

Reading List for CIV4288/5178/6178 Water Treatment

Selected pages from
*Wastewater Engineering:
Treatment and Resource
Recovery* 5th Edition
Metcalf & Eddy

CIV4288 Water Treatment Topic	Section Title	start page	end page
	2-1 WASTEWATER CHARACTERIZATION	60	60
	2-2 SAMPLING AND ANALYTICAL PROCEDURES	61	63
	2-6 AGGREGATE ORGANIC CONSTITUENTS	114	120
Wastewater Flows and Constituents (Week 5)	3-1 WASTEWATER SOURCES AND FLOWRATES	185	200
	3-3 ANALYSIS OF WASTEWATER FLOWRATE DATA	208	214
	3-4 ANALYSIS OF WASTEWATER CONSTITUENTS	214	226
	3-6 SELECTION OF DESIGN FLOWRATES AND MASS LOADINGS	232	234
	3-7 FLOW AND CONSTITUENT LOAD EQUALIZATION	241	247
	4-4 ELEMENTS OF PROCESS DESIGN	291	295
Preliminary Treatment (Week 6)	5-1 SCREENING	310	318
	5-5 Grit Removal	365	379
Primary Treatment (Week 7)	5-6 PRIMARY SEDIMENTATION	382	398
	7-1 OVERVIEW OF BIOLOGICAL WASTEWATER TREATMENT	555	561
	7-4 BACTERIAL GROWTH, ENERGETICS, AND DECAY	573	579
Secondary Treatment- Activated Sludge (Week 8)	7-5 MICROBIAL GROWTH KINETICS	588	596
	7-6 MODELING SUSPENDED GROWTH TREATMENT PROCESSES	597	610
	8-1 INTRODUCTION TO THE ACTIVATED SLUDGE PROCESS	700	704
	8-3 FUNDAMENTALS OF PROCESS SELECTION, DESIGN, AND CONTROL	717	733
	8-5 ACTIVATED SLUDGE PROCESS DESIGN CONSIDERATIONS	742	752
Secondary Treatment- Trickling Filters (Week 9)	9-1 INTRODUCTION TO ATTACHED GROWTH PROCESSES	943	947
	9-2 NONSUBMERGED ATTACHED GROWTH PROCESSES	947	954
Sludge Handling (Week 9)	13-1 SLUDGE SOURCES, CHARACTERISTICS, AND QUANTITIES	1453	1461
	13-3 SLUDGE PROCESSING FLOW DIAGRAMS	1466	1467
Resource Recovery (Week 10)	13-9 ANAEROBIC DIGESTION	1502	1512

Please note: Page numbers in this PDF correspond to page numbers from original textbook.

Click the section titles above to skip directly to the appropriate page.

FIFTH EDITION

Wastewater Engineering

Treatment and Resource Recovery



METCALF & EDDY | AECOM

Wastewater Engineering

Treatment and Resource Recovery

Fifth Edition

This page intentionally left blank

Wastewater Engineering

Treatment and Resource Recovery

Fifth Edition

Metcalf & Eddy | AECOM

Revised by

George Tchobanoglous

Professor Emeritus of Civil and
Environmental Engineering
University of California at Davis

H. David Stensel

Professor of Civil and Environmental
Engineering
University of Washington, Seattle

Ryujiro Tsuchihashi

Wastewater Technical Leader, AECOM

Franklin Burton

Consulting Engineer
Los Altos, CA

Contributing Authors:

Mohammad Abu-Orf

North America Biosolids Practice
Leader, AECOM

Gregory Bowden

Wastewater Technical Leader, AECOM

William Pfrang

Wastewater Treatment Technology
Leader, AECOM

**Mc
Graw
Hill
Education**



WASTEWATER ENGINEERING: TREATMENT AND RESOURCE RECOVERY, FIFTH EDITION

Published by McGraw-Hill Education, 2 Penn Plaza, New York, NY 10121. Copyright © 2014 by McGraw-Hill Education. All rights reserved. Printed in the United States of America. Previous editions © 2003, 1991, and 1979. No part of this publication may be reproduced or distributed in any form or by any means, or stored in a database or retrieval system, without the prior written consent of McGraw-Hill Education, including, but not limited to, in any network or other electronic storage or transmission, or broadcast for distance learning.

Some ancillaries, including electronic and print components, may not be available to customers outside the United States.

This book is printed on acid-free paper.

1 2 3 4 5 6 7 8 9 0 QVR/QVR 1 0 9 8 7 6 5 4 3

ISBN 978-0-07-340118-8
MHID 0-07-340118-8

Senior Vice President, Products & Markets: *Kurt L. Strand*
Vice President, General Manager: *Marty Lange*
Vice President, Content Production & Technology Services: *Kimberly Meriwether David*
Global Brand Manager: *Raghothaman Srinivasan*
Executive Brand Manager: *Bill Stenquist*
Executive Marketing Manager: *Curt Reynolds*
Development Editor: *Lorraine Buczek*
Director, Content Production: *Terri Schiesl*
Senior Project Manager: *Lisa A. Bruford*
Buyer: *Jennifer Pickel*
Media Project Manager: *Prashanthi Nadipalli*
Cover Designer: *Studio Montage, St. Louis, MO*
Cover Image: *Aerial view Blue Plains Advanced Wastewater Treatment Plant, Washington, DC*
Cover Image Credit: *Courtesy DC Water*
Photographs: *All of the photographs for this textbook were taken by George Tchobanoglous, unless otherwise noted.*
Compositor: *RPK Editorial Services, Inc.*
Typeface: *10/12 Times*
Printer: *Quad/Graphics*

All credits appearing on page or at the end of the book are considered to be an extension of the copyright page.

Library of Congress Cataloging-in-Publication Data

The Internet addresses listed in the text were accurate at the time of publication. The inclusion of a website does not indicate an endorsement by the authors or McGraw-Hill Education, and McGraw-Hill Education does not guarantee the accuracy of the information presented at these sites.

About the Authors

George Tchobanoglous is Professor Emeritus in the Department of Civil and Environmental Engineering at the University of California, Davis. He received a B.S. degree in civil engineering from the University of the Pacific, an M.S. degree in sanitary engineering from the University of California at Berkeley, and a Ph.D. from Stanford University in 1969. Dr. Tchobanoglous' research interests are in the areas of wastewater treatment and reuse, wastewater filtration, UV disinfection, aquatic wastewater management systems, wastewater management for small and decentralized wastewater management systems, and solid waste management. He has authored or co-authored over 500 technical publications including 22 textbooks and 8 reference works. The textbooks are used in more than 225 colleges and universities, by practicing engineers, and in universities worldwide both in English and in translation. His books are famous for successfully bridging the gap between academia and the day-to-day world of the engineer. He is a Past President of the Association of Environmental Engineers and Science Professors. Among his many honors, in 2003 Professor Tchobanoglous received the Clarke Prize from the National Water Research Institute. In 2004, he received the Distinguished Service Award for Research and Education in Integrated Waste Management from the Waste-To-Energy Research and Technology Council. In 2004, he was also inducted into the National Academy of Engineering. In 2005, he was awarded an honorary Doctor of Engineering from the Colorado School of Mines. In 2007, he received the Frederick George Pohland Medal awarded by AAEE and AEESP. In 2012 he was made a WEF Fellow. He is a registered Civil Engineer in California.

H. David Stensel is a Professor in the Civil and Environmental Engineering Department at the University of Washington, Seattle, WA. Prior to his academic positions, he spent 10 years in practice developing and applying industrial and municipal wastewater treatment processes. He received a B.S. degree in civil engineering from Union College, Schenectady, NY, and M.E. and Ph.D. degrees in environmental engineering from Cornell University. His principal research interests are in the areas of wastewater treatment, biological nutrient removal, sludge processing methods, resource recovery, and biodegradation of micropollutants. He is a Past Chair of the Environmental Engineering Division of ASCE, has served on the board of the Association of Environmental Engineering Professors and on various committees for ASCE and the Water Environment Federation. He has authored or coauthored over 150 technical publications and a textbook on biological nutrient removal. Research recognition honors include the ASCE Rudolf Hering Medal, the Water Environment Federation Harrison Prescott Eddy Medal twice, and the Bradley Gascoigne Medal. In 2013, he received the Frederick George Pohland Medal awarded by AAEE and AEESP. He is a registered professional engineer, a diplomate in the American Academy of Environmental Engineers and a life member of the American Society of Civil Engineers and the Water Environment Federation.

Ryujiro Tsuchihashi is a technical leader with AECOM. He received his B.S. and M.S. in civil and environmental engineering from Kyoto University, Japan, and a Ph.D. in environmental engineering from the University of California, Davis. The areas of his expertise include wastewater/water reclamation process evaluation and design, evaluation and assessment of water reuse systems, biological nutrient removal, and evaluation of greenhouse gas emission

reduction from wastewater treatment processes. He was a co-author of the textbook “Water Reuse: Issues, Technologies and Applications,” a companion textbook to this textbook. He is a technical practice coordinator for AECOM’s water reuse leadership team. Ryujiro Tsuchihashi is a member of the Water Environment Federation, American Society of Civil Engineer, and International Water Association, and has been an employee of AECOM for 10 years, during which he has worked on various projects in the United States, Australia, Jordan, and Canada.

Franklin Burton served as vice president and chief engineer of the western region of Metcalf & Eddy in Palo Alto, California for 30 years. He retired from Metcalf & Eddy in 1986 and has been in private practice in Los Altos, California, specializing in treatment technology evaluation, facilities design review, energy management, and value engineering. He received his B.S. in mechanical engineering from Lehigh University and an M.S. in civil engineering from the University of Michigan. He was co-author of the third and fourth editions of the Metcalf & Eddy textbook “Wastewater Engineering: Treatment and Reuse.” He has authored over 30 publications on water and wastewater treatment and energy management in water and wastewater applications. He is a registered civil engineer in California and is a life member of the American Society of Civil Engineers, American Water Works Association, and Water Environment Federation.

Mohammad Abu-Orf is AECOM’s North America biosolids practice leader and wastewater director. He received his B.S. in civil engineering from Birzeit University, West Bank, Palestine and received his M.S. and Ph.D. in civil and environmental engineering from the University of Delaware. He worked with Siemens Water Technology and Veolia Water as biosolids director of research and development. He is the main inventor on five patents and authored and co-authored more than 120 publications focusing on conditioning, dewatering, stabilization and energy recovery from biosolids. He was awarded first place for Ph.D. in the student paper competition by the Water Environment Federation for two consecutive years in 1993 and 1994. He coauthored manuals of practice and reports for the Water Environment Research Foundation. He served as an editor of the Specialty Group for Sludge Management of the International Water Association for six years and served on the editorial board of the biosolids technical bulletin of the Water Environment Federation. Mohammad Abu-Orf has been an employee of AECOM for 6 years.

Gregory Bowden is a technical leader with AECOM. He received his B.S. in chemical engineering from Oklahoma State University and a Ph.D. in chemical engineering from the University of Texas at Austin. He worked for Hoechst Celanese (Celanese AG) for 10 years as a senior process engineer, supporting wastewater treatment facility operations at chemical production plants in North America. He also worked as a project manager in the US Filter/Veolia North American Technology Center. His areas of expertise include industrial wastewater treatment, biological and physical/chemical nutrient removal technologies and biological process modeling. Greg Bowden is a member of the Water Environment Federation and has been an AECOM employee for 9 years.

William Pfrang is a Vice-President of AECOM and Technical Director of their Metro-New York Water Division. He began his professional career with Metcalf & Eddy, Inc., as a civil engineer in 1968. During his career, he has specialized in municipal wastewater treatment plant design including master planning, alternative process assessments, conceptual, and detailed design. Globally, he has been the lead engineer for wastewater treatment projects in the United States, Southeast Asia, South America, and the Middle East. He received his B.S. and M.S. in civil engineering from Northeastern University. He is a registered professional engineer, a member of the American Academy of Environmental Engineers, and the Water Environment Federation. William Pfrang has been an employee of the firm for over 40 years.

