plans and Specifications

Do's	Don'ts
General	
1. Verify that the design of all electrical equipment structural pads, foundations, etc., have been included on the plans.	1. Do not use a new product, new idea, or new design on an experimental basis without prior approval.
2. Provide proper submergence conditions on inlet of all conduits. Submergence >1/2 conduit diameter.	2. Do not use sonic meters on liquids with air entrainment or solid concentration over 1.0 percent.
3. Doppler meter can be used only for fluids with entrained solids or air bubbles.	3. Do not locate access ladders on walls with solar or decorator blocks; there is no place to attach supports.
4. Send drawings to appropriate regulatory agencies.	
5. Update cost estimates on a regular basis. Discuss the cost factors at quality control meetings.	
6. Determine how many contractors plan to bid. Make calls to contractors with enough time to issue an addendum if one is needed.	
7. Check to see how each proposed structure will effect the surrounding structures during and after construction.	
8. Check thoroughly for mechanical interference. Do not put something where space is not available.	
9. Obtain permit approval for wastewater treatment plant improvements prior to advertising for construction.	
Process Units	
1. If water elevation has less than a 2-ft drop between treatment units, carefully check hydraulic calculations.	1. Do not design vertical drop pipe out of the bottom of an outlet box.
Pumping	
1. Provide isolation and check valves on pump discharge.	1. Do not try to split flows from pumps using throttling control valves.
2. Provide proper ventilation for all pump stations, wetwells, clearwells, and transfer wells.	
3. For pump station design, use Hydraulic Institute Standards.	

TABLE 27-1 Design Checklist for Review of Design Plans and Specifications—cont'd

Do's	Don'ts
Aeration	
1. Use and specify high-temperature gaskets for all air piping.	1. Do not use fine diffusers without air filter.
2. Ductile iron air pipe should be unlined.	
Chemical Feed	
1. Use black steel pipe for pressure ammonia and chlorine applications.	1. Do not use fiberglass for HFS chemical storage tank.
2. Use Schedule 80 PVC pipe for vacuum and chlorine applications.	2. Do not use rubber gaskets for ammonia application.
3. Provide heat tracing and insulation for all exposed caustic lines and storage tanks.	
4. Use cross-linked polyethylene material for a hydrofluosilicic acid (HFS) storage tank.	
5. Provide concrete coating protection for HFS feed and storage areas.	
6. Verify material compatibility of equipment, tanks, pipes, gaskets, etc. with the material or chemical being stored, pumped, or piped.	
7. Provide explosion-proof equipment for ammonia feed area.	
Sludge Handling	
1. Use minimum 15-cm (6-in.) piping for all sludge applications.	1. Do not use propeller or turbine meters for measuring raw sewage, sludge, or other liquid that could contain rags or other debris.
2. Use plug valves on sludge application lines.	2. Do not use common sludge piping for sludge withdraw from two or more clarifiers operating in parallel.
Yard Piping	
1. Buried steel pipe should have cathodic (corrosion) protection.	1. Do not use PVC piping on any type of heated fluid.
2. Provide freeze protection for all exposed piping under 30 cm.	
3. Provide adequate support for all aboveground piping.	

treatment plant that a designer must use as a guide during the entire design phase of a project.

27-2 Develop a comprehensive checklist for design review that must be used by a review team. Arrange the checklist items under the following headings:
(a) facility plan information that must be carried out in the plans and specifications
(b) process train and instrumentation
(c) plant layout and piping
(d) buildings and equipment
(e) plant hydraulics
(f) roads, drainage, and landscaping
(g) operation and maintenance flexibility
(h) expansion flexibility

27-3 You have been hired as an environmental engineer by a small consulting engineering company to complete the final plans and specifications of a medium-sized wastewater treatment plant. There is one additional civil environmental engineer in your organization. The facility plan for the project was prepared by another engineer who is no longer with this organization. Discuss how you would organize the project components, including revisions to the facility plan, design tasks and review process, review team, and project time table. The project must be completed in 18 months. The project involves upgrading of the existing primary treatment facility and pumping station and adding a new activated sludge treatment facility.

27-4 Complete Problem 27-3 assuming that a large consulting engineering company has hired you as a junior design engineer to assist a project manager who prepared the facility plan.

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A

Physical and Chemical Properties of Water

The principal physical and chemical properties of water commonly used in wastewater treatment plant design are summarized in Tables A-1, A-2, and A-3 in this appendix. Most of the information is developed from Refs. 1–5.

TABLE A-1 Properties of Water

Molecular formula	= H_2O
Molecular weight	= 18
Ionization constant (K_w) at 25°C	= 10^{-14}
Specific weight, γ at 4°C	= 9.81 kN/m^3 (62.43 lb$_f$/ft^3)
1 U.S. gal weighs (average)	= 8.34 lb
Density at 4°C	= 1 g/cm^3 (1.94 slug/ft^3)
Specific heat	= 1 cal/g°C (1 Btu/lb°F)
Dynamic viscosity, μ	= 1×10^{-3} N·s/m^2 (2.089×10^{-5} lb$_f$·s/ft^2
Kinematic viscosity, ν	= 1.002×10^{-6} m^2/s (1.078×10^{-5} ft^2/s)
One atm (760 mm Hg or 14.7 psi)	= 10.33 m (33.9 ft) water
Boiling point at 1 atm	= 100°C (212°F)
Melting point at 1 atm	= 0°C (32°F)
Heat of fusion at 0°C	= 80 cal/g (144 Btu/lb)
Heat of vaporization at 100°C	= 540 cal/g (973 Btu/lb)
Modulus of elasticity at 15°C	= 2.15 kN/m^2 (312 × 10^3 lb$_f$/in^2)

Temperature T		Density	Dynamic Viscosity	Kinematic Viscosity	Surface Tension Against Air	Vapor Pressure
		ρ	$\mu \times 10^3$	$\nu \times 10^6$	$\sigma \times 10^3$	p
°C	°F	(g/cm^3)	(N·s/m^2)	(m^2/s)	(N/m)	(mm Hg)
0	32	0.9998	1.787	1.787	75.60	4.579
3.98	39.2	1.0000	1.568	1.568	75.00	6.092
5	41	0.9999	1.519	1.519	74.90	6.543
10	50	0.9997	1.307	1.307	74.22	9.209
15	59	0.9991	1.139	1.140	73.49	12.788
20	68	0.9982	1.002	1.004	72.75	17.535
25	77	0.9970	0.890	0.893	71.97	23.756
30	86	0.9957	0.798	0.801	71.18	31.824
35	95	0.9941	0.719	0.724	70.37	42.175
40	104	0.9922	0.653	0.658	69.56	55.324
45	113	0.9903	0.596	0.602	68.74	71.880
50	122	0.9881	0.547	0.553	67.91	92.510

TABLE A-2 Solubility of Oxygen and Other Gases in Water

Temperature T		Dissolved Oxygen Saturation[a] C_s (mg/L)						Solubility of Other Gases (mg/L)		
		Chloride Concentration (mg/L)					Decrease in Oxygen Concentration per 100 mg Chloride			
°C	°F	0	5000	10,000	15,000	20,000		Nitrogen	Carbon Dioxide	Air
0	32	14.62	13.79	12.97	12.14	11.32	0.017	22.81	1.00	37.50
5	41	12.80	12.09	11.39	10.70	10.01	0.014	20.16	0.83	32.94
10	50	11.33	10.73	10.13	9.55	8.98	0.012	17.93	0.70	29.27
15	59	10.15	9.65	9.14	8.63	8.14	0.010	16.15	0.59	26.25
20	68	9.17	8.73	8.30	7.86	7.42	0.009	14.72	0.51	23.74
25	77	8.38	7.96	7.56	7.15	6.74	0.008	13.55	0.43	21.58
30	86	7.63	7.25	6.86	6.49	6.13	0.008	12.53	0.38	19.60

[a]The solubility of oxygen (C_s) exposed to air containing 20.9 percent oxygen by volume at 1 atmosphere (760 mm Hg). Under any other barometric pressure, the solubility can be calculated from the equation

$$C_s' = \frac{C_s(P - p)}{(760 - p)}$$

where p = vapor pressure at the temperature of water, P = atmospheric pressure, mm Hg.

TABLE A-3 Barometric Pressure with Altitude

Elevation above Sea Level		Absolute Pressure in Head of Water (H_{abso})	
m	ft	m	ft
0	0	10.3	33.9
305	1000	10.0	32.8
457	1500	9.8	32.1
610	2000	9.6	31.5
1219	4000	8.9	29.2
1829	6000	8.3	27.2
2438	8000	7.7	25.2
3048	10,000	7.1	23.4
4572	15,000	5.9	19.2

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B

Head Loss Constants in Open Channels and Pressure Pipes

In this appendix the constants commonly used to calculate minor head losses in open channels and pressure conduits are given. Many of these constants have been used in the Design Example to calculate the losses caused by contraction and expansion and exit and entrance in the appurtenances. The values of most of these constants may be obtained in many hydraulics texts or handbooks. The readers are referred to the references cited at the end of this appendix.

A procedure for computing the depth of flow in a flume, effluent trough, or launder receiving flow from a free-falling weir [Eq. (11-12)] is also given in Sec. 11-10-2, Step E,6. Discussion on this subject may be found in Chapters 11–13.

B-1 MINOR LOSSES IN OPEN CHANNELS

B-1-1 Sudden Contraction or Inlet Losses

Sharp-cornered entrance	0.5	$\left(\dfrac{v_1^2}{2g} - \dfrac{v_2^2}{2g}\right)$
Round-cornered entrance	0.25	$\left(\dfrac{v_1^2}{2g} - \dfrac{v_2^2}{2g}\right)$

Bell-mouthed entrance	0.05	$\left(\frac{v_1^2}{2g} - \frac{v_2^2}{2g}\right)$

where v_1 and v_2 are velocities downstream and upstream of the contraction.