Contents

<i>About the Authors</i>	v
<i>Preface</i>	xxiii
<i>Acknowledgments</i>	xxvii
<i>Foreword</i>	xxix
1 Introduction to Wastewater Treatment and Process Analysis 1	
1-1	Evolution of Wastewater Treatment 4 <i>Treatment Objectives</i> 5 <i>Current Health and Environmental Concerns</i> 5 <i>Sustainability Considerations</i> 5
1-2	Evolution of Regulations of Significance to Wastewater Engineering 6 <i>Establishment of Environmental Protection Agency</i> 6 <i>Important Federal Regulations</i> 6 <i>Other Federal Regulations</i> 9 <i>State and Regional Regulations</i> 9
1-3	Characteristics of Wastewater 9 <i>Sources of Wastewater</i> 9 <i>Types of Collection Systems</i> 9 <i>Wastewater Constituents</i> 10
1-4	Classification of Wastewater Treatment Methods 10 <i>Physical Unit Processes</i> 10 <i>Chemical Unit Processes</i> 12 <i>Biological Unit Processes</i> 12
1-5	Application of Treatment Methods 12 <i>Wastewater Processing</i> 12 <i>Residuals Processing</i> 13 <i>Typical Treatment Process Flow Diagrams</i> 13
1-6	Status of Wastewater Treatment in the United States 17 <i>Recent Survey Results</i> 18 <i>Trends</i> 18
1-7	Introduction to Process Analysis 19 <i>Mass-Balance Analysis</i> 19 <i>Application of the Mass-Balance Analysis</i> 21
1-8	Reactors Used in Wastewater Treatment 22 <i>Types of Reactors</i> 22 <i>Hydraulic Characteristics of Reactors</i> 24 <i>Application of Reactors</i> 25
1-9	Modeling Ideal Flow in Reactors 26 <i>Ideal Flow in Complete-Mix Reactor</i> 26 <i>Ideal Plug-Flow Reactor</i> 27
1-10	Introduction to Process Kinetics 29 <i>Types of Reactions</i> 29 <i>Rate of Reaction</i> 30 <i>Specific Reaction Rate</i> 31 <i>Effects of Temperature on Reaction Rate Coefficients</i> 31 <i>Reaction Order</i> 33 <i>Rate Expressions Used in Wastewater Treatment</i> 34 <i>Analysis of Reaction Rate Coefficients</i> 39
1-11	Introduction to Treatment Process Modeling 42 <i>Batch Reactor with Reaction</i> 43 <i>Complete-Mix Reactor with Reaction</i> 43 <i>Complete-Mix Reactors in Series with Reaction</i> 44 <i>Ideal Plug-Flow Reactor with Reaction</i> 47 <i>Comparison of Complete-Mix and Plug-Flow Reactors with Reaction</i> 48 <i>Plug-Flow Reactor with Axial Dispersion and Reaction</i> 50 <i>Other Reactor Flow Regimes and Reactor Combinations</i> 51 <i>Problems and Discussion Topics</i> 53
2 Wastewater Characteristics 57	
2-1	Wastewater Characterization 60 <i>Wastewater Properties and Constituents</i> 60 <i>Constituents of Concern in Wastewater Treatment</i> 60
2-2	Sampling and Analytical Procedures 60 <i>Sampling</i> 63 <i>Methods of Analysis</i> 65 <i>Units of Expression for Physical and Chemical Parameters</i> 66 <i>Useful Chemical Relationships</i> 66

2-3	Physical Properties 73 <i>Sources of Physical Properties</i> 73 Solids 73 <i>Particle Size and Particle Size Measurement</i> 76 <i>Particle Size Distribution</i> 80 <i>Nanoparticles and Nanocomposites</i> 83 Turbidity 83 <i>Relationship Between Turbidity and TSS</i> 85 Color 85 <i>Absorption/Transmittance</i> 85 Temperature 87 <i>Thermal Energy Content of Wastewater</i> 89 Conductivity 89 <i>Density, Specific Gravity, and Specific Weight</i> 89	<i>Unregulated Trace Organic Compounds</i> 133 <i>Analysis of Individual Organic Compounds</i> 133
2-4	Inorganic Nonmetallic Constituents 90 <i>Sources of Inorganic Nonmetallic Constituents</i> 90 pH 90 Chlorides 91 Alkalinity 92 Nitrogen 92 Phosphorus 96 Sulfur 97 Gases 98 Odors 103	2-8 Radionuclides in Wastewater 136 <i>Sources of Radionuclide</i> 137 <i>Units of Expression</i> 137 <i>Description of Isotopes Found in Wastewater and Sludge</i> 137 <i>Treatment Technologies for the Removal of Radionuclides</i> 137
2-5	Metallic Constituents 111 <i>Sources of Metallic Constituents</i> 112 Importance of Metals 113 <i>Sampling and Methods of Analysis</i> 114 <i>Typical Effluent Discharge Limits for Metals</i> 114	2-9 Biological Constituents 139 <i>Sources of Microorganisms in Wastewater</i> 140 <i>Enumeration and Identification of Microorganisms</i> 144 <i>Pathogenic Organisms and Prions</i> 151 <i>Evolving Pathogenic Microorganisms</i> 161
2-6	Aggregate Organic Constituents 114 <i>Sources of Aggregate Organic Constituents</i> 114 <i>Measurement of Organic Content</i> 114 <i>Biochemical Oxygen Demand (BOD)</i> 115 <i>Total and Soluble Chemical Oxygen Demand (COD and SCOD)</i> 123 <i>Total and Dissolved Organic Carbon (TOC and DOTC)</i> 123 <i>UV-Absorbing Organic Constituents</i> 124 <i>Theoretical Oxygen Demand (ThOD)</i> 125 <i>Interrelationships between BOD, COD, and TOC</i> 125 Oil and Grease 127 Surfactants 128 <i>Chemical Energy in Wastewater and Biosolids</i> 129	2-10 Toxicity 161 <i>Sources of Toxicity</i> 161 <i>Evolution and Application of Toxicity Testing</i> 162 <i>Toxicity Testing</i> 163 <i>Analysis of Toxicity Test Results</i> 165 <i>Application of Toxicity Test Results</i> 167 <i>Identification of Toxicity Components</i> 169 Problems and Discussion Topics 171
3-1	3 Wastewater Flowrates and Constituent Loadings 183	
3-2	Wastewater Sources and Flowrates 185 <i>Municipal Uses of Water</i> 185 <i>Domestic Wastewater Sources and Flowrates</i> 186 <i>Strategies for Reducing Interior Water Use and Wastewater Flowrates</i> 189 <i>Water Use in Other Parts of the World</i> 194 <i>Sources and Rates of Industrial (Nondomestic) Wastewater Flows</i> 194 <i>Variations in Wastewater Flowrates</i> 195 <i>Long-Term Multiyear Variations Due to Conservation</i> 198 <i>Impact of Water Conservation on Future Planning</i> 200	
3-3	Impact of Collection System on Wastewater Flowrates 200 <i>Infiltration/Inflow</i> 200 <i>Inflow into Collection Systems</i> 202 <i>Exfiltration from Collection Systems</i> 204 <i>Combined Collection System Flowrates</i> 205 <i>Direct Measurement of Combined Sewer Flowrates and Wastewater Characteristics</i> 207 <i>Calculation of Combined Sewer Flowrates</i> 207	

3-3	Analysis of Wastewater Flowrate Data 208 <i>Statistical Analysis of Flowrate Data</i> 208 <i>Developing Design Parameters from Flowrate Data</i> 211 <i>Observed Variability in Influent Flowrates</i> 212	<i>Owner Needs</i> 270 <i>Environmental Considerations</i> 270 <i>Compatibility with Existing Facilities</i> 271 <i>Energy and Resource Requirements</i> 271 <i>Cost Considerations</i> 272 <i>Other Design Considerations</i> 273
3-4	Analysis of Wastewater Constituents 214 <i>Wastewater Constituents Discharged By Individuals</i> 214 <i>Constituent Concentrations Based on Individual Mass Discharges</i> 218 <i>Mineral Increase Resulting from Water Use</i> 218 <i>Composition of Wastewater in Collection Systems</i> 219 <i>Variations in Constituent Concentrations</i> 219 <i>Statistical Analysis of Constituent Concentrations</i> 225 <i>Observed Variability in Influent Constituent Concentrations</i> 225	4-2 Considerations in Process Selection 274 <i>Important Factors in Process Selection</i> 274 <i>Process Selection Based on Reaction Kinetics</i> 276 <i>Process Selection Based On Mass Transfer</i> 277 <i>Process Design Based on Loading Criteria</i> 277 <i>Bench-Scale Tests and Test-Bed Pilot-Scale Studies</i> 277 <i>Wastewater Discharge Permit Requirements</i> 278
3-5	Analysis of Constituent Mass Loading Data 226 <i>Simple Average</i> 226 <i>Flow-Weighted Average</i> 226 <i>Mass Loadings</i> 229 <i>Effect of Mass Loading Variability on Treatment Plant Performance</i> 231	4-3 Treatment Process Reliability and Selection of Design Values 279 <i>Variability in Wastewater Treatment</i> 280 <i>Selection of Process Design Parameters to Meet Discharge Permit Limits</i> 286 <i>Performance of Combined Processes</i> 289
3-6	Selection of Design Flowrates and Mass Loadings 232 <i>Design Flowrates</i> 234 <i>Design Mass Loadings</i> 240	4-4 Elements of Process Design 291 <i>Design Period</i> 291 <i>Treatment Process Flow Diagrams</i> 291 <i>Process Design Criteria</i> 291 <i>Preliminary Sizing</i> 292 <i>Solids Balance</i> 293 <i>Plant Layout</i> 294 <i>Plant Hydraulics</i> 295 <i>Energy Management</i> 296
3-7	Flow and Constituent Load Equalization 241 <i>Description/Application of Flow Equalization</i> 242 <i>The Benefits of Flow Equalization</i> 243 <i>Design Considerations</i> 243 <i>Equalization of Constituent Mass Loading Rates</i> 253 <i>Equalization of Sludge and Biosolids Processing Return Flows</i> 253 Problems and Discussion Topics 254	4-5 Implementation of Wastewater Management Programs 297 <i>Facilities Planning</i> 297 <i>Design</i> 297 <i>Value Engineering</i> 298 <i>Construction</i> 298 <i>Facilities Startup and Operation</i> 299
4	Wastewater Treatment Process Selection, Design, and Implementation 263	4-6 Financing 299 <i>Long-Term Municipal Debt Financing</i> 299 <i>Non-Debt Financing</i> 300 <i>Leasing</i> 300 <i>Privatization</i> 300 Problems and Discussion Topics 300
4-1	Planning for New and Upgrading Existing Wastewater Treatment Plants 265 <i>Need to Upgrade Existing Wastewater Treatment Plants</i> 265 <i>Planning for New Wastewater Treatment Plants</i> 266 <i>Treatment Process Design Considerations</i> 266	5 Physical Unit Processes 305
5-1	Screening 310 <i>Classification of Screens</i> 310 <i>Screenings Characteristics and Quantities</i> 311 <i>Coarse Screens (Bar Racks)</i> 312	