B-1-2 Sudden Enlargement or Outlet Losses

Sharp-cornered outlet	0.2–1.0	$\left(\frac{v_2^2}{2g} - \frac{v_1^2}{2g}\right)$
Bell-mouthed outlet	0.1	$\left(\frac{v_2^2}{2g} - \frac{v_1^2}{2g}\right)$

where v_1 and v_2 are velocities downstream and upstream of the enlargement.

B-1-3 Syphons

Head loss	2.78	$\frac{v^2}{2g}$

B-1-4 Manhole, Junction, and Division Boxes

Head loss in manhole, junction, and division boxes depends on the size of the appurtenances, change in direction, and the contour of the bottom.

In a straight-through manhole where there is no change in pipe size:

Manhole losses	0.05	$\frac{v^2}{2g}$
Terminal manhole	1	$\frac{v^2}{2g}$

In large junction-division boxes in which the velocity is small:

Exit loss	1.0	$\frac{v^2}{2g}$
Entrance loss	0.5	$\frac{v^2}{2g}$

If sewer size changes or change in direction occurs in a manhole or junction box:

45° no shaping	0.4	$\dfrac{v^2}{2g}$
45° with shaping	0.3	$\dfrac{v^2}{2g}$
90° no shaping	1.3	$\dfrac{v^2}{2g}$
90° with shaping	1.0	$\dfrac{v^2}{2g}$

B-2 MINOR LOSSES IN PRESSURE CONDUITS

B-2-1 Gate Valve

Fully open	0.19	$\dfrac{V^2}{2g}$
One-fourth closed	1.15	$\dfrac{V^2}{2g}$
One-half closed	5.6	$\dfrac{V^2}{2g}$
Three-fourths closed	24.0	$\dfrac{V^2}{2g}$

B-2-2 Butterfly Valve

Fully open	0.3	$\dfrac{V^2}{2g}$
Angle closed, 20°	1.4	$\dfrac{V^2}{2g}$
Angle closed, 40°	10	$\dfrac{V^2}{2g}$
Angle closed, 60°	94	$\dfrac{V^2}{2g}$

B-2-3 Check Valve

Swing check	(0.6–2.3)	$\frac{V^2}{2g}$
Swing check fully open	2.5	$\frac{V^2}{2g}$

B-2-4 Sluice Gate

Fully open	(0.2–0.8)	$\frac{V^2}{2g}$

B-2-5 Plug Globe Valve

Fully open	4.0	$\frac{V^2}{2g}$

B-2-6 Entrance

Pipe projecting into tank	0.83	$\frac{V^2}{2g}$
End of pipe flushed with tank	0.50	$\frac{V^2}{2g}$
Slightly rounded	0.23	$\frac{V^2}{2g}$
Bell-mouthed	0.04	$\frac{V^2}{2g}$

B-2-7 Exit

From pipe into still water	1.0	$\frac{V^2}{2g}$

B-2-8 Venturi Meter

Angle of divergence 5°	(0.10–0.15)	$\frac{V^2}{2g}$ (throat velocity)

Angle of divergence 15°	(0.06–0.33)	$\frac{V^2}{2g}$ (throat velocity)

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C

Treatment Plant Cost Equations

C-1 COST BASIS

The generalized construction, as well as operation and maintenance cost equations for primary and secondary treatment unit operations and processes, are given in this appendix. Most of these cost equations are derived from the EPA's *Areawide Assessment Procedures Manual: Performance and Cost.*[1] Other equations are developed from the manufacturers' information and other sources. The cost estimates based on these equations are valid only for comparison and evaluation of alternatives in the facility plans. There may be a considerable difference between cost estimates for comparative purposes and the actual construction costs of the facility that are developed after completion of plans and specifications.

All construction costs in Ref. 1 are indexed to September 1976 (ENR Index = 2475). The cost equations presented in this appendix have been updated to May 1996 dollar (ENR Index = 5572). To adjust for other time periods, appropriate cost indexes, such as those in the following list, should be used when appropriate:

- ENR Building Cost Index—appears weekly in *Engineering News Record,* McGraw-Hill
- ENR Construction Cost Index—appears weekly in *Engineering News Record,* McGraw-Hill
- BLS Labor Cost/Index—appears monthly in *Employment and Earnings,* Bureau of Labor Statistics
- BLS Wholesale Price Index—appears monthly in *Wholesale Prices and Price Indexes,* Bureau of Labor Statistics

Table C-1 provides the reader with the generalized basis of all cost curves originally developed by EPA in Ref. 1. The construction costs do not include piping, electrical, instrumentation, site preparation, engineering and construction, supervision, and contingencies. These costs are added as a percentage of installed construction costs of the com-

TABLE C-1 *General Cost and Design Basis for Cost Curves in Ref. 1*

Basis of Costs

1. ENR = 2475, September 1976
2. Labor rate, including fringe benefits = \$7.50/h *Note:* Labor costs are based on a man-year of 1500 h. This represents a 5-day work week; an average of 29 days for holidays, vacations, and sick leave; and $6\frac{1}{2}$ hours of productive work time per day.
3. Energy costs
   a. Electric power = \$0.02/kW·h
   b. Fuel oil = \$0.37/gal
   c. Gasoline = \$0.60/gal
4. Land = \$1000/acre
5. Chemical costs
   a. Chlorine 150-lb cylinder = \$360/ton
      1-ton cylinder = \$260/ton
      tank car = \$160/ton
   b. Quicklime = \$25/ton
   c. Hydrated lime = \$30/ton (expressed as CaO)
   d. Polymer (dry) = \$1.50/lb
   e. Ferric chloride = \$100/ton
   f. Alum = \$72/ton
   g. Sulfuric acid = \$50/ton

Design Basis

1. Construction costs, and operation and maintenance costs are based on design average flow unless otherwise noted.
2. Operation and maintenance costs include
   a. Labor costs for operation, preventive maintenance, and minor repairs
   b. Materials costs include replacement parts and major repair work (normally performed by outside contractors).
   c. Chemical costs
   d. Fuel costs
   e. Electrical power costs
3. Construction costs do not include land costs (except for landfilling), external piping, electrical, instrumentation, site work, contingency, engineering and construction supervision, or miscellaneous structures.

TABLE C-2 Development of Capital Costs

	Avg.	Range[b]		
Total capital cost of unit processes specific to each cost estimate			$ ______	
Misc. structures Eq. (C-53)[a]			$ ______	
Subtotal 1				$ ______
Piping	10%	8–15%	$ ______	
Electrical	8%	5–12%	$ ______	
Instrumentation	5%	3–10%	$ ______	
Site preparation	5%	1–10%	$ ______	
Subtotal 2				$ ______
Engineering and construction supervision @ 15%[c]			$ ______	
Contingencies % 15%[c]			$ ______	
Total capital cost[d]				$ ______

[a]Miscellaneous structures include administrative offices, laboratories, shops, and garage facilities.
[b]Range because of level of complexity, degree of instrumentation, subsoil conditions, configuration of site, etc.
[c]Percentage of subtotal 2.
[d]See Tables 6-11 and 6-12 for calculation procedure.

ponents specific to the unit or system being evaluated for comparative cost estimates. Table C-2 represents a general format for identifing the costs, with representative percentages for the items indicated.

C-2 COST EQUATIONS

A total of 55 equations for capital and O&M costs for different processes are provided in this appendix. The variables used in these equations are:

- CC = capital costs, dollars
- O&M = annual operation and maintenance cost, $/year
- Q = average design flow through the facility, m^3/d

Each cost equation given in this appendix includes source, a summary of the design basis, assumptions, and the cost basis. Generalized adjustment factors are also provided to modify the cost data for any desired solids and hydraulic loadings, detention times, solids concentrations, periods of operation, and the like.

The cost equations presented in this appendix include the following processes:

1. Gravity sewers, Eqs. (C-1) and (C-2)
2. Low-lift pump station, Eqs. (C-3) and (C-4)
3. Preliminary treatment, with and without bar screens, Eqs. (C-5)–(C-8)
4. Primary sedimentation with sludge pumps, Eqs. (C-9) and (C-10)

5. Mineral addition (ferric chloride), Eqs. (C-11) and (C-12)
6. Conventional activated sludge aeration with diffused air, Eqs. (C-13) and (C-14)
7. Activated sludge with nitrification in single stage, Eqs. (C-15) and (C-16)
8. Final clarifier (flocculator-type) used with aeration basin, Eqs. (C-17) and (C-18)
9. High-rate trickling filter, Eqs. (C-19) and (C-20)
10. Clarifier for high-rate trickling filter (includes recycle pumps), Eqs. (C-21) and (C-22)
11. Separate nitrification with clarifier, Eqs. (C-23) and (C-24)
12. Separate denitrification with clarifier, Eqs. (C-25) and (C-26)
13. Gravity filtration (dual media), Eqs. (C-27) and (C-28)
14. Activated carbon absorption, Eqs. (C-29) and (C-30)
15. Chlorination (disinfection), Eqs. (C-31) and (C-32)
16. Dechlorination using sulfur dioxide, Eqs. (C-33) and (C-34)
17. UV disinfection, Eqs. (C-35) and (C-36)
18. Sludge pumping, Eqs. (C-37) and (C-38)
19. Gravity thickener, Eqs. (C-39) and (C-40)
20. Aerobic digester, Eqs. (C-41) and (C-42)
21. Two-stage anaerobic digestors, Eqs. (C-43) and (C-44)
22. Sludge-drying beds, Eqs. (C-45) and (C-46)
23. Filter press or belt filter, Eqs. (C-47) and (C-48)
24. Landfilling (biological sludge, excluding transportation), Eqs. (C-49) and (C-50)
25. Biosolids utilization, Eqs. (C-51) and (C-52)
26. Miscellaneous structures, Eqs. (C-53) and (C-54)
27. Support personnel, Eq. (C-55)

C-3 PROCEDURE FOR DEVELOPING COST EQUATIONS

The capital and annual O&M cost equations for different processes were developed using Microsoft EXCEL Spread Sheet. The given information was graphed in an *XY* format, with flows on the *X*-axis and the capital or O&M costs on the *Y*-axis. Using EXCEL's Trendline feature, a trendline was added to the graph. This feature includes an option for the determination of the trendline function equation and R^2. Several functions were tried. These include logarithmic, exponential, linear, polynomial, and more. The function that gave the R^2 value closest to 1 was selected and the equation was obtained. This equation was used to calculate the capital and O&M costs for each of the flows compared with inputted values to assure that values are within the acceptable range. The applicable range of these equations are in the flow range of 3800 m^3/d (1 mgd) to 190,000 m^3/d (50 mgd).

For the cost estimates of many other unit operations and processes, such as chemical coagulation, nitrification-denitrification, carbon adsorption, ammonia stripping, ion exchange, membrane processes, and so on, the reader should consult the EPA *Areawide Assessment Procedures Manual: Performance and Cost.*[1] Many other sources listed in the references can also be used for similar cost curves.[2-5] Several computer programs for obtaining costs and performance data for various treatment units are also available in the literature.[6-9]

1. *Gravity sewer*

$$CC = 4{,}512\ Q^{0.568} \tag{C-1}$$

$$\text{O\&M} = 97\ Q^{0.301} \tag{C-2}$$

Service life = 50 years. Peaking factors 3 for average flow less than 43.8 L/s (1 mgd) and 2 for average flows greater than 438 L/s (10 mgd). Slope 0.002 to 0.005. Velocity not less than 0.61 mps (2 fps) when flowing full at peak flow. Minimum sewer size = 20 cm (8 in) costs include installation and repairing but do not include right-of-way or aerial crossings, etc.[1]

2. *Low-lift Pump Station*

$$CC = -0.00004\ Q^2 + 34\ Q + 316{,}261 \tag{C-3}$$

$$\text{O\&M} = 0.80\ Q + 18{,}526 \tag{C-4}$$

Service life = 15 years. Construction costs include fully enclosed wet well/dry well structure with piping and valves within structure, peaking factor = 2 Q with largest unit out of service, standby pump, and mechanical bar screens. Package pumping stations used for lower flows. Equations are developed for TDH = 3.05 M (10 ft), operating costs for TDH other than 3.05 m are obtained by using an O&M equation with effective flow Q_E[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{\text{TDH}_{\text{DESIGN}}}{\text{TDH(3.05 m or 10 ft)}}$$

3. *Preliminary Treatment* (Screening plus Grit Removal)[1]

a. With bar screen

$$CC = 674\ Q^{0.611} \tag{C-5}$$

$$\text{O\&M} = 0.96\ Q + 25{,}038 \tag{C-6}$$

b. Without bar screen

$$CC = 531\ Q^{0.616} \tag{C-7}$$

$$\text{O\&M} = 0.96\ Q + 25{,}038 \tag{C-8}$$

Service life = 30 years. Construction costs include flow channels and superstructures, mechanical bar screens, grinders for screenings, gravity grit chamber with mechanical grit handling equipment, Parshall flume, and flow-recording equipment. If low-lift pumping is used prior to preliminary treatment, the equations without bar screens may be used. The suspended matter larger than 1.5 cm (5/8 in.) and grit coarser than 0.208 mm (65 mesh) will be removed. Removal of BOD_5 and TSS by pretreatment is assumed negligible.[1]

4. *Primary Sedimentation with Sludge Pumps*[1]

$$CC = -0.00002\,Q^2 + 19.29\,Q + 220{,}389 \tag{C-9}$$

$$\text{O\&M} = 1.69\,Q + 11{,}376 \tag{C-10}$$

Service life = 50 years. The clarifier is designed for an overflow rate of 32.6 $m^3/m^2 \cdot d$ (800 gpd/ft^2) at average Q. To adjust costs for alternative surface overflow rate, use CC equation with effective flow Q_E

$$Q_E = Q_{\text{DESIGN}} \times \frac{32.6\ m^3/m^2\cdot d}{\text{actual design surface overflow rate}}$$

The costs include primary sludge pumps pumping sludge at solids concentration of 4 percent to an assumed head of 3.05 m (10 ft).[1]

5. *Mineral Addition (Ferric Chloride)*

$$CC = 0.000002\,Q^2 + 3.6\,Q + 44{,}624 \tag{C-11}$$

$$\text{O\&M} = 9.68\,Q + 22{,}392 \tag{C-12}$$

Service life = 20 years. The rapid-mix tank is constructed of concrete with stainless steel mixer. Multiple basins are provided for volumes greater than 42.6 m^3 (1500 ft^3). Cost includes liquid containing 35 percent $FeCl_3$ and dosage of 100 mg/L ($FeCl_3$) chemical storage for 15 days. Price of building is included except for plants with small flows. O&M cost adjustments for other $FeCl_3$ dosages can be made by using O&M equation with effective flow Q_E[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{FeCl_3\ \text{dose}}{100\ \text{mg/L}}$$

6. *Conventional Activated Sludge with Diffused Air*

$$CC = 72\,Q + 368{,}043 \tag{C-13}$$

$$\text{O\&M} = 4.58\,Q + 36{,}295 \tag{C-14}$$

Service life = 40 years. Construction costs include costs of basins, air supply equipment and piping, and blower building. Clarifiers and return sludge pumps are not included. Diffused aeration equipment supplies 1.2 g O_2 per g of BOD_5 removed. The MLVSS concentration = 2,000 mg/L, aeration period 6 h, and $F/M < 0.5\ d^{-1}$.

7. *Activated Sludge with Nitrification in Single Stage*

$$CC = 90\,Q + 612{,}777 \tag{C-15}$$

$$\text{O\&M} = 93\,Q^{0.834} \tag{C-16}$$

Service life = 40 years. Construction costs include aeration tanks and aeration devices. Clarifiers and return sludge pumps are not included. The aeration basin is plug flow, and nitrification takes place in the later portions of the basin. Oxygen requirement is 1.5 g O_2 per g of BOD_5 removed plus 4.6 g O_2 per gram of NH_3-N oxidized.

MLVSS is greater than 2000 mg/L, and mean cell residence time = 15 days.[1] $F/M < 0.3\ d^{-1}$.

8. *Final Clarifier, Used with Aeration Basin*

$$CC = 2941\ Q^{0.609} \tag{C-17}$$

$$\text{O\&M} = 3.32\ Q + 5{,}842 \tag{C-18}$$

Service life = 40 years. The clarifier is a flocculator type having a design overflow rate of 24.5 $m^3/m^2 \cdot d$ (600 gpd/ft^2). Costs include sludge return and waste sludge pumps at solids concentration of 1 percent. TDH = 3.05 m (10 ft). Pumps are non-clog centrifugal pumps having a total pumping capacity of 50 percent of plant capacity and spare pumps included. The costs are for circular clarifiers having a surface area greater than 46.5 m^2 (500 ft^2) and diameters less than 61 m (200 ft). Costs also apply to rectangular clarifiers when the surface area is less than 46.5 m^2 (500 ft^2). To adjust costs for other overflow rates, use *CC* equation with the effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{24.5\ m^3/m^2 \cdot d\ (600\ gpd/ft^2)}{\text{actual design surface overflow rate}}$$

9. *High-Rate Trickling Filter*

$$CC = -0.00007\ Q^2 + 56.89\ Q + 244.791 \tag{C-19}$$

$$\text{O\&M} = 278\ Q^{0.505} \tag{C-20}$$

Service life = 50 years. The construction costs include circular filter units with rotating distributor arms, synthetic media 1.8 m (6 ft), and underdrains. Organic loading rate = 0.72 kg/m^3 (45 lb $BOD_5/10^3\ ft^3 \cdot d$) and hydraulic loading rate = 28.3 $m^3/m^2 \cdot d$ (693 gpd/ft^2) at 3 : 1 recycle ratio. The costs do not include clarifiers and recycling equipment. Electrical power is not included.[1]

10. *Clarifier for High-Rate Trickling Filter (Includes Recycle Pumps)*

$$CC = -0.00005\ Q^2 + 44.77\ Q + 323{,}702 \tag{C-21}$$

$$\text{O\&M} = -0.000003\ Q^2 + 5.2\ Q + 5733 \tag{C-22}$$

Service life = 40 years. The equation is to be used in conjunction with high-rate trickling filter only. Construction costs include sludge pumps, effluent recycle pumps, clarifier mechanisms, and internal piping. Design overflow rate = 32.6 $m^3/m^2 \cdot d$ (800 gpd/ft^2) at average design flow. The recycle pumping capacity = 3 times the average wastewater flow. The costs are for circular clarifiers having a surface area greater than 46.45 m^2 (500 ft^2) and diameters less than 61 m (200 ft). Costs also apply to rectangular clarifiers when the surface area is less than 46.5 m^2 (500 ft^2). To adjust costs for other overflow rates, use *CC* equation with effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{32.6\ m^3/m^2 \cdot d\ (800\ gpd/ft^2)}{\text{actual design surface overflow rate}}$$

11. *Separate Nitrification with Clarifier*

$$CC = -0.00006\, Q^2 + 77\, Q + 636{,}567 \tag{C-23}$$

$$O\&M = -0.000005\, Q^2 + 3\, Q + 52{,}683 \tag{C-24}$$

Service life = 40 years. The system follows a high-rate activated sludge. The construction costs include nitrification tanks, aeration devices, clarifiers, and sludge recycle and waste sludge pumps, but not pH adjustment facilities. Aeration period = 3 h. Oxygen requirement = 1.5 g oxygen per g BOD_5 removed plus 4.6 g oxygen per g NH_4^+-N oxidized. Sludge pumps are sized for 100 percent recycle but operate at 50 percent recycle. Final clarifier is designed for an overflow rate of 24.5 $m^3/m^2 \cdot d$ (600 gpd/ft^2).[1]