<p><i>Fine Screens</i> 318 <i>Microscreens</i> 323 <i>Screenings Handling, Processing, and Disposal</i> 324</p> <p>5-2 <i>Coarse Solids Reduction</i> 325 <i>Comminutors</i> 325 <i>Macerators</i> 326 <i>Grinders</i> 327 <i>Design Considerations</i> 327</p> <p>5-3 <i>Mixing and Flocculation</i> 327 <i>Continuous Rapid Mixing in Wastewater Treatment</i> 328 <i>Continuous Mixing in Wastewater Treatment</i> 329 <i>Energy Dissipation in Mixing and Flocculation</i> 330 <i>Time Scale in Mixing</i> 332 <i>Types of Mixers Used for Rapid Mixing in Wastewater Treatment</i> 332 <i>Types of Mixers Used for Maintaining Solids in Suspension in Wastewater Treatment and Chemical Mixing</i> 335 <i>Types of Mixers Used for Flocculation in Wastewater Treatment</i> 338 <i>Types of Mixers Used for Continuous Mixing in Wastewater Treatment</i> 341 <i>New Developments in Mixing Technology</i> 344</p> <p>5-4 <i>Gravity Separation Theory</i> 344 <i>Description</i> 345 <i>Particle Settling Theory</i> 346 <i>Discrete Particle Settling</i> 350 <i>Flocculent Particle Settling</i> 354 <i>Inclined Plate and Tube Settling</i> 356 <i>Hindered (Zone) Settling</i> 360 <i>Compression Settling</i> 364 <i>Gravity Separation in an Accelerated Flow Field</i> 364</p> <p>5-5 <i>Grit Removal</i> 365 <i>Wastewater Grit Characteristics</i> 366 <i>Grit Separators for Wastewater</i> 370 <i>Grit Separators for Combined Wastewater and Stormwater</i> 379 <i>Grit Washing</i> 380 <i>Grit Drying</i> 382 <i>Disposal of Grit</i> 382 <i>Solids (Sludge) Degritting</i> 382</p> <p>5-6 <i>Primary Sedimentation</i> 382 <i>Description</i> 383 <i>Sedimentation Tank Performance</i> 391 <i>Design Considerations</i> 393</p>	<p><i>Characteristics and Quantities of Solids (Sludge) and Scum</i> 397</p> <p>5-7 <i>High-Rate Clarification</i> 398 <i>Enhanced Particle Flocculation</i> 398 <i>Analysis of Ballasted Particle Flocculation and Settling</i> 399 <i>Process Application</i> 401</p> <p>5-8 <i>Flotation</i> 403 <i>Description</i> 404 <i>Design Considerations for Dissolved-Air Flotation Systems</i> 406</p> <p>5-9 <i>New Approaches for Primary Treatment</i> 408 <i>Microscreening of Raw Wastewater</i> 409 <i>Charged Bubble Flotation</i> 409 <i>Primary Effluent Filtration</i> 410</p> <p>5-10 <i>Gas Liquid Mass Transfer</i> 411 <i>Historical Development of Gas Transfer Theories</i> 411 <i>The Two-Film Theory of Gas Transfer</i> 412 <i>Absorption of Gases Under Turbulent Conditions</i> 415 <i>Absorption of Gases Under Quiescent Conditions</i> 417 <i>Desorption (Removal) of Gases</i> 418</p> <p>5-11 <i>Aeration Systems</i> 419 <i>Oxygen Transfer</i> 419 <i>Evaluation of Alpha (α) Correction Factor</i> 421 <i>Types of Aeration Systems</i> 424 <i>Diffused-Air Aeration</i> 424 <i>Mechanical Aerators</i> 436 <i>Energy Requirement for Mixing in Aeration Systems</i> 439 <i>Generation and Dissolution of High-Purity Oxygen</i> 440 <i>Postaeration</i> 443 <i>Problems and Discussion Topics</i> 448</p>	<h2>6 Chemical Unit Processes 455</h2> <p>6-1 <i>Role of Chemical Unit Processes in Wastewater Treatment</i> 458 <i>Applications of Chemical Unit Processes</i> 458 <i>Considerations in the Use of Chemical Unit Processes</i> 459</p> <p>6-2 <i>Fundamentals of Chemical Coagulation</i> 460 <i>Basic Definitions</i> 461 <i>Nature of Particles in Wastewater</i> 461 <i>Development and Measurement of Surface Charge</i> 462</p>
---	---	---

	<i>Particle-Particle Interactions</i>	463		<i>Photolysis Processes</i>	522
	<i>Particle Destabilization and Aggregation with Polyelectrolytes</i>	466		<i>Basic Considerations for Photolysis Processes</i>	522
	<i>Particle Destabilization and Removal with Hydrolyzed Metal Ions</i>	468		<i>Photolysis Process Limitations</i>	528
6-3	Chemical Precipitation for Improved Plant Performance	473	6-10	Chemical Neutralization, Scale Control, and Stabilization	529
	<i>Chemical Reactions in Wastewater Precipitation Applications</i>	474		<i>pH Adjustment</i>	529
	<i>Chemically Enhanced Primary Treatment (CEPT)</i>	477		<i>Analysis of Scaling Potential</i>	530
	<i>Independent Physical-Chemical Treatment</i>	478		<i>Scale Control</i>	535
	<i>Estimation of Sludge Quantities from Chemical Precipitation</i>	479		<i>Stabilization</i>	536
6-4	Chemical Phosphorus Removal	481	6-11	Chemical Storage, Feeding, Piping, and Control Systems	536
	<i>Chemicals Used for Phosphorus Removal</i>	481		<i>Chemical Storage and Handling</i>	536
	<i>Phosphorus Removal from the Liquid Stream with Metal Salts</i>	487		<i>Dry Chemical Feed Systems</i>	538
	<i>Phosphorus Removal from the Liquid Stream with Calcium</i>	489		<i>Liquid Chemical Feed Systems</i>	542
	<i>Strategies for Chemical Phosphorus Removal</i>	491		<i>Gas Chemical Feed Systems</i>	542
6-5	Chemical Formation of Struvite for Ammonium and Phosphorus Removal	492		<i>Initial Chemical Mixing</i>	543
	<i>Chemistry of Struvite Formation</i>	493		Problems and Discussion Topics	544
	<i>Control and/or Mitigation Measures for the Formation of Struvite</i>	496			
	<i>Enhanced Struvite Formation for Nutrient Removal</i>	496			
6-6	Chemical Precipitation for Removal of Heavy Metals and Dissolved Substances	498	7	Fundamentals of Biological Treatment	551
	<i>Precipitation Reactions</i>	498			
	<i>Co-precipitation with Phosphorus</i>	500	7-1	Overview of Biological Wastewater Treatment	555
6-7	Conventional Chemical Oxidation	501		<i>Objectives of Biological Treatment</i>	555
	<i>Applications for Conventional Chemical Oxidation</i>	501		<i>Role of Microorganisms in Wastewater Treatment</i>	555
	<i>Oxidants Used in Chemical Oxidation Processes</i>	501			
	<i>Fundamentals of Chemical Oxidation</i>	503	7-2	Composition and Classification of Microorganisms	561
	<i>Chemical Oxidation of Organic Constituents</i>	508		<i>Cell Components</i>	562
	<i>Chemical Oxidation of Ammonium</i>	508		<i>Cell Composition</i>	564
	<i>Chemical Oxidation Process Limitations</i>	510		<i>Environmental Factors</i>	564
6-8	Advanced Oxidation	510		<i>Microorganism Identification and Classification</i>	565
	<i>Applications for Advanced Oxidation</i>	510		<i>Use of Molecular Tools</i>	568
	<i>Processes for Advanced Oxidation</i>	513	7-3	Introduction to Microbial Metabolism	571
	<i>Basic Considerations for Advanced Oxidation Processes</i>	517		<i>Carbon and Energy Sources for Microbial Growth</i>	571
	<i>Advanced Oxidation Process Limitations</i>	520		<i>Nutrient and Growth Factor Requirements</i>	573
6-9	Photolysis	521	7-4	Bacterial Growth, Energetics, and Decay	573
	<i>Applications for Photolysis</i>	521		<i>Bacterial Reproduction</i>	574

	<i>Biomass Synthesis Yields for Different Growth Conditions</i> 587	
	<i>Biomass Decay</i> 587	
	<i>Observed versus Synthesis Yield</i> 588	
7-5	Microbial Growth Kinetics 588	
	<i>Microbial Growth Kinetics Terminology</i> 589	
	<i>Rate of Utilization of Soluble Substrates</i> 589	
	<i>Other Rate Expressions for Soluble Substrate Utilization</i> 591	
	<i>Rate of Soluble Substrate Production from Biodegradable Particulate Organic Matter</i> 591	
	<i>Net Biomass Growth Rate</i> 592	
	<i>Kinetic Coefficients for Substrate Utilization and Biomass Growth</i> 593	
	<i>Rate of Oxygen Uptake</i> 593	
	<i>Effects of Temperature</i> 594	
	<i>Total Volatile Suspended Solids and Active Biomass</i> 594	
	<i>Net Biomass Yield and Observed Yield</i> 595	
7-6	Modeling Suspended Growth Treatment Processes 597	
	<i>Description of Suspended Growth Treatment Processes</i> 597	
	<i>Solids Retention Time</i> 597	
	<i>Biomass Mass Balance</i> 598	
	<i>Substrate Mass Balance</i> 600	
	<i>Mixed Liquor Solids Concentration and Solids Production</i> 600	
	<i>The Observed Yield</i> 603	
	<i>Oxygen Requirements</i> 603	
	<i>Design and Operating Parameters</i> 606	
	<i>Process Performance and Stability</i> 607	
	<i>Modeling Plug-Flow Reactors</i> 609	
7-7	Substrate Removal in Attached Growth Treatment Process 610	
	<i>Biofilm Characteristics</i> 611	
	<i>Biomass Characterization</i> 611	
	<i>Mechanistic Models</i> 612	
	<i>Substrate Flux in Biofilms</i> 612	
	<i>Substrate Mass Balance for Biofilm</i> 613	
	<i>Substrate Flux Limitations</i> 613	
7-8	Aerobic Oxidation 615	
	<i>Process Description</i> 615	
	<i>Microbiology</i> 615	
	<i>Process Operation Issues</i> 616	
	<i>Stoichiometry of Aerobic Biological Oxidation</i> 617	
	<i>Growth Kinetics</i> 617	
	<i>Environmental Factors</i> 618	
7-9	Biological Oxidation of Inorganic Nitrogen 618	
	<i>Process Description</i> 619	
	<i>Microbiology</i> 619	
	<i>Stoichiometry of Biological Nitrification</i> 622	
	<i>Nitification Kinetics</i> 624	
	<i>AOB Kinetics</i> 626	
	<i>NOB Kinetics</i> 627	
	<i>Environmental Factors</i> 628	
7-10	Denitrification 631	
	<i>Process Description</i> 632	
	<i>Microbiology</i> 633	
	<i>Stoichiometry of Biological Denitrification and Denitritation</i> 634	
	<i>Organic Substrate Requirements for Denitrification and Denitritation</i> 635	
	<i>Denitrification Kinetics</i> 637	
	<i>Environmental Factors</i> 640	
7-11	Anaerobic Ammonium Oxidation 640	
	<i>Process Description</i> 640	
	<i>Microbiology</i> 641	
	<i>Anammox Stoichiometry</i> 641	
	<i>Growth Kinetics</i> 644	
	<i>Environmental Factors</i> 645	
7-12	Greenhouse Gas from Biological Nitrogen Transformations 645	
	<i>Source of Nitrous Oxide Emissions</i> 645	
	<i>Nitrous Oxide Production Pathways</i> 646	
7-13	Enhanced Biological Phosphorus Removal 648	
	<i>Process Description</i> 648	
	<i>Processes Occurring in the Anaerobic Zone</i> 650	
	<i>Processes Occurring in a Downstream Aerobic or Anoxic Zone</i> 650	
	<i>Microbiology</i> 651	
	<i>Other Process Considerations for EBPR</i> 652	
	<i>Stoichiometry of Enhanced Biological Phosphorus Removal</i> 653	
	<i>Growth Kinetics</i> 655	
	<i>Environmental Factors</i> 655	
7-14	Anaerobic Fermentation and Oxidation 655	
	<i>Process Description</i> 656	
	<i>Microbiology</i> 657	
	<i>Stoichiometry of Anaerobic Fermentation and Oxidation</i> 659	
	<i>Process Kinetics</i> 660	
	<i>Environmental Factors</i> 663	
7-15	Biological Removal of Toxic and Recalcitrant Organic Compounds 663	
	<i>Development of Biological Treatment Methods</i> 664	