12. *Separate Denitrification with Clarifier*

$$CC = -0.00004\, Q^2 + 45\, Q + 398{,}838 \tag{C-25}$$

$$O\&M = 14.5\, Q + 61{,}827 \tag{C-26}$$

Service life = 40 years. Construction costs include uncovered denitrification tanks, mixers, methanol feed at a rate of 3 g per g NO_3-N removed, and storage for 30-day supply. Reaction period in denitrification tank = 2 h; MLVSS concentration = 2000 mg/L. Denitrification recycle pumps sized for 100 percent recycle but operated at 50 percent recycle. The design overflow for the clarifier = 24.5 $m^3/m^2 \cdot d$.[1]

13. *Gravity Filtration (Dual Media)*

$$CC = 2903\, Q^{0.656} \tag{C-27}$$

$$O\&M = 194\, Q^{0.693} \tag{C-28}$$

Service life = 30 years. Construction costs include facilities for backwash storage, all feed and backwash pumps, piping, and filter building and pipe gallery. The design is based on average hydraulic loading of 9.8 $m^3/m^2 \cdot h$ (4 gpm/ft^2) and a backwash ratio of 36.7 $m^3/m^2 \cdot h$ (15 gpm/ft^2). The backwash holding tank capacity is 2 times the backwash quantity. To adjust for hydraulic loading rate other than above, use CC equations with effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{9.8\ \text{m}^3/\text{m}^2\cdot\text{d}(4\ \text{gpd/ft}^2)}{\text{actual design hydraulic loading on filters}}$$

14. *Activated Carbon Absorption*

$$CC = -0.0002\, Q^2 + 156\, Q + 796.55 \tag{C-29}$$

$$O\&M = -0.00001\, Q^2 + 14\, Q + 229{,}458 \tag{C-30}$$

Service life = 35 years. Construction cost is based on multiple carbon columns, feed and backwash pumps, piping, operations building, carbon regeneration facilities, and necessary storage tanks. The design empty bed contact time of carbon column is 30 minutes.[1]

15. *Chlorination*

$$CC = 795\,Q^{0.598} \tag{C-31}$$

$$\text{O\&M} = -0.000001\,Q^2 + 2.36\,Q + 24{,}813 \tag{C-32}$$

Service life = 15 years. Construction costs include chlorine building, chlorine storage and handling facilities, chlorinators, injector, and plug-flow contact chamber. The chlorine dosage is 10 mg/L, and the contact period is 30 min at average flow. Chlorine residual of 1 mg/L is expected in secondary effluent. For higher chlorine dosage, use O&M equation with effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{\text{actual chlorine dosage, mg/L}}{10\text{ mg/L}}$$

16. *Dechlorination Using Sulfur Dioxide*

$$CC = 1170\,Q^{0.429} \tag{C-33}$$

$$\text{O\&M} = -0.000001Q^2 + 0.97Q + 15{,}058 \tag{C-34}$$

Service life = 30 years. The construction costs include SO_2 storage facility, feed system, mixers, and reaction tank (1-min detention time). The SO_2 dosage is 1 mg/L SO_2 per mg/L chlorine residual. For higher SO_2 dosage, use O&M equation with effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{\text{actual } SO_2 \text{ dosage}}{1\text{ mg/L } SO_2}$$

17. *UV Disinfection*

$$CC = -3 \times 10^{-5}\,Q^2 + 11.85\,Q + 142{,}439 \tag{C-35}$$

$$\text{O\&M} = -3 \times 10^{-6}\,Q^2 + 1.038\,Q + 4585 \tag{C-36}$$

Service life = 15 years. Construction costs include UV modules, power distribution center, panel, interconnecting cables to modules, lamps, UV monitoring system, UV safety and cleaning equipment, building, channels and gates, pipings, walkways, electrical power source, and miscellaneous accessories. The O&M costs include power, lamp replacement, cleaning labor, and other miscellaneous items. The UV system is capable of providing required disinfection at peak design flow of 2–3 times the average flow. All cost data are obtained from the equipment manufacturers.

18. *Sludge Pumping*

$$CC = -0.00005\,Q^2 + 44.77\,Q + 323{,}702 \tag{C-37}$$

$$\text{O\&M} = -0.000003\,Q^2 + 5.2\,Q + 5733 \tag{C-38}$$

Service life = 10 years. The costs are based on sludge mass of 0.227 kg/m^3 (1900 lb/10^6 gal) at 4 percent solid concentration, i.e., 5.7 L/m^3 (5700 gal/10^6 gal) for combined primary and secondary sludge after thickening. The pumps and nonclog centrifugal pumps and TDH = 9.15 m (30 ft). For pumping costs for sludge of different quantity and characteristics, use *CC* and O&M equations with effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{\text{actual sludge mass}}{\begin{array}{c}0.227\ \text{kg/m}^3\\(1900\ \text{lb}/10^6\ \text{gal})\end{array}} \times \frac{4\ \text{percent}}{\begin{array}{c}\text{actual solids}\\\text{concentration (percent)}\end{array}}$$

19. *Gravity Thickener*

$$CC = 177\,Q^{0.68} \tag{C-39}$$

$$\text{O\&M} = -0.0000003\,Q^2 + 0.18\,Q + 4136 \tag{C-40}$$

Service life = 50 years. Construction costs include thickener and all related mechanical equipment for thickening of secondary sludge receiving solids at a rate of 0.098 kg/m^3 (820 lb/10^6 gal) and solids loading of 29.3 kg/m^3·d(6 lb/ft^3·d). Sludge pumps are not included. O&M costs do not include polymer or metal addition. For different sludge quantities, concentrations, and thickening properties, use *CC* and O&M equations with effective Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{29.3\ \text{kg/m}^2\cdot\text{d}(6\ \text{lb/ft}^2\cdot\text{d})}{\text{actual solids loading}} \times \frac{\text{actual solids mass}}{0.098\ \text{kg/m}^3\ (820\ \text{lb}/10^6\ \text{gal})}$$

20. *Aerobic Digester*

$$CC = -0.00002\,Q^2 + 23.7\,Q + 208{,}627 \tag{C-41}$$

$$\text{O\&M} = 8.54\,Q^{0.916} \tag{C-42}$$

Service life = 40 years. Construction costs include digestion period of 20 days, sludge flow of 5.7 L/m^3 (5700 gal/10^6 ·gal), or 0.227 kg/m^3 (1900 gal/10^6·gal) at 4 percent solids concentration. The mechanical aerators provide an oxygen requirement of 1.6 g O_2/g VSS destroyed and mixing requirements of 26.3 W/m^3 (134 hp/10^6·gal). To adjust costs for different design conditions, use *CC* and O&M equations with effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{20\ \text{days}}{\begin{array}{c}\text{actual}\\\text{digestion period}\end{array}} \times \frac{\text{actual solids mass}}{\begin{array}{c}0.227\ \text{kg/m}^3\\(1900\text{lb}/10^6\cdot\text{gal})\end{array}} \times \frac{4\ \text{percent}}{\begin{array}{c}\text{actual solids}\\\text{concentration (percent)}\end{array}}$$

21. *Two-Stage Anaerobic Digesters*

$$CC = -0.00002\,Q^2 + 21.28\,Q + 471{,}486 \tag{C-43}$$

$$\text{O\&M} = 0.67\,Q + 26{,}748 \tag{C-44}$$

Service life = 50 years. Capital costs include covered digestion tanks, heat exchanger, gas mixing and collection equipment, and control building. The digesters receive combined thickened sludge at a rate of 0.227 kg/m^3 (1900 lb/10^6·gal) raw wastewater, solids content 4 percent with 75 percent volatile matter. Solids loading rate = 2.56 kg/m^3·d (0.16 lb/ft^3·d). Operating temperature = 29–43°C. Digestor gas is used for heating. The digested sludge has a solids content of 0.108 kg/m^3 (900 lb/10^6·gal) at 2.5 percent solids. To adjust costs for a different design factor, use *CC* and O&M equations with effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{\text{actual solids mass, kg/m}^3}{0.227 \text{ kg/m}^3 (1900 \text{ lb}/10^6\text{·gal})}$$

22. *Sludge-Drying Beds*

$$CC = 89\, Q^{0.854} \tag{C-45}$$

$$\text{O\&M} = -0.00002\, Q^2 + 2.57\, Q + 8003 \tag{C-46}$$

Service life = 20 years. Construction costs include sand beds, sludge inlets, underdrains, cell dividers, sludge pipings, underdrain return, and other structural elements of the beds. The sludge solids and solids loading on the drying beds are 0.108 kg/m^3 (900 lb of sludge/10^6·gal) and 97.6 kg/m^2 per year (20 lb/ft^3· yr). To adjust costs for different sludge quantities, characteristics, and loading rates, use CC and O&M equations with effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{\begin{array}{c}\text{actual solids mass}\\ \text{in digested sludge}\end{array}}{0.108 \text{ kg/m}^3 (900 \text{ lb}/10^6\text{·gal})} \times \frac{97.6 \text{ kg/m}^2\text{·yr}(20 \text{ lb/ft}^2\text{·yr})}{\text{actual design solids loading}}$$

23. *Filter Press or Belt Filter*

$$CC = 10{,}255\, Q^{0.481} \tag{C-47}$$

$$\text{O\&M} = 3165\, Q^{0.348} \tag{C-48}$$

Service life = 15 years. Construction costs include filtration equipment, conveyor equipment, chemical feed and storage facilities, conditioning tanks, sludge storage tanks, and building. Digested primary and secondary sludge quantity 0.108 kg/m^3 (900 lb/10^6·gal) at 2.5 percent solids. Conditioning chemicals dosage, $FeCl_3$ = 4.2 g/m^3 (35 lb/10^6·gal), CaO = 10.8 g/m^3 (90 lb/10^6·gal). To adjust costs for different sludge characteristics, use *CC* and O&M equations with effective flow Q_E.[1]

$$Q_E = Q_{\text{DESIGN}} \times \frac{\text{actual solids mass in digested sludge}}{0.108 \text{ kg/m}^3 (900 \text{ lb}/10^6\text{·gal})}$$