	Aerobic Biodegradation	665	General Process Design Considerations	754
	Abiotic Losses	666	Complete-Mix Activated-Sludge Process	
	Modeling Biotic and Abiotic Losses	669	Design	754
7-16	Biological Removal of Trace Organic Compounds	671	Sequencing Batch Reactor Process Design	771
	Removal of Trace Organic Compounds	672	Staged Activated-Sludge Process Design	782
	Steady-State Fate Model	672	Alternative Processes for BOD Removal and Nitrification	786
7-17	Biological Removal of Heavy Metals	674	Processes for Biological Nitrogen Removal	795
	Problems and Discussion Topics	674	Process Development	796
			Overview of Types of Biological Nitrogen-Removal Processes	797
8	Suspended Growth Biological Treatment Processes	697	General Process Design Considerations	802
8-1	Introduction to the Activated-Sludge Process	700	Preanoxic Denitrification Processes	804
	Historical Development of Activated Sludge Process	701	Postanoxic Denitrification Processes	831
	Basic Process Description	701	Low DO and Cyclic Nitrification/Denitrification Processes	833
	Evolution of the Conventional Activated-Sludge Process	702	Alternative Process Configurations for Biological Nitrogen Removal	838
	Nutrient Removal Processes	706	Denitrification with External Carbon Addition	848
8-2	Wastewater Characterization	707	Process Control and Performance	860
	Key Wastewater Constituents for Process Design	707	Processes for Enhanced Biological Phosphorus Removal	861
	Measurement Methods for Wastewater Characterization	712	Process Development	861
	Recycle Flows and Loadings	716	Overview of Enhanced Biological Phosphorus Removal Processes	862
8-3	Fundamentals of Process Selection, Design, and Control	717	General Process Design Considerations	864
	Overall Considerations in Treatment Process Implementation	717	Operational Factors That Affect Enhanced Biological Phosphorus Removal	878
	Important Factors in Process Selection and Design	717	Enhanced Biological Phosphorus Removal Process Design	880
	Process Control	726	Provision for Chemical Addition	883
	Operational Problems in Activated Sludge Systems with Secondary Clarifiers	732	Process Control and Performance Optimization	884
	Operational Problems with MBR Systems	738		
8-4	Selector Types and Design Consideration	738	Aeration Tank Design for Activated-Sludge Processes	885
	Selector Types and Design Considerations	739	Aeration System	885
	Poor Settling Even with Use of Selector	741	Aeration Tanks and Appurtenances	886
8-5	Activated Sludge Process Design Considerations	742		
	Steady-State Design Approach	742	Analysis of Liquid-Solids Separation for Activated-Sludge Processes with Clarifiers	889
	Use of Simulation Model	744	Solids Separation by Secondary Clarifiers	889
	Model Matrix Format, Components, and Reactions	747	Assessing Sludge Thickening Characteristics	891
	Other Simulation Model Applications	751	Clarifier Design Based on Solids Flux Analysis	893
8-6	Processes for Bod Removal and Nitrification	752	Clarifier Design Based on State Point Analysis	900
	Overview of BOD Removal and Nitrification Processes	752		
			Design Considerations for Secondary Clarifiers	906
			Types of Sedimentation Tank	906
			Sidewater Depth	910

	<i>Flow Distribution</i> 910	<i>Design of Physical Facilities</i> 1003
	<i>Tank Inlet Design</i> 910	<i>IFAS Process Design Analysis</i> 1005
	<i>Weir Placement and Loading</i> 912	<i>BOD and Nitrification Design Approach</i> 1008
	<i>Scum Removal and Management</i> 912	
8-12	Solids Separation for Membrane Bioreactors 913	Moving Bed Biofilm Reactor (MBBR) 1015
	<i>Design Parameter</i> 913	<i>Background</i> 1015
	<i>Membrane Properties</i> 914	<i>MBBR Process Applications</i> 1016
	<i>Membrane Design and Operating Characteristics</i> 917	<i>MBBR Process Advantages and Disadvantages</i> 1016
	<i>Membrane Usage</i> 917	<i>Design of Physical Facilities</i> 1019
	<i>Membrane Fouling Issues</i> 917	<i>MBBR Process Design Analysis</i> 1020
	Problems and Discussion Topics 919	<i>BOD Removal and Nitrification Design</i> 1021
9	Attached Growth and Combined Biological Treatment Processes 941	
9-1	Introduction to Attached Growth Processes 943	Submerged Aerobic Attached Growth Processes 1026
	<i>Types of Attached Growth Processes</i> 943	<i>Process Development</i> 1026
	<i>Mass Transfer Limitations in Attached Growth Processes</i> 947	<i>Process Applications</i> 1027
9-2	Nonsubmerged Attached Growth Processes 947	<i>Process Advantages and Disadvantages</i> 1027
	<i>General Process Description</i> 947	<i>Design of Physical Facilities</i> 1029
	<i>Trickling Filter Classification and Applications</i> 950	<i>BAF Process Design Analysis</i> 1031
	<i>Advantages and Disadvantages of Trickling Filters</i> 953	<i>FBBR Process Design Analysis</i> 1034
	<i>Physical Facilities for Trickling Filters</i> 954	
	<i>Design Considerations for Physical Facilities</i> 957	Attached Growth Denitrification Processes 1034
	<i>Process Design Considerations for BOD Removal</i> 968	<i>Process Development</i> 1034
	<i>Process Analysis for BOD Removal</i> 972	<i>Description and Application of Attached Growth Denitrification Processes</i> 1035
	<i>Process Analysis for Nitrification</i> 978	<i>Process Design Analysis of Postanoxic Attached Growth Denitrification</i> 1037
9-3	Sequential Combined Trickling Filter and Suspended Solids Processes 987	<i>Operational Considerations for Postanoxic Attached Growth Denitrification</i> 1041
	<i>Process Development</i> 987	
	<i>Process Applications</i> 987	Emerging Biofilm Processes 1045
	<i>Trickling Filter/Solids Contact Process</i> 988	<i>Membrane Biofilm Reactors</i> 1045
	<i>Trickling Filter/Activated Sludge Process</i> 990	<i>Biofilm Airlift Reactors</i> 1046
	<i>Series Trickling-Filter Activated-Sludge Process</i> 997	<i>Aerobic Granules Reactor</i> 1046
9-4	Integrated Fixed-Film Activated Sludge Process 997	Problems and Discussion Topics 1046
	<i>Process Development</i> 998	
	<i>Process Applications</i> 1000	
	<i>IFAS Process Advantages and Disadvantages</i> 1002	
10	Anaerobic Suspended and Attached Growth Biological Treatment Processes 1059	
10-1	The Rationale for Anaerobic Treatment 1061	
	<i>Advantages of Anaerobic Treatment Processes</i> 1061	
	<i>Disadvantages of Anaerobic Treatment Processes</i> 1062	
	<i>Summary Assessment</i> 1063	
10-2	Development of Anaerobic Technologies 1063	
	<i>Historical Developments in Liquefaction</i> 1063	
	<i>Treatment of Wastewater Sludges</i> 1065	
	<i>Treatment of High Strength Wastes</i> 1066	
	<i>Future Developments</i> 1067	

10-3	Available Anaerobic Technologies 1067 <i>Types of Anaerobic Technologies</i> 1067 <i>Application of Anaerobic Technologies</i> 1071	<i>Available Filtration Technologies</i> 1144 <i>Performance of Different Types of Depth Filters</i> 1146 <i>Considerations Related to Design and Operation of Treatment Facilities</i> 1156 <i>Selection of Filtration Technology</i> 1158 <i>Design Considerations for Granular Medium Filters</i> 1161
10-4	Fundamental Considerations in the Application of Anaerobic Treatment Processes 1075 <i>Characteristics of the Wastewater</i> 1075 <i>Pretreatment of Wastewater</i> 1080 <i>Expected Gas Production</i> 1083 <i>Energy Production Potential</i> 1085 <i>Sulfide Production</i> 1088 <i>Ammonia Toxicity</i> 1090	11-6 Surface Filtration 1171 <i>Available Filtration Technologies</i> 1172 <i>Description of the Surface Filtration Process</i> 1175 <i>Performance of Surface Filters</i> 1178 <i>Design Considerations</i> 1180 <i>Pilot Plant Studies</i> 1180
10-5	Design Considerations for Implementation of Anaerobic Treatment Processes 1090 <i>Treatment Efficiency Needed</i> 1091 <i>General Process Design Parameters</i> 1091 <i>Process Implementation Issues</i> 1093	11-7 Membrane Filtration Processes 1181 <i>Membrane Process Terminology</i> 1181 <i>Membrane Process Classification</i> 1182 <i>Membrane Containment Vessels</i> 1185 <i>Operational Modes for Pressurized Configurations</i> 1189 <i>Process Analysis for MF and UF Membranes</i> 1190 <i>Operating Strategies for MF and UF Membranes</i> 1192 <i>Process Analysis for Reverse Osmosis</i> 1193 <i>Membrane Fouling</i> 1198
10-6	Process Design Examples 1095 <i>Upflow Anaerobic Sludge Blanket Process</i> 1095 <i>Anaerobic Contact Process</i> 1103 <i>Use of Simulation Models</i> 1107	<i>Control of Membrane Fouling</i> 1201 <i>Application and Performance of Membranes</i> 1204 <i>Forward Osmosis: An Emerging Membrane Technology</i> 1212 <i>Pilot-Plant Studies for Membrane Applications</i> 1214 <i>Management of Retentate</i> 1215
10-7	Codigestion of Organic Wastes with Municipal Sludge 1108 Problems and Discussion Topics 1109	11-8 Electrodialysis 1217 <i>Description of the Electrodialysis Process</i> 1217 <i>Electrodialysis Reversal</i> 1218 <i>Power Consumption</i> 1220 <i>Operating Considerations</i> 1222 <i>Electrodialysis Versus Reverse Osmosis</i> 1223
11-1	Need for Additional Wastewater Treatment 1120	11-9 Adsorption 1224 <i>Applications for Adsorption</i> 1224 <i>Types of Adsorbents</i> 1224 <i>Fundamentals of Adsorption Processes</i> 1227 <i>Development of Adsorption Isotherms</i> 1227 <i>Adsorption of Mixtures</i> 1232 <i>Adsorption Capacity</i> 1232 <i>Small Scale Column Tests</i> 1240 <i>Analysis of Powdered Activated Carbon Contactor</i> 1243
11-2	Overview of Technologies Used for Removal of Residual Particulate and Dissolved Constituents 1120 <i>Separation Processes Based on Mass Transfer</i> 1120 <i>Transformation Based on Chemical and Biological Processes</i> 1122 <i>Application of Unit Processes for Removal of Residual Constituents</i> 1123	
11-3	Unit Processes for the Removal of Residual Particulate and Dissolved Constituents 1123 <i>Typical Process Flow Diagrams</i> 1124 <i>Process Performance Expectations</i> 1125	
11-4	Introduction to Depth Filtration 1129 <i>Description of the Filtration Process</i> 1129 <i>Filter Hydraulics</i> 1134 <i>Modeling the Filtration Process</i> 1142	
11-5	Depth Filtration: Selection and Design Considerations 1144	