24. *Landfilling (Biological Sludge, Excluding Transportation)*

$$CC = -0.000005\, Q^2 + 2.33\, Q + 87{,}257 \tag{C-49}$$

$$\text{O\&M} = -0.000001\, Q^2 + 1.11\, Q + 25{,}026 \tag{C-50}$$

Service life = 20 years. Construction costs include site preparation, front-end loaders, monitoring wells, fencing, leachate collection, and treatment. Digested and dewatered sludge at solids concentration of 20 percent with soil blending (1 part soil to 3 parts sludge) is landfilled. Digested solids quantity is 0.108 kg/m^3 (900 lb/10^6·gal) or 545.3 m^3/10^6 m^3 (2.7 yd^3/10^6·gal). Operation and maintenance costs include labor costs for operation, preventive maintenance, and minor repairs and fuel for equipment operation. To adjust costs for different quantities and concentrations, use *CC* and O&M equations with effective flow Q_E.[1]

$$Q_E' = Q_{\text{DESIGN}} \times \frac{\text{actual solids mass in dewatered sludge}}{0.108\ \text{kg/m}^3\ (900\ \text{lb}/10^6\text{·gal})} \times \frac{20\ \text{percent}}{\text{actual solids concentration in sludge}}$$

$$Q_E = Q_E' \times \frac{\text{quantity of sludge landfilled}}{\text{total quantity of sludge produced}}$$

25. *Biosolids Utilization*

$$CC = -5 \times 10^{-6}\, Q^2 + 2.047\, Q + 76{,}790 \tag{C-51}$$

$$\text{O\&M} = -10^{-6}\, Q^2 + 0.978\, Q + 22{,}031 \tag{C-52}$$

Service life = 20 years. The capital cost of dewatered biosolids utilization includes transportation vehicles, application vehicles, sludge loading and unloading apparatus, concrete pad, and storage facility. No land costs are incurred since landowners generally agree to accept the dewatered sludge for land application. The operation costs include oil, gas, preventive maintenance, labor, and material. These cost estimates are developed from the EPA manual.[10]

$$Q_E' = Q_{\text{DESIGN}} \times \frac{\text{actual solids mass in dewatered sludge}}{0.108\ \text{kg/m}^3\ (900\ \text{lb}/10^6\text{·gal})} \times \frac{20\ \text{percent}}{\text{actual solids concentration in sludge}}$$

$$Q_E = Q_E' \times \frac{\text{quantity of sludge utilized as biosolids}}{\text{total quantity of sludge produced}}$$

26. *Miscellanous Structures*

$$CC = 1438\, Q^{0.567} \tag{C-53}$$

$$\text{O\&M} = -0.000003Q^2 + 1.97Q + 57{,}349 \tag{C-54}$$

Service life = 50 years. Construction costs include administrative offices, labratories, machine shops, and garage facilities. Operation and maintenance costs include utilities and normal upkeep.[1]

27. *Support Personnel*

$$O\&M = 8.31\ Q^{0.717} \tag{C-55}$$

Costs include the manpower required to support the operation of wastewater treatment facility. These support functions include supervision and administration, clerical work, labratory work, and administrative costs.[1]

REFERENCES

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9. Macal, C. M., *Simulation of Waste Treatment (SWAT) Model: a Method for Cost-Effective Analysis of Wastewater Treatment Strategies,* Energy and Environmental Systems Division, Argonne National Laboratory, Department of Energy, Springfield, VA, 1978.
10. U.S. Environmental Protection Agency, *Process Design Manual: Land Application of Municipal Sludge,* EPA-625/1-83-016, Municipal Environmental Laboratory, Cincinnati, OH, October 1983.

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Manufacturers and Suppliers of Wastewater Treatment Plant Equipment

An alphabetical listing of the manufacturers of water pollution control equipment for design of wastewater treatment plants is provided in this appendix. This list is developed from the *1997–98 Buyers' Guide of Water Environmental Federation, 1997 Catalog and Buyers Guide of Pollution Equipment News,* and *1997–98 Buyers' Guide of Water and Waste Digest.* The keywords for the listing are based on major functions, pollutants, and class of equipment. Each listing, where possible, includes the complete address, phone number, and fax number. Toll-free phone number and web site have also been listed when available.

Other Catalog and Buyers Guides of pollution control equipment buyers are found in *Water, Industrial Wastes, Water Engineering and Management, Industrial Water Engineering, Plant Engineering, Specifying Engineering, Instrument Society of America, Thomas Register,* and many others.

LISTING BY TYPE

Aerators, Diffusers and Aeration Systems

Aeration Industries Inc.
Aercor
Aqua-Aerobic Systems Inc.
Environmental Dynamics, Inc.
EnviroQuip Inc.
EnviroQuip International Inc.
Infilco Degrement Inc.

Jet Tech Inc.[a]
Lakeside Equipment Co.
Lightnin Aerators
Mass Transfer System Inc.
Philadelphia Mixers
Sanitaire WPCC
Zimpro[a]

Aerobic Wastewater Treatment System and Package Plants

Davco Manufacturing Division[a]
Eimco Process Equipment Co.
Envirex[a]
Environmental Dynamics Inc.
EnviroQuip Inc.
Hydro-Aerobics, Inc.
Jet-Tech[a]
Parkson Corporation
Sanitaire WPCC

Anaerobic Wastewater Treatment

Biothane Corp.
CBI Walker, Inc.
Davco Manufacturing Division[a]
Envirex[a]
Infilco Degremont Inc.
Westech Engineering Inc.
Zimpro[a]

Bar Screens

ENVIREX, USF
FMC Corporation, MHS Division
Infilco Degremont Inc.
Parkson Corporation
Vulcan Industries, Inc.

[a]A Division of U.S. Filter

Biosolids

BioGrow Div., Wheelabrator Water Technologies
Biotech International

Blowers and Compressors

Dresser Industries Roots Operations
Hoffman Air & Filtration Systems
Lamson Centrifugal Blowers
Semblex, Inc.
Spencer Turbine Co.
Turblex Inc.

Centrifuge

Alfa-Laval Separation, Inc. of Sharples Division
Andritz Ruthnor Inc.
Humboldt Decanter Inc.
Naro Separation, Inc.

Chemical Feed Equipment

Acrison Inc.
Fluid Dynamics Inc.
LMT Milton Roy
Merrick Industries Inc.
PulsaFeeder, Inc.
Semblex, Inc.
Wallace & Tiernan Inc.[a]

Chemicals

Carus Chemical Company
The Dow Chemical Company
Eaglebrook Inc.
Fe_3 Inc.
National Lime Association
Nutech Environmental Corp.
Polydyne Inc.
Sepro Corp.
Tetra Technologies Inc.

Chlorinators and Dechlorinators

Capital Controls Co.
Chlorinators Inc.
Gardiner Equipment Co.
Pollution Control Engineering, Inc.
Wallace & Tiernan Inc.[a]

Clarifiers and Sedimentation Basins

Dorr Oliver Inc.
Eimco Process Equipment Co.
FMC Corp., MHS Division
Envirex[a]
EnviroQuip Inc.
High-Tech Incorporated
Infilco Degremont Inc.
Lakeside Equipment Corp.
F.B. Leopold
NRG Co. Inc.
Smith & Loveless Inc.
United Industries Inc.
Walker Process Equipment, Div. of McNish Corp.
Westech Engineering Inc.

Coagulants

Cytec Industries Inc.
Nalco Chemical Company
Stockhausen Inc.
Tetra Technologies Inc.

Comminutors

EnviroQuip International Inc.
Franklin Miller Inc.
Ingersoll-Dresser Pump Co.
JWC Environmental
Robbins & Myers Inc.

Controls

Aurora Pump
Automation Products, Inc.
Bailey-Fischer & Porter
Baldor Electric Co.
Eagle Microsystems
Omron Electronics, Inc.
Phoenix Contacts Inc.
Westinghouse Electric Corporation

Demineralization

Osmonics Inc.
Tetra Technologies Inc.
U.S. Filter Recovery Services

Denitrification

Austgen-Biojet Wastewater System
Envirex[a]
Eimco Process Equipment Co.
Infilco Degremont Inc.
Jet Tech Inc.[a]
Parkson Corporation
U.S. Filter Recovery Services

Dewatering Equipment Services

Alfa Lavel Separation Inc. of Sharples Division
Alfa Laval Thermal Inc.
Andritz Ruthner Inc.
Asdor, Edwards & Jones Group[a]
Ashbrook Corp.
Eimco Process Equipment Co.
Envirex[a]
EnviroQuip Inc.
Humboldt Decanter Inc.
Komline-Sanderson
Roediger Pittsburg Inc.
Somat Corp.
Zimpro[a]

Anaerobic Digestion Tank Equipment

Biothane Corp.
Eimco Process Equipment Co.
Envirex[a]
Infilco Degremont Inc.
Kruger Inc.
Walker Process Equipment
Westech Engineering Inc.

Disinfection Equipment

Bailey-Fischer & Porter
Capital Controls Company
Gardiner Equipment Company
Infilco Degremont Inc.
Ozonia North America
Trojan Technologies Inc.
Ultra Tech International Inc.
Wallace & Tiernan Inc.[a]

Feeders, Automatic

Aerison Inc.
Bailey Fischer & Porter
Fluid Dynamics Inc.
Merrick Industries Inc.
Semblex, Inc.
Wallace & Tiernan Inc.
Wheelabrator Technologies

Filter Equipment

Agency Environmental Inc.
Aqua Aerobics Systems Inc.
Davco Manufacturing Division[a]
Eimco Process Equipment Co.
Eimco Filtration System
EnviroQuip Inc.
General Filter Products[a]
Infilco Degremont
F.B. Leopold
Tetra Technologies Inc.

Filter Media

Agency Environmental Inc.
Eimco Process Equipment Co.
EnviroQuip Inc.
F.B. Leopold
General Filter Products[a]
Infilco Degremont Inc.
Parkson Corp.
Tetra Technologies Inc.
UNIFILT Corp.

Flocculants

Allied Colloids
Fe_3 Inc.
Nalco Chemical Co.
Norchem Industries
Stockhausen Inc.
Summit Research Labs
Tetra Technologies Inc.

Flocculating Equipment

Chemineer Inc.
Davco Manufacturing Div.[a]
Eimco Process Equipment Co.
FMC Corporation, MHS Division
Kruger Inc.
Lightenin
Philadelphia Mixers
Walker Process Equipment
Westech Engineering, Inc.

Flotation Equipment

Eimco Process Equipment Co.
Envirex[a]
Infilco Degremont
Krofta Engineering Corp.
Walker Process Equipment
Westech Engineering Inc.

Flow Measurement

Badger Meter Inc.
Bailey Fischer & Porter
Controlotron
Dynasonics, Inc.
Inventron, Inc.
Milltronics Inc.
Polysonics Inc.
Sparling Instruments Co.

GRIT COLLECTION

E & I Corp
Envirex[a]
Eutek System Inc.
HIL Technology Inc.
Infilco Degremont Inc.
John Meunier Inc.
Jones & Attwood Inc.
Smith and Loveless Inc.
Westech Engineering Inc.

Instruments (Recording and Control)

Badger Meter, Inc.
Bailey-Fischer & Porter
Bristol Babcock Inc.
Capital Controls Co.
Chemetrics Inc.
Great lakes Instruments, Inc.
Hatch Company
Wallace & Tiernan Inc.[a]

Phosphate Removal Systems

Austgen-Biojet Wastewater Systems
Eimco Process Equipment Co.
Envirex[a]
Infilco Degremont Inc.
Jet Tech Inc.[a]
Kruger Inc.

Pumps

Alfa Lavol Pumps Inc.
Aurora Pump
Bornemann Pumps Inc.
Davis EMU
Discflo Corp.
EBARA International Inc.
Gorman Rupp Co.
Goulds Pumps Inc.
Hayward Gordon Ltd.
Homa Pump Technology
Ingersoll-Dresser Pump Co.
ITT A-C Pump/ITT Marlow
Komline-Sanderson
LMT Milton Roy
Myers FE Co.
Netzsch Inc.
Paco Pumps Inc.
Patterson Pump
Penn Valley Pump Co.
Prominent Fluid Controls Inc.
Pulsa Feeder Inc.
Robbins & Mayer Inc.
Smith & Loveless Inc.
Vaughan Co. Inc.
Wallace & Tiernan Inc.[a]
Watson-Marlow, Inc.
Yeomans Pump

Reverse Osmosis Equipment

Memtec America Corp.
Memtek[a]
Osmonics Inc.
Zenon Environmental Inc.

Rotating Biological Contactors

Schreiber Corp Inc.
Envirex[a]
Walker Process Equipment, Div. of
McNish Corp.

Scada

Alpine Technology Inc.
Biotronics Technologies Inc.
Consolidated Electric Co.[a]
Kruger Inc.
Phoenix Contact Inc.
Westinghouse Electric Corp.

Screening Equipment

Antritz Ruthner Inc.
Envirex[a]
FMC Corp., MHS Division
Hycor Corp.
Infilco Degremont Inc.
Jones & Attwood Inc.
Parkson Corp.
Vulcon Industries, Inc.

Sequencing Batch Reactors

Austgen-Biojet Wastewater System
Jet Tech, Inc.[a]
Walden Inc.

Sludge Concentration/ Thickening Equipment

Alfa-Laval Separation Inc., Sharples Division
Andritz Ruthner, Inc.
Ashbrook Corp.
Bird Machine Co.
Eimco Process Equipment Co.
Envirex[a]
EnviroQuip Inc.
Gravity Flow Systems, Inc.
Hi-Tech Incorporated
Humboldt Decanter Inc.
Komline-Sanderson
Krofta Engineering Corp.
Lakeside Equipment Corp.
Parkson Corp.
Roediger Pittsburg, Inc.
Somat Corp.
Walker Process Equipment
Westech Engineering Inc.

Tertiary Processes

CBI Walker, Inc.
Envirex[a]
Infilco Degremont, Inc.
Kruger Inc.
Memtec America Corp., Memcore Division
Zimpro[a]
Zenon Environmental Inc

Sludge Handling and Control

Asdor, Edwards & Jones Group[a]
Ashbrook Corp.
Bird Machine Co.
Envirex[a]
FMC Corporation, MHS Division
Gravity Flow Systems
RDP Company
Serpentix Conveyor Corp.
Walker Process Equipment, Div. of McNish Corp.
Zimpro[a]

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www.acrison.com

AERATION INDUSTRIES INC.
4100 Peavey Road
Chaska, MN 55318
(612) 448-6789
Fax (612) 448-7293

AERCOR
6 Industrial Drive
Sterling, MA 01564
(508) 422-7505
Fax (508) 422-7551

AGENCY ENVIRONMENTAL INC.
2600 Cabover Drive, Suite J
Hanover, MD 21075
(410) 760-6878
Fax (410) 760-6877

ALFA LAVAL PUMPS INC.
P.O. Box 1426
Kenosha, WI 53141-1426
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Fax (414) 942-0177

ALFA LAVAL SEPARATION INC., SHARPLES DIVISION
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Warminster, PA 18974-0556
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or (215) 443-4012
Fax (215) 443-4139
or (215) 443-4011

ALFA LAVAL THERMAL INC.
4093 Gettysburg St.
Ventura, CA 93003
(805) 642-9721
Fax (805) 642-8222

ALLIED COLLOIDS
2301 Wilroy Rd., P.O. Box 820
Suffolk, VA 23439-0820
(757) 934-3700
Fax (757) 538-0204
www.alliedcolloids.com

ALPINE TECHNOLOGY INC.
2 Cielo Center, Suite 300
1250 Capitol of Texas Highway
Austin, TX 78746
(512) 329-2809
Fax (512) 328-4792

ANDRITZ RUTHNER INC.
1010 Commercial Blvd. S.
Arlington, TX 76017
(817) 465-5611
Fax (817) 468-3961

AQUA AEROBIC SYSTEMS INC.
6306 N. Alpine Road
P.O. Box 93-0785
Rockford, IL 61130
(815) 654-2501
Fax (815) 654-2508
www.aqua-aerobic.com

ASDOR, EDWARDS & JONES GROUP[a]
1255 Nicholson Road
New Market,
ON L3Y7V1 Canada
(905) 836-7700

ASHBROOK CORP.
11600 E. Hardy
Houston, TX 77093
(713) 449-0322
Fax (713) 449-1324

AURORA PUMP
800 Airport Rd.
North Aurora, IL 60542
(630) 859-7000
Fax (630) 859-7060
www.gspump.com

AUSTGEN-BIOJET WASTEWATER SYSTEM
500 Sansome St.
Suite 500
San Francisco, CA 94111-0221

AUTOMATION PRODUCT, INC.
3030 Maxroy Street
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(713) 869-0361 or (800) 231-2062
Fax (713) 869-7332

BADGER METER, INC.
6116 East 15th Street
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Tulsa, OK 74112
(918) 836-8411
Fax (918) 832-9962

BAILEY-FISCHER & PORTER
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Warminster, PA 18974
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Fax (215) 674-7183
www.ebpa.com/bfp/bfphome.htm

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BIO GRO DIV., WHEELABRATOR WATER TECHNOLOGIES
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(334) 341-7064
www.biotech.com

BIOTHANE CORP.
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Fax (609) 541-3366
www.biothane.com

BIOTRONICS TECHNOLOGY INC.
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Eastmound Drive
Waukesha, WI 53186

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South Walpole, MA 02071-9103
(508) 668-0400
Fax (508) 668-6855

BORNEMANN PUMPS INC.
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921 Matthews-Mint Hill Road, Suite K
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Watertown, CT 06795
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Fax (860) 945-2278
www.bristolbabcock.com

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Colmar, PA 18915
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www.capitalcontrols.com

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Fax (815) 224-6697
www.caruschem.com

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CHEMETRICS INC.
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www.chemetrics.com

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Dayton, OH 45401
(513) 454-3200
Fax (513) 454-3379
www.chemineer.com

CHLORINATORS INC.
P.O. Box 1822
Palm City, FL 34990-6822
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Fax (561) 287-3238

CONSOLIDATED ELECTRIC CO.[a]
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Fax (612) 224-3628

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Fax (516) 231-3334
www.controlotron.com

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www.discflo.com

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www.dorr-oliver.com

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Fax (610) 323-0114

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www.eden.com/~ envirogp/

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www.fibergrate.com

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Fax (215) 822-4342
www.wateronline.com/companies/fmc/index.html

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www.franklinmiller.com

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INFILCO DEGREMONT INC.
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www.wateronline.com/companies/infilco
www.infilcodegremont.com

INGERSOLL-DRESSER PUMP CO.
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INVENTRON INC.
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Fax (313) 513-7705
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Fax (513) 482-2569
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www.lightnin-mixers.com

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LMT MILTON ROY
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Acton, MA 01720

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Fall River, MA 02780-4732
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Lynn Haven, FL 32444
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Fax (904) 265-9768

MILLTRONICS INC.
709 Stadium Drive
Arlington, TX 76011
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Fax (817) 277-3894
www.milltronics.com

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1101 Myers Pkwy
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Fax (419) 289-6658

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www.libertynet.org/~nrgco

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www.nutechenvironmental.com

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Schaumburg, IL 60173
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OSMONICS INC.
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Fax (612) 933-0141
www.osmonics.com

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Fax (281) 934-6090
www.paco-pumps.com

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Fax (706) 886-0023
www.pattersonpumps.com

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Warrington, PA 18976-1800
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PHOENIX CONTACT INC.
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Fax (717) 944-1625
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(713) 530-0885

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POLLUTION CONTROL ENGINEERING, INC.
27 Mauchly, Ste. 212
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2883 Brighton-Henrietta Town Line Rd.
Rochester, NY 14623
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www.sanitaire.com

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100 Schreiber Drive
Trussville, AL 35173
(205) 655-7466
Fax (205) 655-7669
www.schreiber-water.com

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1635 W. Walnut St.
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11550 North Meridian
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www.serpentix.com

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Fax (913) 888-2173
www.smithandloveless.com