	<i>Activated Sludge-Powdered Activated Carbon Treatment</i> 1244	
	<i>Carbon Regeneration</i> 1245	
	<i>Adsorption Process Limitations</i> 1245	
11-10	<i>Gas Stripping</i> 1245	
	<i>Analysis of Gas Stripping</i> 1245	
	<i>Design of Stripping Towers</i> 1256	
	<i>Air Stripping Applications</i> 1261	
11-11	<i>Ion Exchange</i> 1261	
	<i>Ion Exchange Materials</i> 1262	
	<i>Typical Ion Exchange Reactions</i> 1263	
	<i>Exchange Capacity of Ion Exchange Resins</i> 1264	
	<i>Ion Exchange Chemistry</i> 1266	
	<i>Application of Ion Exchange</i> 1270	
	<i>Operational Considerations</i> 1275	
11-12	<i>Distillation</i> 1275	
	<i>Distillation Processes</i> 1276	
	<i>Performance Expectations in Reclamation Applications</i> 1277	
	<i>Operating Problems</i> 1278	
	<i>Disposal of Concentrated Waste</i> 1278	
	<i>Problems and Discussion Topics</i> 1278	
12	Disinfection Processes 1291	
12-1	Introduction to Disinfectants Used in Wastewater 1294	
	<i>Characteristics for an Ideal Disinfectant</i> 1294	
	<i>Disinfection Agents and Methods</i> 1294	
	<i>Mechanisms Used to Explain Action of Disinfectants</i> 1296	
	<i>Comparison of Disinfectants</i> 1297	
12-2	Disinfection Process Considerations 1297	
	<i>Physical Facilities Used for Disinfection</i> 1297	
	<i>Factors Affecting Performance</i> 1300	
	<i>Development of the CT Concept for Predicting Disinfection Performance</i> 1306	
	<i>Application of the CT Concept to Wastewater Disinfection</i> 1307	
	<i>Performance Comparison of Disinfection Technologies</i> 1308	
12-3	Disinfection with Chlorine 1312	
	<i>Characteristics of Chlorine Compounds</i> 1312	
	<i>Chemistry of Chlorine Compounds</i> 1314	
	<i>Breakpoint Reaction with Chlorine</i> 1316	
	<i>Effectiveness of Free and Combined Chlorine as Disinfectants</i> 1320	
	<i>Measurement and Reporting of Disinfection Process Performance</i> 1322	
	<i>Factors that Affect Disinfection of Wastewater with Chlorine Compounds</i> 1323	
	<i>Modeling the Chlorine Disinfection Process</i> 1328	
	<i>Required Chlorine Dosages for Disinfection</i> 1329	
	<i>Formation and Control of Disinfection Byproducts (DBPs)</i> 1333	
	<i>Environmental Impacts of Disinfection with Chlorine</i> 1336	
12-4	Disinfection with Chlorine Dioxide 1337	
	<i>Characteristics of Chlorine Dioxide</i> 1337	
	<i>Chlorine Dioxide Chemistry</i> 1337	
	<i>Effectiveness of Chlorine Dioxide as a Disinfectant</i> 1338	
	<i>Modeling the Chlorine Dioxide Disinfection Process</i> 1338	
	<i>Required Chlorine Dioxide Dosages for Disinfection</i> 1338	
	<i>Byproduct Formation and Control</i> 1338	
	<i>Environmental Impacts</i> 1339	
12-5	Dechlorination 1339	
	<i>Dechlorination of Treated Wastewater with Sulfur Dioxide</i> 1339	
	<i>Dechlorination of Treated Wastewater with Sodium Based Compounds</i> 1341	
	<i>Dechlorination with Hydrogen Peroxide</i> 1342	
	<i>Dechlorination with Activated Carbon</i> 1342	
	<i>Dechlorination of Chlorine Dioxide with Sulfur Dioxide</i> 1342	
12-6	Design of Chlorination and Dechlorination Facilities 1343	
	<i>Sizing Chlorination Facilities</i> 1343	
	<i>Disinfection Process Flow Diagrams</i> 1344	
	<i>Dosage Control</i> 1347	
	<i>Injection and Initial Mixing</i> 1349	
	<i>Chlorine Contact Basin Design</i> 1349	
	<i>Assessing the Hydraulic Performance of Existing Chlorine Contact Basins</i> 1359	
	<i>Outlet Control and Chlorine Residual Measurement</i> 1365	
	<i>Chlorine Storage Facilities</i> 1365	
	<i>Chemical Containment Facilities</i> 1366	
	<i>Dechlorination Facilities</i> 1366	
12-7	Disinfection with Ozone 1367	
	<i>Ozone Properties</i> 1367	
	<i>Ozone Chemistry</i> 1368	
	<i>Effectiveness of Ozone as a Disinfectant</i> 1369	
	<i>Modeling the Ozone Disinfection Process</i> 1369	
	<i>Required Ozone Dosages for Disinfection</i> 1372	
	<i>Estimation of the CT Value</i> 1372	

12-8	<i>Byproduct Formation and Control</i> 1374 <i>Environmental Impacts of Using Ozone</i> 1374 <i>Other Benefits of Using Ozone</i> 1375 <i>Ozone Disinfection Systems Components</i> 1375 12-9 <i>Other Chemical Disinfection Methods</i> 1378 <i>Peracetic Acid</i> 1379 <i>Use of Peroxone as a Disinfectant</i> 1380 <i>Sequential Chlorination</i> 1381 <i>Combined Chemical Disinfection Processes</i> 1381 12-10 <i>Ultraviolet (UV) Radiation Disinfection</i> 1382 <i>Source of UV Radiation</i> 1383 <i>Types of UV Lamps</i> 1384 <i>UV Disinfection System Configurations</i> 1387 <i>Quartz Sleeve Cleaning Systems</i> 1390 <i>Mechanism of Inactivation by UV Irradiation</i> 1391 <i>Germicidal Effectiveness of UV Irradiation</i> 1393 <i>Estimating UV Dose</i> 1399 <i>Ultraviolet Disinfection Guidelines</i> 1404 <i>Relationship of UV Guidelines to UV System Design</i> 1405 <i>Validation of UV Reactor or System Performance</i> 1405 <i>Factors Effecting UV System Design</i> 1413 <i>Selection and Sizing of a UV Disinfection System</i> 1420 <i>Use of Spot-Check Bioassay to Validate UV System Performance</i> 1422 <i>Troubleshooting UV Disinfection Systems</i> 1426 <i>Environmental Impacts of UV Radiation Disinfection</i> 1428 12-10 <i>Disinfection By Pasteurization</i> 1428 <i>Description of the Pasteurization Process</i> 1428 <i>Thermal Disinfection Kinetics</i> 1429 <i>Germicidal Effectiveness of Pasteurization</i> 1433 <i>Regulatory Requirements</i> 1433 <i>Application of Pasteurization for Disinfection</i> 1433 Problems and Discussion Topics 1434	 13-2 <i>Regulations for the Reuse and Disposition of Sludge in the United States</i> 1461 <i>Land Application</i> 1461 <i>Surface Disposition</i> 1462 <i>Pathogen and Vector Attraction Reduction</i> 1462 <i>Incineration</i> 1463 13-3 <i>Sludge Processing Flow Diagrams</i> 1466 13-4 <i>Sludge and Scum Pumping</i> 1467 <i>Pumps</i> 1467 <i>Headloss Determination</i> 1475 <i>Sludge Piping</i> 1480 13-5 <i>Preliminary Sludge Processing Operations</i> 1481 <i>Grinding</i> 1481 <i>Screening</i> 1482 <i>Degritting</i> 1482 <i>Blending</i> 1483 <i>Storage</i> 1484 13-6 <i>Thickening</i> 1486 <i>Application</i> 1486 <i>Description and Design of Thickeners</i> 1487 13-7 <i>Introduction to Sludge Stabilization</i> 1497 13-8 <i>Alkaline Stabilization</i> 1498 <i>Chemical Reactions in Lime Stabilization</i> 1498 <i>Heat Generation</i> 1499 <i>Application of Alkaline Stabilization Processes</i> 1500 13-9 <i>Anaerobic Digestion</i> 1502 <i>Process Fundamentals</i> 1503 <i>Description of Mesophilic Anaerobic Digestion Processes</i> 1504 <i>Process Design for Mesophilic Anaerobic Digestion</i> 1506 <i>Selection of Tank Design and Mixing System</i> 1512 <i>Methods for Enhancing Sludge Loading and Digester Performance</i> 1520 <i>Gas Production, Collection, and Use</i> 1520 <i>Digester Heating</i> 1525 <i>Advanced Anaerobic Digestion</i> 1530 <i>Sludge Pre-treatment for Anaerobic Digestion</i> 1533 <i>Co-digestion with Other Organic Waste Material</i> 1538 13-10 <i>Aerobic Digestion</i> 1541 <i>Process Description</i> 1542 <i>Conventional Air Aerobic Digestion</i> 1544 <i>Dual Digestion</i> 1549 <i>Autothermal Thermophilic Aerobic Digestion (ATAD)</i> 1549
-------------	---	--

	<i>Improved ATAD Systems</i>	1553
	<i>High-Purity Oxygen Digestion</i>	1553
	Problems and Discussion Topics	1554
14	Biosolids Processing, Resource Recovery and Beneficial Use	1561
14-1	Chemical Conditioning	1564
	<i>Polymers</i>	1564
	<i>Factors Affecting Polymer Conditioning</i>	1565
	<i>Polymer Dosage Determination</i>	1565
	<i>Mixing</i>	1566
	<i>Conditioning Makeup and Feed</i>	1567
14-2	Dewatering	1567
	<i>Overview of Dewatering Technologies</i>	1568
	<i>Centrifugation</i>	1571
	<i>Belt-Filter Press</i>	1574
	<i>Rotary Press</i>	1577
	<i>Screw Press</i>	1580
	<i>Filter Presses</i>	1583
	<i>Electro-Dewatering</i>	1585
	<i>Sludge Drying Beds</i>	1588
	<i>Reed Beds</i>	1592
	<i>Lagoons</i>	1593
14-3	Heat Drying	1593
	<i>Heat-Transfer Methods</i>	1593
	<i>Process Description</i>	1595
	<i>Product Characteristics and Use</i>	1599
	<i>Product Transport and Storage</i>	1600
	<i>Fire and Explosion Hazards</i>	1601
	<i>Air Pollution and Odor Control</i>	1601
14-4	Advanced Thermal Oxidation	1602
	<i>Fundamental Aspects of Complete Combustion</i>	1603
	<i>Multiple-Hearth Incineration</i>	1606
	<i>Fluidized-Bed Incineration</i>	1608
	<i>Energy Recovery from Thermal Oxidation</i>	1610
	<i>Coincineration with Municipal Solid Waste</i>	1611
	<i>Air-Pollution Control</i>	1612
14-5	Composting	1613
	<i>Process Microbiology</i>	1614
	<i>Composting Process Stages</i>	1614
	<i>Composting Process Steps</i>	1614
	<i>Composting Methods</i>	1616
	<i>Design Considerations</i>	1618
	<i>Co-composting with Municipal Solid Wastes</i>	1620
	<i>Public Health and Environmental Issues</i>	1620
14-6	Sludge and Biosolids Conveyance and Storage	1621
	<i>Conveyance Methods</i>	1621
	<i>Storage</i>	1622
14-7	Solids Mass Balances	1623
	<i>Preparation of Solids Mass Balances</i>	1623
	<i>Performance Data for Solids Processing Facilities</i>	1623
	<i>Impact of Return Flows and Loads</i>	1623
14-8	Resource Recovery from Sludges and Biosolids	1636
	<i>Recovery of Nutrients</i>	1637
	<i>Agricultural Land Application</i>	1637
	<i>Non-Agricultural Land Applications</i>	1637
14-9	Energy Recovery from Sludge and Biosolids	1638
	<i>Energy Recovery through Anaerobic Digestion</i>	1638
	<i>Energy Recovery by Thermal Oxidation</i>	1639
	<i>Energy Recovery from Dried Material through Gasification and Pyrolysis</i>	1639
	<i>Production of Oil and Liquid Fuel</i>	1640
14-10	Application of Biosolids to Land	1640
	<i>Benefits of Land Application</i>	1640
	<i>U.S. EPA Regulations for Beneficial Use and Disposal of Biosolids</i>	1640
	<i>Management Practices</i>	1641
	<i>Site Evaluation and Selection</i>	1643
	<i>Design Loading Rates</i>	1644
	<i>Application Methods</i>	1648
	<i>Application to Dedicated Lands</i>	1650
	<i>Landfilling</i>	1651
	Problems and Discussion Topics	1651
15	Plant Recycle Flow Treatment and Nutrient Recovery	1659
15-1	Sidestream Identification and Characterization	1661
	<i>Sidestreams Derived from Primary and Secondary Sludges</i>	1662
	<i>Sidestreams Derived from Fermented Primary and Digested Primary and Secondary Sludges</i>	1662
15-2	Mitigating Recycle Flows and Loads	1667
	<i>Sidestream Pretreatment</i>	1667
	<i>Equalization of Sidestream Flows and Loads</i>	1667
15-3	Reduction of Suspended Solids and Colloidal Material	1673
	<i>Sidestreams Derived from Sludge Thickening</i>	1673

	<i>Sidestreams Derived from Biosolids</i>	1717
	<i>Dewatering</i>	1673
	<i>Removal of Colloidal Matter</i>	1674
15-4	Physiochemical Processes for Phosphorus Recovery	1674
	<i>Description of the Crystallization Process</i>	1675
	<i>Recovery of Phosphorus as Magnesium Ammonium Phosphate (Struvite)</i>	1678
	<i>Recovery of Phosphorus as Calcium Phosphate</i>	1683
	<i>Phosphorus Recovery from Mainstream Processes</i>	1684
15-5	Physiochemical Processes for Ammonia Recovery and Destruction	1686
	<i>Recovery of Ammonia by Air Stripping and Acid Absorption</i>	1686
	<i>Recovery of Ammonia by Steam Stripping</i>	1690
	<i>Air Stripping with Thermocatalytic Destruction of Ammonia</i>	1692
15-6	Beneficial Use of Recovered Phosphate and Ammonium Products	1693
	<i>Magnesium Ammonium Phosphate Hexahydrate (Struvite)</i>	1693
	<i>Calcium Phosphate (Hydroxapatite)</i>	1694
	<i>Ammonium Sulfate</i>	1694
	<i>Ammonium Nitrate</i>	1695
15-7	Biological Removal of Nitrogen from Sidestreams	1696
	<i>Nitrogen Removal Processes</i>	1696
	<i>Separate Treatment Processes for Nitrogen Removal</i>	1697
	<i>Integrated Sidestream-Mainstream Treatment and Bioaugmentation</i>	1699
15-8	Nitrification and Denitrification Processes	1700
	<i>Fundamental Process Considerations</i>	1700
	<i>Treatment Processes</i>	1703
15-9	Nitritation and Denitritation Processes	1706
	<i>Fundamental Process Considerations</i>	1706
	<i>Treatment Processes</i>	1709
15-10	Partial Nitritation and Anaerobic Ammonium Oxidation (Deammonification) Processes	1709
	<i>Fundamental Process Considerations</i>	1710
	<i>Treatment Processes</i>	1715
15-11	Process Design Considerations for Biological Treatment Processes	1715
	<i>Sidestream Characteristics and Treatment Objectives</i>	1716
	<i>Design Loading and Load Equalization</i>	1717
	<i>Sidestream Pretreatment</i>	1717
	<i>Sidestream Reactor Volume</i>	1718
	<i>Aeration System</i>	1718
	<i>Sludge Retention Time and Mixed Liquor Suspended Solids Concentration</i>	1721
	<i>Chemical Requirements</i>	1721
	<i>Operating Temperature and pH</i>	1723
	<i>Operating pH</i>	1723
	<i>Energy Balance to Determine Reactor Cooling Requirements</i>	1723
	Problems and Discussion Topics	1728
16	Air Emissions from Wastewater Treatment Facilities and Their Control	1737
16-1	Types of Emissions	1739
16-2	Regulatory Requirements	1739
	<i>Ambient Air Quality and Attainment Status</i>	1739
	<i>Preconstruction and Operating Permitting Programs</i>	1741
	<i>Stationary Source Control Technology Requirements</i>	1741
16-3	Odor Management	1742
	<i>Types of Odors</i>	1742
	<i>Sources of Odors</i>	1742
	<i>Measurement of Odors</i>	1745
	<i>Odor Dispersion Modeling</i>	1746
	<i>Movement of Odors from Wastewater Treatment Facilities</i>	1746
	<i>Strategies for Odor Management</i>	1747
	<i>Odor Treatment Methods</i>	1751
	<i>Selection and Design of Odor Control Facilities</i>	1760
	<i>Design Considerations for Chemical Scrubbers</i>	1760
	<i>Design Considerations for Odor Control Biofilters</i>	1762
16-4	Control of Volatile Organic Carbon Emissions	1767
	<i>Physical Properties of Selected VOCs</i>	1768
	<i>Emission of VOCs</i>	1768
	<i>Mass Transfer Rates for VOCs</i>	1771
	<i>Mass Transfer of VOCs from Surface and Diffused-Air Aeration Processes</i>	1771

	<i>Control Strategies for VOCs</i> 1774	
	<i>Treatment of Off-Gases</i> 1774	
16-5	Emissions from the Combustion Of Gases And Solids 1777	
	<i>Sources of Fuels</i> 1777	
	<i>Combustion Systems Used at Wastewater Treatment Plants</i> 1778	
	<i>Emissions of Concern from Combustion Sources</i> 1779	
	<i>Flaring of Digester Gas</i> 1780	
16-6	Emission of Greenhouse Gases 1784	
	<i>Framework for Greenhouse Gases Reduction</i> 1784	
	<i>Assessment Protocols</i> 1784	
	<i>Opportunities for GHG Reduction at Wastewater Treatment Facilities</i> 1791	
	Problems and Discussion Topics 1793	
17	Energy Considerations in Wastewater Management 1797	
17-1	Factors Driving Energy Management 1799	
	<i>Potential for Energy Cost Savings</i> 1799	
	<i>Energy Supply Reliability</i> 1800	
	<i>Considerations for Sustainability</i> 1800	
17-2	Energy in Wastewater 1800	
	<i>Chemical Energy</i> 1800	
	<i>Thermal Energy</i> 1804	
	<i>Hydraulic Energy</i> 1805	
17-3	Fundamentals of a Heat Balance 1807	
	<i>Concept of a Heat Balance</i> 1807	
	<i>Preparation of a Heat Balance</i> 1808	
17-4	Energy Usage in Wastewater Treatment Plants 1809	
	<i>Types of Energy Sources Used at Wastewater Treatment Facilities</i> 1810	
	<i>Energy Use for Wastewater Treatment</i> 1810	
	<i>Energy Use by Individual Treatment Processes</i> 1810	
	<i>Advanced and New Wastewater Treatment Technologies</i> 1811	
17-5	Energy Audits and Benchmarking 1813	
	<i>Benchmarking Energy Usage</i> 1814	
	<i>Benchmarking Protocol</i> 1815	
17-6	Recovery and Utilization of Chemical Energy 1819	
	<i>Fuels Derived from Wastewater</i> 1819	
	<i>Energy Recovery from Gaseous Fuels with Engines and Turbines</i> 1821	
	<i>Energy Recovery from Gaseous Fuels with Boilers</i> 1824	
	<i>Energy Recovery from Solid Fuels</i> 1826	
	<i>Energy Recovery from Syngas</i> 1833	
	<i>Energy Recovery with Fuel Cell</i> 1833	
17-7	Recovery and Utilization of Thermal Energy 1834	
	<i>Sources of Heat</i> 1835	
	<i>Demands for Heat</i> 1836	
	<i>Devices for Waste Heat Recovery and Utilization</i> 1838	
	<i>Design Considerations for Thermal Energy Recovery Systems</i> 1843	
17-8	Recovery and Utilization of Hydraulic Potential Energy 1846	
	<i>Type of Hydraulic Potential Energy Recovery Devices</i> 1846	
	<i>Application of Hydraulic Energy Recovery Devices</i> 1847	
	<i>Use of Residual Pressure Head in Treatment Processes</i> 1849	
17-9	Energy Management 1850	
	<i>Process Optimization and Modification for Energy Saving</i> 1850	
	<i>Process Modification for Increased Energy Production</i> 1856	
	<i>Peak Flowrate Management (Peak Energy Usage)</i> 1857	
	<i>Selection of Energy Sources</i> 1858	
17-10	Future Opportunities for Alternative Wastewater Treatment Processes 1858	
	<i>Enhanced Energy Recovery of Particulate Organic Matter</i> 1858	
	<i>Reduced Energy Usage in Biological Treatment</i> 1859	
	<i>Reduced Energy Usage through the Use of Alternative Treatment Processes</i> 1859	
	<i>Prospects for the Future</i> 1860	
	Problems and Discussion Topics 1860	
18	Wastewater Management: Future Challenges and Opportunities 1865	
18-1	Future Challenges and Opportunities 1867	
	<i>Asset Management</i> 1867	
	<i>Design for Energy and Resource Recovery</i> 1869	

	<i>Design of Wastewater Treatment Plants for Potable Reuse</i> 1869	<i>SSO Guidance</i> 1895
	<i>Decentralized (Satellite) Wastewater Treatment</i> 1872	<i>Wet-Weather Management Options</i> 1895
	<i>Low Impact Development</i> 1873	Discussion Topics 1899
	<i>Triple Bottom Line</i> 1875	
18-2	Impact of Population Demographics, Climate Change and Sea Level Rise, Uncontrollable Events, and Unintended Consequences 1875	
	<i>Impact of Population Demographics</i> 1876	
	<i>Impact of Climate Change and Sea Level Rise</i> 1877	
	<i>Impact of Uncontrollable Events</i> 1879	
	<i>Impact of the Law of Unintended Consequences</i> 1879	
18-3	Upgrading Treatment Plant Performance Through Process Optimization and/or Operational Changes 1882	
	<i>Process Optimization</i> 1882	
	<i>Operational Changes to Improve Plant Performance</i> 1886	
18-4	Upgrading Treatment Plant Performance Through Process Modification 1889	
	<i>Upgrading Physical Facilities</i> 1889	
	<i>Upgrading to Meet New Constituent Removal Requirements</i> 1890	
18-5	Management of Wet-Weather Flows 1890	
	<i>SSO Policy Issues</i> 1892	
		Appendices
		A Conversion Factors 1901
		B Physical Properties of Selected Gases and the Composition of Air 1909
		C Physical Properties of Water 1913
		D Statistical Analysis of Data 1917
		E Dissolved Oxygen Concentration in Water as a Function of Temperature, Salinity, and Barometric Pressure 1923
		F Carbonate Equilibrium 1925
		G Moody Diagrams for the Analysis of Flow in Pipes 1929
		H Analysis of Nonideal Flow in Reactors using Tracers 1931
		I Modeling Nonideal Flow in Reactors 1941
		Indexes
		Name Index 1953
		Subject Index 1966

This page intentionally left blank

Preface

Since completion of the fourth edition of this textbook, the field of wastewater engineering has evolved at a rapid pace. Some of the more significant changes include:

1. A new view of wastewater as a source of energy, nutrients, and potable water;
2. More stringent discharge requirements related to nitrogen and phosphorus;
3. Enhanced understanding of the fundamental microbiology and physiology of the microorganisms responsible for the removal of nitrogen and phosphorus and other constituents;
4. An appreciation of the importance of the separate treatment of return flows with respect to meeting more stringent standards for nitrogen removal and opportunities for nutrient recovery;
5. Increased emphasis on the treatment of sludge and the management of biosolids; and
6. Increased awareness of carbon footprint impacts and greenhouse gas emissions, and an emphasis on the development of energy-neutral or energy-positive wastewater plants through more efficient use of chemical and heat energy in wastewater.