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Coatesville, PA 19320
(610) 384-7000
Fax (610) 380-8500
www.somatcorp.com

SPARLING INSTRUMENTS CO.
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El Monte, CA 91731
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SPENCER TURBINE CO.
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E

Selected Chemical Elements

TABLE E-1 Chemical Symbols, Atomic Numbers, Atomic Weights, and Valences of Common Elements[a]

Element	Symbol	Atomic Number	Atomic Weight	Valence
Aluminum	Al	13	26.9815	3
Antimony	Sb	51	121.75	3,5
Argon	Ar	18	39.948	—
Arsenic	As	33	74.9216	3,5
Barium	Ba	56	137.34	2
Beryllium	Be	4	9.0122	—
Bismuth	Bi	83	208.980	3,5
Boron	B	5	10.811	3
Bromine	Br	35	79.909	1,3,5,7
Cadmium	Cd	48	112.40	2
Calcium	Ca	20	40.08	2
Carbon	C	6	12.01115	4
Chlorine	Cl	17	35.453	1,3,5,7
Chromium	Cr	24	51.996	2,3,6
Cobalt	Co	27	58.9332	2,3
Copper	Cu	29	63.54	1,2
Fluorine	F	9	18.9984	1
Gold	Au	79	196.967	1,3
Helium	He	2	4.0026	—
Hydrogen	H	1	1.00797	1
Iodine	I	53	126.9044	1,3,5,7
Iron	Fe	26	55.847	2,3
Lead	Pb	82	207.19	2,4
Lithium	Li	3	6.939	1
Magnesium	Mg	12	24.312	2
Manganese	Mn	25	54.9380	2,3,4,6,7
Mercury	Hg	80	200.59	1,2
Molybdenum	Mo	42	94.94	3,4,6
Neon	Ne	10	20.183	—
Nickel	Ni	28	58.71	2,3
Nitrogen	N	7	14.0067	3,5
Oxygen	O	8	15.9994	2
Phosphorus	P	15	30.9738	3,5
Platinum	Pt	78	195.09	2,4
Potassium	K	19	39.102	1
Radium	Ra	88	226.05	—
Silicon	Si	14	28.086	4
Silver	Ag	47	107.870	1

TABLE E-1 Chemical Symbols, Atomic Numbers, Atomic Weights, and Valences of Common Elements[a]—cont'd

Element	Symbol	Atomic Number	Atomic Weight	Valence
Sodium	Na	11	22.9898	1
Strontium	Sr	38	87.62	2
Sulfur	S	16	32.064	2,4,6
Tin	Sn	50	118.69	2,4
Titanium	Ti	22	47.90	—
Tungsten	W	74	183.85	3,4
Uranium	U	92	238.03	4,6
Zinc	Zn	30	65.37	2

[a]For a complete list, consult a handbook of chemistry and physics.

F

Abbreviations and Symbols, Useful Constants, Unit Conversions, Design Parameters, and Units of Expression for Wastewater Treatment

TABLE F-1 Abbreviations and Symbols

Abbreviations and Symbols		Abbreviations and Symbols	
Acceleration due to gravity (LT^{-2})	g	Gallons per minute (L^3T^{-1})	gal/min or gpm
Area (L^2)	A	Gallons per second (L^3T^{-1})	gal/s or gps
Atmosphere	atm	Horsepower	hp
British thermal unit	Btu	Horsepower-hour	hp·h
Calorie	cal	Hour	h
Centimeter	cm	Inch	in.
Centipoise	cp	Joule	J
Cubic centimeter	cm^3	Kilocalorie	kcal
Cubic feet per minute	cfm or ft^3/min	Kilonewton per cubic meter	kN/m^3
Cubic feet per second	cfs or ft^3/s	Liter (L^3)	L
Cubic meter	m^3	Million gallons per day (L^3T^{-1})	mgd
Degree Celsius	°C	Pound force	lb_f
Degree Fahrenheit	°F	Pound mass	lb_m
Degree Kelvin	°K	Standard temperature and pressure	STP (273°K, 760 mmHg)
Degree Rankine	°R	Watt	W
Density (ML^{-3})	ρ		
Discharge (L^3T^{-1})	Q		
Energy	E		
Feet per minute (LT^{-1})	fpm or ft/min		
Feet per second (LT^{-1})	fps or ft/s		
Gallon (L^3)	gal		

M = mass; L = length; T = time.

TABLE F-2 Values of Useful Constants

Acceleration caused by gravity, $g = 9.807$ m/s^2 (32.174 ft/s^2)
Standard atmosphere = 101.325 kN/m^2 (14.696 lb_f/in^2)
= 101.325 kPa (1.013 bars)
= 10.333 m (33.899 ft) of water
Molecular weight of air = 29.1
Density of air (15°C and 1 atm) = 1.23 kg/m^3 (0.0766 lb_m/ft^3)
Dynamic viscosity of air, μ (15°C and 1 atm) = 17.8×10^{-6} N·s/m^2 (0.372×10^{-6} lb_f·s/ft^2)
Kinematic viscosity of air, ν (15° and 1 atm) = 14.4×10^{-6} m^2/s (155×10^{-6} ft^2/s)
Specific heat of air = 1005 J/kg°·K (0.24 Btu/lb_m°·R)

TABLE F-3 Conversion Factors, Mixed Units

Length (L)

mile	yard	ft	in.	m	cm
1	1760	5280	6.336×10^4	1.609×10^3	1.609×10^5
5.68×10^{-4}	1	3	36	0.9144	91.44
1.894×10^{-4}	0.333	1	12	0.3048	30.48
1.578×10^{-5}	0.028	0.083	1	0.0254	2.54
6.214×10^{-4}	1.094	3.281	39.37	1	100

Area (A)

mile^2	acre	yard^2	ft^2	in^2	m^2
1	640[a]	3.098×10^6	2.788×10^7	4.014×10^9	2.59×10^6
1.563×10^{-3}	1	4840	43,560	6.27×10^6	4047
3.228×10^{-7}	2.066×10^{-4}	1	9	1296	0.836
3.587×10^{-8}	2.3×10^{-5}	0.111	1	144	0.093
2.491×10^{-10}	1.59×10^{-7}	7.716×10^{-4}	6.944×10^{-3}	1	6.452×10^{-4}
3.861×10^{-7}	2.5×10^{-4}	1.196	10.764	1550	1

Volume (V)

acre·ft	U.S. gal	ft^3	in.^{3}	L	m^3	cm^3
1	325,851	43,560	75.3×10^6	1.23×10^6	1230	1.23×10^9
3.07×10^{-6}	1	0.134	231.552	3.785	3.875×10^{-3}	3875.412
2.3×10^{-5}	7.481	1	1728	28.317	0.028	28,316.846
1.33×10^{-8}	4.329×10^{-3}	5.787×10^{-4}	1	0.016	1.639×10^{-5}	16.387
8.1×10^{-7}	0.264	0.035	61.024	1	1×10^{-3}	1000
8.13×10^{-4}	264.2	35.31	6.10×10^4	1000	1	10^6

Time (T)

yr	mon	d	h	min	s
1	12	365	8760	525,600	3.1536×10^7

Velocity (L/T)

ft/s	ft/min	m/s	m/min	cm/s
1	60	0.3048	18.29	30.48
0.017	1	5.08×10^{-3}	0.3048	0.5080
3.281	196.8	1	60	100
0.055	3.28	0.017	1	1.70
0.032	1.969	0.01	0.588	1

[a] 1 acre = 0.4047 ha (hectare). 1 ha = 10,000 m^2.

Continued

TABLE F-3 Conversion Factors, Mixed Units—cont'd

Discharge (L^3/T)

mgd	gpm	ft^3/s	ft^3/min	L/s	m^3/d
1	694.4	1.547	92.82	43.747	3.78×10^3
1.44×10^{-3}	1	2.228×10^{-3}	0.134	0.063	5.45
0.646	448.9	1	60	28.317	2446.59
0.011	7.481	0.017	1	0.472	40.78
0.023	15.851	0.035	2.119	1	86.41
2.64×10^{-4}	0.183	4.09×10^{-4}	0.025	0.012	1

Mass (M)

ton	lb_m	grain	ounce (oz)	kg	g
1	2000	1.4×10^7	32,000	907.185	907,184.70
0.0005	1	7000	16	0.454	454
7.14×10^{-8}	1.429×10^{-4}	1	2.29×10^{-3}	6.48×10^{-5}	0.065
3.125×10^{-5}	0.0625	437.56	1	0.028	28.35
1.10×10^{-3}	2.205	1.54×10^4	35.274	1	1000
1.10×10^{-6}	2.20×10^{-3}	15.43	0.035	10^{-3}	1

Temperature (T)

°F	°C	°K	°R
°F	$\frac{5}{9}$ (°F − 32)	$\frac{5}{9}$ °F + 255.38	°F + 459.69
$\frac{9}{5}$ °C + 32	°C	°C + 273.16	$\frac{9}{5}$ °C + 491.69
$\frac{9}{5}$ °K − 459.69	°K − 273.16	°K	$\frac{9}{5}$ °K
°R − 459.69	$\frac{5}{9}$ °R − 273.16	$\frac{5}{9}$ °R	°R

Density (M/L^3)

lb/ft^3	lb/gal (U.S.)	kg/m^3	kg/L	g/cm^3
1	0.1337	16.019	0.01602	0.01602
7.48	1	119.815	0.1198	0.1198
0.0624	8.345×10^{-3}	1	0.001	0.001
62.43	8.345	1000	1	1

TABLE F-3 Conversion Factors, Mixed Units—cont'd

Pressure (F/L^2)

lb/in.^{2}	ft water	in Hg	atm	mm Hg	kg/cm^2	N/m^2
1	2.307	2.036	0.068	51.71	0.0703	6894.8
0.4335	1	0.8825	0.0295	22.414	0.0305	2989
0.4912	1.133	1	0.033	25.40	0.035	3386.44
14.70	33.93	29.92	1	760	1.033	1.013×10^5
0.019	0.045	0.039	1.30×10^{-3}	1	1.36×10^{-3}	133.34
14.225	32.783	28.96	0.968	744.657	1	98,070
1.45×10^{-4}	3.35×10^{-4}	2.96×10^{-4}	9.87×10^{-6}	7.50×10^{-3}	1.02×10^{-5}	1

Force (F)

lb_f	N	dyne
1	4.448	4.448×10^5
0.225	1	10^5
2.25×10^{-6}	10^{-5}	1

Energy (E)

kW·h	hp·h	Btu	J	kJ	calories
1	1.341	3412	3.6×10^6	3600	8.6×10^5
0.7457	1	2545	2.684×10^6	2684.5	6.4×10^5
2.930×10^{-4}	3.929×10^{-4}	1	1054.8	1.055	252
2.778×10^{-7}	3.72×10^{-7}	9.48×10^{-4}	1	0.001	0.239
2.778×10^{-4}	3.72×10^{-4}	0.948	1000	1	239
1.16×10^{-6}	1.56×10^{-6}	3.97×10^{-3}	4.186	4.18×10^{-3}	1

Power (P)

kW	Btu/min	hp	ft·lb/s	kg·m/s	cal/min
1	56.89	1.341	737.6	102	14,330
0.018	1	0.024	12.97	1.793	252
0.746	42.44	1	550	76.09	10,690
1.35×10^{-3}	0.077	1.82×10^{-3}	1	0.138	19.43
9.76×10^{-3}	0.558	0.013	7.233	1	137.55
6.977×10^{-5}	3.97×10^{-3}	9.355×10^{-5}	0.0514	7.12×10^{-3}	1

Continued

TABLE F-3 Conversion Factors, Mixed Units—cont'd

Viscosity

Dynamic Absolute Viscosity (μ)

cp	$lb_f \cdot s/ft^2$	$lb_m/ft \cdot s$	$g/cm \cdot s$	$N \cdot s/m^2$	$kg/m \cdot s$	dp
1	2.09×10^{-5}	6.72×10^{-4}	0.01	1×10^{-3}	1×10^{-3}	1×10^{-3}
4.78×10^4	1	32.15	478.47	47.85	47.85	47.85
1488.09	0.031	1	14.881	1.488	1.488	1.488
100	2.09×10^{-3}	0.0672	1	0.10	0.10	0.10
1000	0.021	0.672	10	1	1	1

Kinematic Viscosity (ν)

centistoke	ft^2/s	cm^2/s	m^2/s	myriastoke
1	1.076×10^{-5}	0.01	1.0×10^{-6}	1.0×10^{-6}
9.29×10^4	1	929.368	0.093	0.093
100	1.076×10^{-3}	1	1.0×10^{-4}	1.0×10^{-4}
10^6	10.76	10^4	1	1

TABLE F-4 Design Parameter and Units of Expressions for Wastewater Treatment Plant Design

Design Parameter	Typical Design Values (SI units)	SI Units	Factor K Divide by K ← to Obtain Multiply by K → to Obtain	U.S. Customary Units
SCREENING				
m^3 screening/10^3m^3 flow	3.4×10^{-3}–8.0×10^{-2}	$m^3/10^3m^3$	133.68	$ft^3/10^6 \cdot gal$
GRIT REMOVAL				
Volume,				
m^3 grit/10^3m^3 flow	7.4×10^{-3}–9.0×10^{-2}	$m^3/10^3m^3$	133.68	$ft^3/10^6 \cdot gal$
Surface overflow rate,				
m^3 flow/m^2 surface area·d	898–2040	$m^3/m^2 \cdot d$	24.55	gpd/ft^2
Detention time, s	10–100	—	—	—
FLOW EQUALIZATION				
Power for mixing,				
kW/m^3 tank volume	0.004–0.008	kW/m^3	5.20	$hp/10^3$ gal
Air supply,				
m^3 air/m^3 tank volume·min	0.01–0.015	$m^3/m^3 \cdot min$	133.68	$ft^3/10^3$ gal min
SEDIMENTATION				
Surface overflow rate,				
m^3 flow/m^2 surface area·d	20–82	$m^3/m^2 \cdot d$	24.55	gpd/ft^2
Solids loading rate,				
kg solids/m^2 surface area·d (final clarifier)	40–100	$kg/m^2 \cdot d$	0.205	$lb/ft^2 \cdot d$

Continued

TABLE F-4 Design Parameter and Units of Expressions for Wastewater Treatment Plant Design—cont'd

Design Parameter	Typical Design Values (SI units)	SI Units	Factor K Divide by K ← to Obtain Multiply by K → to Obtain	U.S. Customary Units
Detention time, h	0.5–5	—	—	—
Weir loading rate,				
m^3 flow/m weir length·d	103–500	m^3/m·d	80.52	gpd/ft
Volume of sludge,				
m^3 sludge/10^3 m^3 flow	2.2–11.22	$m^3/10^3 m^3$	133.68	$ft^3/10^6$·gal
Weight of dry sludge solids,				
g dry solids/m^3 flow	60–240	g/m^3	8.35	$lb/10^6$·gal
Sludge concentration,				
percent total solids in sludge	1–10	—	—	1–10
ACTIVATED SLUDGE				
BOD_5 loading				
kg BOD_5/m^3 aeration tank·d	0.4–1.6	kg/m^3·d	62.43	$lb/10^3 ft^3$·d
g BOD_5/g volatile solids·d	0.1–1.0	g/g·d	1	lb/lb·d
kg BOD_5/m^2 surface area·d	0.015–0.15	kg/m^2·d	0.205	lb/ft^2·d
Air requirement,				
m^3 air/kg BOD_5 removed	19–95	m^3/kg	16.02	ft^3/lb
m^3 air/m^3 wastewater treated	3.7–15	m^3/m^3	0.134	ft^3/gal
Oxygen requirement,				
kg O_2/kg BOD_5 applied·d	0.7–2.4	kg/kg·d	1	lb/lb·d

Oxygen transfer rate,				
kg O_2 transferred/kW h	1.2–2.4	kg/kW·h	1.644	lb/hp·h
kg O_2 transferred/m^3 wastewater·h	—	kg/m^3·h	0.063	lb/ft^3·h
Return sludge				
Return flow/Incoming flow	0.1–0.5	—	—	—
BIOLOGICAL FILTERS				
(Trickling filter and biological contactor)				
Hydraulic loading				
Low rate, m^3flow/m^2·d surface area	1–4	m^3/m^2·d	24.54	gpd/ft^2
High rate, m^3flow/m^2·d surface area	8–40	m^3/m^2·d	24.54	gpd/ft^2
BOD_5 loading				
Low rate, kg BOD_5/m^3·d filter volume	0.08–0.4	kg/m^3·d	62.43	lb/$10^3 ft^3$·d
High rate, kg BOD_5/m^3·d filter volume	0.3–1.0	kg/m^3·d	62.43	lb/$10^3 ft^3$·d
Recirculation				
Low rate (percent of incoming flow)	0–50	—	—	—
High rate (percent of incoming flow)	25–300	—	—	—
SAND FILTER				
Hydraulic loading				
m^3flow/m^2·min surface area	0.08–0.48	m^3/m^2·min	24.54	gpm/ft^2

Continued

TABLE F-4 Design Parameter and Units of Expressions for Wastewater Treatment Plant Design—cont'd

Design Parameter	Typical Design Values (SI units)	SI Units	Factor K Divide by K ← to Obtain Multiply by K → to Obtain	U.S. Customary Units
OXIDATION POND				
BOD_5 loading				
kg BOD_5/ha·d surface area	15–160	kg/ha·d	0.892	lb/acre·d
CHLORINATION				
Dosage				
mg/L of chlorine applied (feed rate)	4–40	mg/L	8.35	lb/10^6·gal
SLUDGE THICKENING				
Sludge loading (gravity),				
kg dry solids/m^2·d surface area	20–100	kg/m^2·d	0.205	lb/ft^2·d
Surface overflow rate,				
m^3 wastewater/m^2·d surface area	14–36	m^3/m^2·d	24.54	gal/ft^2·d
DIGESTION				
Organic loading,				
kg volatile solids/m^3·d digester volume	0.8–8	kg/m^3·d	62.43	lb/10^3ft^3·d
BOD_5 loading rate,				
kg BOD_5/m^3·d digester volume	0.16–1.6	kg/m^3·d	62.43	lb/10^3ft^3·d
Sludge heating,				
W/m^2·surface area°C	0.60–5.0	W/m^2°C	0.1763	Btu/h·ft^2·°F

SLUDGE DEWATERING				
Sludge drying bed				
kg dry solids/m^2 area·yr	100–400	kg/m^2·yr	0.205	lb/ft^2·yr
Feed depth in m/yr wet sludge	0.9–3.6	m/yr	3.281	ft/yr
m^2·area/persons	0.12–0.30	m^2/persons	10.76	ft^2/person
Pressure or vacuum filtration,				
kg dry solid/m^2·h filter surface	6–60	kg/m^2·h	0.205	lb/ft^2·h
m^3wet sludge/m^2·h surface area	0.2–1.2	m^3/m^2·h	3.281	ft^3/ft^2·h
kN/m^2 (kPa) pressure applied	300–2000	kPa, kN/m^2	0.145	lb/in^2
kN/m^2 (kPa) vacuum applied	40–70	kPa, kN/m^2	0.296	in Hg
Heat drying,				
kJ heat energy required/kg water evaporated	600–1000	kJ/kg	0.430	Btu/lb
kg water evaporated/h	—	kg/h	2.205	lb/h
kg wet sludge/m^2heating·h	—	kg/m^2h	0.205	lb/ft^2·h
Incineration,				
kg sludge/m^2 effective hearth area·h	36–60	kg/m^2 h	0.205	lb/ft^2·h
LAND DISPOSAL OF WASTEWATER AND SLUDGE				
kg solids/ha field area	—	kg/ha	0.892	lb/acre
m^3 liquid/ha field area·d	—	m^3/ha·d	106.91	gal/acre·d

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About the Author

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