The 5th edition of this textbook has been prepared to address the significant changes cited above. Increased understanding of the importance of pre-treatment processes is addressed in Chap. 5. Advances in biological treatment are addressed in Chaps. 7 through 10. New developments in disinfection are considered in Chap. 12. The management of sludge and biosolids is now covered in Chaps. 13 and 14. Return flow treatment is considered in Chap. 15. Energy management is considered in Chap. 17. An emphasis of this fifth edition is to present practical design and operational data, while maintaining a solid theoretical discussion of the technologies and applications. Input from AECOM's process engineers and outside reviewers was sought to provide the user with a source of real-world practical information, the likes of which is not available in any single source.

IMPORTANT FEATURES OF THIS BOOK

In the 4th edition of this book, a separate chapter was devoted to the fundamentals of process analysis, including an introduction to the preparation of mass balances and reaction kinetics. Because introductory courses on process analysis and modeling are now taught at most colleges and universities, the material on the fundamentals of process analysis from the 4th edition has been condensed and is now included in Secs. 1–7 through 1–11 in Chap. 1. The material on process analysis has been retained as a reference source for students that have already had a separate course on modeling and as an introduction to the subject for students who may not have had an introductory course.

Following the practice in the 4th edition, more than 150 example problems have been worked out in detail to enhance the readers' understanding of the basic concepts presented in the text. To aid in the planning, analysis, and design of wastewater management systems, design data and information are summarized and presented in more than 400 tables, most of which are new. To illustrate the principles and facilities involved in the field of wastewater management, over 850 individual illustrations, graphs, diagrams, and

photographs are included. An additional 120 drawings are included in tables. More than 375 homework problems and discussion topics are included to help the readers of this textbook hone their analytical skills and enhance their mastery of the material. Extensive references are also provided for each chapter.

The International System (SI) of Units is used in the 5th edition. The use of SI units is consistent with teaching practice in most US universities and in most countries throughout the world. In general, dual sets of units (i.e., SI and US customary) have been used for the data tables. Where the use of double units was not possible, conversion factors are included as a footnote to the table.

To further increase the utility of this textbook, several appendixes have been included. Conversion factors from International System (SI) of Units to US Customary Units and the reverse are presented in Appendixes A–1 and A–2, respectively. Conversion factors used commonly for the analysis and design of wastewater management systems are presented in Appendix A–3. Abbreviations for SI and US customary units are presented in Appendixes A–4 and A–5, respectively. Physical characteristics of air and selected gases and water are presented in Appendixes B and C, respectively. The statistical analysis is reviewed in Appendix D. Dissolved oxygen concentrations in water as a function of temperature are presented in Appendix E. Carbonate equilibrium is considered in Appendix F. Moody diagrams for the analysis of flow in pipes are presented in Appendix G. The analysis of nonideal flow in reactors is considered in Appendix H. Modeling nonideal flow in reactors is addressed in Appendix I.

USE OF THIS BOOK

Enough material is presented in this textbook to support a variety of courses for one or two semesters, or three quarters at either the undergraduate or graduate level. The book can be used both as a class textbook or class reference to supplement instructors' notes. The specific topics to be covered will depend on the time available and the course objectives. Suggested course outlines are presented below.

For a one semester introductory course on wastewater treatment, the following material is suggested.

Topic	Chapter	Sections
Introduction to wastewater treatment	1	1–1 to 1–6
Wastewater characteristics	2	All
Wastewater flowrates and constituent loadings	3	All
Physical unit processes	5	5–1 to 5–8
Chemical unit processes	6	6–1 to 6–3
Introduction to biological treatment of wastewater	7	All
Disinfection	12	12–1 to 12–5, 12–9
Biosolids management	13, 14	All
Process selection, design, and implementation	4	All
Advanced treatment processes (optional)	6, 11	6–7, 6–8, 11–5 to 11–7

For a two semester course on wastewater treatment, the following material is suggested.

Topic	Chapter	Sections
Introduction to wastewater treatment	1	1–1 to 1–6
Wastewater characteristics	2	All
Wastewater flowrates and constituent loadings	3	All
Process selection, design, and implementation	4	4–1 to 4–5
Physical unit operations	5	All
Chemical unit operations	6	All
Introduction to biological treatment of wastewater	7	All
Suspended growth biological treatment processes	8	All
Attached growth and combined biological treatment processes	9	9–1 to 9–5
Anaerobic treatment processes	10	10–1 to 10–5
Disinfection	12	All
Sludge Management	13	All
Biosolids management	14	All
Treatment of return flows	15	All

For a one semester course on biological wastewater treatment, the following material is suggested.

Topic	Chapter	Sections
Introduction to wastewater treatment	1	1–1 to 1–6
Wastewater characteristics	2	All
Process selection, design, and implementation	4	4–2, 4–4, 4–5
Introduction to biological treatment of wastewater	7	7–1 to 7–8
Suspended growth processes	8	8–1 to 8–3
Attached growth biological treatment processes	9	All
Anaerobic treatment processes	10	10–1 to 10–5
Anaerobic sludge treatment	13	13–9, 13–10

For a one semester course on physical and chemical unit processes, the following material is suggested. It should be noted that material listed below could be supplemented with additional examples from water treatment.

Topic	Chapter	Sections
Process selection, design, and implementation	4	4–1 to 4–4
Introduction to physical unit processes		
Mixing and flocculation	5	5–3
Sedimentation	5	5–4, 5–6, 5–7,
Gas transfer	5	5–10, 5–11
Filtration (conventional depth filtration)	11	11–3, 11–4, 11–6
Membrane filtration	11	11–7
Adsorption	11	11–9
Gas stripping	11	11–10
UV disinfection	12	12–9
Introduction to chemical unit processes		6–2
Coagulation	6	6–2
Chemical precipitation	6	6–3, 6–4, 6–6
Ion exchange	11	11–11
Water stabilization	6	6–10
Chemical oxidation (conventional)	6	6–7
Advanced oxidation processes	6	6–8
Photolysis	6	6–9

Acknowledgments

This textbook is a tribute to the engineers and scientists who continue to push forward the practice and technologies of the wastewater industry. These advances continue to offer the world cleaner water resources and sustainable water supplies. The book could not have been written without the efforts of numerous individuals including the primary writers, contributing authors, individuals with specialized skills, technical reviewers, outside reviewers, and practitioners who contributed real life experiences.

Contributing authors from AECOM included: Dr. Mohammad Abu-Orf who revised and updated Chaps. 13 and 14, Dr. Gregory Bowden who wrote Chap. 15, and Mr. William Pfrang who revised and updated Chap. 5. Their assistance is acknowledged gratefully. Dr. Harold Leverenz of the University of California at Davis, is singled out for special acknowledgment for extraordinary contributions to the development of the graphics used throughout the text, the revision of Chap. 6, and individual section write ups. Others deserving special acknowledgment, in alphabetical order, are: Mr. Russel Adams an environmental consultant provided comprehensive reviews of Chaps. 3, 11, and 12; Dr. Heidi Gough of the University of Washington wrote the molecular biology section of Chap. 7; Dr. April Gu of Northeastern University who helped write and provided material for Chap. 9; Ms. Emily Legault of HDR Engineers provided thoughtful and comprehensive reviews of Chaps. 2, 3, 7, 8, 11, and 12; Mr. Mladen Novakovic of AECOM contributed to the development of Chap. 5; Mr. Terry Goss of AECOM contributed extensively to the development of Chaps. 13 and 14; and Mr. Dennis Totzke of Applied Technologies had significant involvement in the development of Chap. 10.

The review of the manuscript was critical to maintain the quality of the text. Outside reviewers, arranged alphabetically, who provided critical reviews included: Dr. Onder Caliskaner of Kennedy/Jenks Consultants reviewed portions of Chap. 11; Dr. Robert Cooper of BioVir laboratories reviewed the section on microbiology in Chap. 2; Ms. Libia Diaz of the University of California at Davis reviewed the homework problems; Dr. Robert Emerick of Stantec Engineers, reviewed the section on UV disinfection in Chap. 12; Dr. David Hokanson of Trussell Technologies reviewed portions of Chap. 11; Ms. Amelia Holmes of University of California at Davis reviewed the homework problems; Dr. Kurt Ohlinger of Sacramento Regional County Sanitation District provided review for phosphorus recovery. Dr. Edward Schroeder professor emeritus of the University of California at Davis reviewed portions of Chaps. 1 and 2.

A number of current and former AECOM engineers contributed to the development of the manuscript by providing design information and by reviewing specific portions of the text. Listed in alphabetical order they are:

Mr. Michael Adkins	Mr. Joerg Blischke	Dr. Patrick Coleman
Mr. David Ammerman	Mr. Gary Breitwisch	Mr. Nicholas Cooper
Ms. Jane Atkinson	Dr. Dominique Brocard	Mr. Grant Davies
Mr. Simon Baker	Mr. Nathan Cassity	Mr. Daniel Donahue
Dr. William Barber	Mr. Chi Yun Chris Chen	Mr. Ralph Eschborn
Mr. David Bingham	Mr. William Clunie	Mr. Bryce Figgore

Mr. Steven Freedman	Dr. Mark Laquidara	Ms. Lucy Pugh
Mr. Lee Glueckstein	Dr. David Lycon	Mr. Jeffrey Reade
Mr. Terry Goss	Mr. Jim Marx	Mr. Dennis Sanchez
Mr. Gary Hanson	Mr. Chris Macey	Mr. Ralph Schroedel
Mr. Brian Harrington	Mr. Bradley McClain	Dr. Keith Sears
Mr. Derek Hatanaka	Mr. Alexander Mofidi	Mr. Gerald Stevens
Mr. Gregory Heath	Mr. Paul Moulton	Dr. Beverley Stinson
Mr. Roger Hessel	Mr. Mladen Novakovic	Mr. Jean-Yves Urbain
Dr. Richard Irwin	Mr. Kevin Oldfield	Mr. Kevin Voit
Mr. Jay Kemp	Mr. Ahmed Al-Omari	Mr. Thomas Weber
Mr. King Fai Alex Kwan	Mr. Robert Pape	Mr. Simon Wills
Mr. Pertti Laitinen	Mr. Frederick Pope	

Finally, the production of this textbook could not have been completed without the guidance and assistance of the following individuals. Mr. William Stenquist, Executive Editor and Ms. Lorraine Buczek, Development Editor of the McGraw Hill Book Company. Ms. Rose Kernan and Ms. Erin McConnell of RPK Editorial Services, Inc. provided service above and beyond in working with the authors to produce the textbook. The collective efforts of these individuals were invaluable and greatly appreciated.

George Tchobanoglous
H. David Stensel
Ryujiro Tsuchihashi
Franklin Burton

Foreword

One hundred years have passed since the three-volume “American Sewerage Practice” treatise was published in 1914–1915 by Leonard Metcalf and Harrison P. Eddy. The initial publication quickly became the standard of care and established the foundation for modern wastewater treatment. The original concept of combining theory with a strong compliment of practical data and design guidance continues on in the fifth edition. The wealth of practical information continues to be a cornerstone of Metcalf & Eddy publications, and has led to its reputation as the number one wastewater practice textbook. In this fifth edition over 150 example problems and over 375 homework problems are provided.

The textbook has become a widely used teaching resource for universities and colleges and a reference for engineering firms throughout the world and is now published in Chinese, Greek, Italian, Japanese, Korean, and Spanish.

New advances in technology continue to occur at a record pace in all fields including wastewater treatment. As a result this fifth edition includes numerous advances and represents the current state of the art information. AECOM takes great pride in presenting this Metcalf & Eddy textbook, a comprehensive compilation of the best wastewater practices in use today.

The manuscript was developed by a team of primary writers including Dr. George Tchobanoglous, Dr. H. David Stensel, Dr. Ryujiro Tsuehishi, Dr. Mohammad Abu-Orf, Mr. William Pfrang and Dr. Gregory Bowden. In addition to our primary authors, over 55 AECOM employees and outside technical specialists contributed in reviews and provided practical data and guidance.

I would also like to acknowledge Mr. Bill Stenquist, Executive Editor, McGraw-Hill, who was instrumental in bringing the resources of McGraw-Hill to this project.

The fifth edition textbook could not have been developed without the enthusiastic support of AECOM. I thank Mr. John M. Dionisio, Chairman and Chief Executive Officer, Mr. Robert Andrews, Chief Executive, Water, and Mr. James T. Kunz, Senior Vice President—Program Director.

Jekabs P. Vittands
Senior Vice President
AECOM

This page intentionally left blank

2

Wastewater Characteristics

2-1	WASTEWATER CHARACTERIZATION 60 <i>Wastewater Properties and Constituents</i> 60 <i>Constituents of Concern in Wastewater Treatment</i> 60
2-2	SAMPLING AND ANALYTICAL PROCEDURES 60 <i>Sampling</i> 63 <i>Methods of Analysis</i> 65 <i>Units of Expression for Physical and Chemical Parameters</i> 66 <i>Useful Chemical Relationships</i> 66
2-3	PHYSICAL PROPERTIES 73 <i>Sources of Physical Properties</i> 73 <i>Solids</i> 73 <i>Particle Size and Particle Size Measurement</i> 76 <i>Particle Size Distribution</i> 80 <i>Nanoparticles and Nanocomposites</i> 83 <i>Turbidity</i> 83 <i>Relationship Between Turbidity and TSS</i> 85 <i>Color</i> 85 <i>Absorption/Transmittance</i> 85 <i>Temperature</i> 87 <i>Thermal Energy Content of Wastewater</i> 89 <i>Conductivity</i> 89 <i>Density, Specific Gravity, and Specific Weight</i> 89
2-4	INORGANIC NONMETALLIC CONSTITUENTS 90 <i>Sources of Inorganic Nonmetallic Constituents</i> 90 <i>pH</i> 90 <i>Chlorides</i> 91 <i>Alkalinity</i> 92 <i>Nitrogen</i> 92 <i>Phosphorus</i> 96 <i>Sulfur</i> 97 <i>Gases</i> 98 <i>Odors</i> 103
2-5	METALLIC CONSTITUENTS 111 <i>Sources of Metallic Constituents</i> 112 <i>Importance of Metals</i> 113 <i>Sampling and Methods of Analysis</i> 114 <i>Typical Effluent Discharge Limits for Metals</i> 114
2-6	AGGREGATE ORGANIC CONSTITUENTS 114 <i>Sources of Aggregate Organic Constituents</i> 114 <i>Measurement of Organic Content</i> 114

<i>Biochemical Oxygen Demand (BOD)</i>	115
<i>Total and Soluble Chemical Oxygen Demand (COD and SCOD)</i>	123
<i>Total and Dissolved Organic Carbon (TOC and DTOC)</i>	123
<i>UV-Absorbing Organic Constituents</i>	124
<i>Theoretical Oxygen Demand (ThOD)</i>	125
<i>Interrelationships between BOD, COD, and TOC</i>	125
<i>Oil and Grease</i>	127
<i>Surfactants</i>	128
<i>Chemical Energy in Wastewater and Biosolids</i>	129
2-7 INDIVIDUAL ORGANIC COMPOUNDS	131
<i>Sources of Individual Organic Compounds</i>	132
<i>Priority Pollutants</i>	132
<i>Volatile Organic Compounds (VOCs)</i>	132
<i>Disinfection Byproducts</i>	132
<i>Pesticides and Agricultural Chemicals</i>	133
<i>Unregulated Trace Organic Compounds</i>	133
<i>Analysis of Individual Organic Compounds</i>	133
2-8 RADIONUCLIDES IN WASTEWATER	136
<i>Sources of Radionuclide</i>	137
<i>Units of Expression</i>	137
<i>Description of Isotopes Found in Wastewater and Sludge</i>	137
<i>Treatment Technologies for the Removal of Radionuclides</i>	137
2-9 BIOLOGICAL CONSTITUENTS	139
<i>Sources of Microorganisms in Wastewater</i>	140
<i>Enumeration and Identification of Microorganisms</i>	144
<i>Pathogenic Organisms and Prions</i>	151
<i>Evolving Pathogenic Microorganisms</i>	161
2-10 TOXICITY	161
<i>Sources of Toxicity</i>	161
<i>Evolution and Application of Toxicity Testing</i>	162
<i>Toxicity Testing</i>	163
<i>Analysis of Toxicity Test Results</i>	165
<i>Application of Toxicity Test Results</i>	167
<i>Identification of Toxicity Components</i>	169
PROBLEMS AND DISCUSSION TOPICS	171
REFERENCES	178

WORKING TERMINOLOGY

Term	Definition
Absorbance	A measure of the amount of light, of a specified wavelength, that is absorbed by the constituents in solution.
Antibodies	Proteins used by the immune system to neutralize specific foreign material such as bacteria and viruses.

Term	Definition
Antigen	Any substance that stimulates the immune system to produce antibodies to protect against it.
Anthropogenic compounds	Chemical compounds created by humans, often resistant to biodegradation.
Bacteria	Microscopic organisms typically ranging from approximately 0.5 to 5 μm in length. Municipal wastewater can contain a wide variety and concentration of bacteria, including those pathogenic to humans.
Coliform group of bacteria	Coliforms include several genera of bacteria belonging to the family <i>Enterobacteriaceae</i> . Coliform organisms are common in the environment and in the feces of humans and warm blooded animals. Members of the coliform group include <i>Escherichia</i> , <i>Enterobacter</i> , <i>Klebsiella</i> , and <i>Citrobacter</i> among others.
Disinfection byproducts (DBPs)	A variety of organic compounds that are formed in treated wastewater when a strong oxidant for the purpose of disinfection such as chlorine or ozone is combined with residual organic matter.
Emerging contaminants	Constituents, which have been identified in water, that are considered for regulatory action pending the development of additional information on health and environmental impacts.
Endocrine disrupting compounds (EDCs)	Synthetic and natural compounds that mimic, block, stimulate, or inhibit natural hormones in the endocrine systems of animals, including humans. The origins of EDCs include pesticides, pharmaceutically active chemicals (PhACs), personal care products (PCPs), herbicides, industrial chemicals, and disinfection byproducts.
Enteric virus	An intracellular parasite that is an obligate human pathogen, which means it can only replicate in the human host.
<i>Escherichia coli</i> (<i>E. coli</i>)	A species of the total coliform group that is specific to the intestinal tract of humans and warm blooded animals, commonly associated with fecal contamination. Although most <i>E. coli</i> are non-pathogenic, some strains (serotypes) such as <i>E. coli</i> O157:H7 can cause serious disease.
Fecal coliforms	A thermo-tolerant sub-group of total coliform group found in the intestinal tract of humans and warm blooded animals.
Helminths	A group of parasitic worms; worldwide, helminths are one of the principal causative agents of human disease. Helminths and helminth ova (eggs) are found increasingly in untreated municipal wastewater in the United States.
Higher heating value (HHV)	The amount of heat produced by complete combustion of a unit quantity of fuel.
Lower heating value (LHV)	The lower heating value is obtained by deducting the latent heat of vaporization of water vapor formed by combustion from the HHV.
Nanoparticles	Small objects or particles, ranging in size from 1 to 100 nm, which behave as an entire unit with respect to their properties and transport.
Pathogens	Microorganisms capable of causing diseases of varying severity.
Personal care products	Products such as shampoo, hair conditioner, deodorants, and body lotion.
Pharmaceutically active compounds (PhACs)	Chemicals synthesized for medical purposes (e.g., antibiotics).
Priority pollutants	Constituents, both inorganic and organic, that have been identified by the Environmental Protection Agency as known or suspected carcinogens, mutagens, teratogens, or highly toxic are to be regulated by categorical discharge standards.
Protozoa	Protozoa are single-celled organisms that lack a cell wall. They are common in fresh and marine water and some can grow in soil and other locations.
Trace constituent	A diverse classification of constituents found in low concentrations in untreated wastewater and not readily removed by conventional secondary treatment.
Trace organics	Organic compounds detected at nanogram or microgram per liter concentrations in untreated and treated wastewater by the means of sophisticated laboratory instrumentation.
Viruses	Viruses are infectious agents able to multiply only within a host cell.

An understanding of the nature of wastewater is essential in the design and operation of collection, treatment, and reuse facilities, and in the engineering management of environmental quality. To promote this understanding, the information in this chapter is presented in ten sections dealing with (1) an introduction to the characterization of wastewater, (2) sampling and analytical procedures, (3) physical properties, (4) inorganic nonmetallic constituents, (5) metallic constituents, (6) aggregate organic constituents, (7) individual organic constituents and compounds, (8) microbial characteristics, (9) radio-nuclides, and (10) toxicity tests. The material in this chapter has been organized in a manner similar to that used in Standard Methods (2012), the standard reference work for the characterization of wastewater in the field of environmental engineering.

2–1 WASTEWATER CHARACTERIZATION

The principal constituents in wastewater, derived from domestic, municipal, and industrial sources, are: human excreta (i.e., feces and urine), shower/bath water, food waste, personal and household maintenance products, along with a wide variety of other inorganic and organic compounds in trace amounts. Given the wide variety of constituents that may be found in wastewater, it is common practice to characterize wastewater in terms of its physical properties and its chemical and biologic constituents. The physical properties and constituents found in wastewater and the constituents of concern in wastewater treatment are introduced briefly in the following discussion.

Wastewater Properties and Constituents

The principal physical properties and the chemical and biological constituents of wastewater and their sources are reported in Table 2–1. It should be noted that many of the physical properties and chemical and biological characteristics listed in Table 2–1 are interrelated. For example, temperature, a physical property, affects both the amounts of gases dissolved in the wastewater as well as the biological activity in the wastewater. Another distinction that can be made about the constituents reported in Table 2–1 is whether they are aggregate or individual constituents.

Constituents of Concern in Wastewater Treatment

The important constituents of concern in wastewater treatment are listed in Table 2–2. Secondary treatment standards, as reported in Table 1–2 in Chap. 1, are concerned with the removal of biodegradable organics, total suspended solids, and pathogens. Many of the more stringent standards that have been developed recently deal with the removal of nutrients, heavy metals, and priority pollutants. When wastewater is to be reused, standards normally include additional requirements for the removal of refractory organics, heavy metals, and in some cases, dissolved inorganic solids.

2–2 SAMPLING AND ANALYTICAL PROCEDURES

Proper sampling and analytical techniques are of fundamental importance in the characterization of wastewater. Sampling techniques, the methods of analysis, the units of measurement for chemical constituents, and some useful concepts from chemistry are considered in the next sections.

Table 2-1**Common analyses used to assess the constituents found in wastewater^a**

Test^a	Abbreviation/ definition	Use or significance of test results
Physical characteristics		
Total solids	TS	
Total volatile solids	TVS	
Total fixed solids	TFS	
Total suspended solids	TSS	
Volatile suspended solids	VSS	
Fixed suspended solids	FSS	
Total dissolved solids	TDS (TS – TSS)	
Volatile dissolved solids	VDS	
Total fixed dissolved solids	FDS	
Settleable solids	SS	To determine those solids that will settle by gravity in a specified time period
Particle size	PS	To assess the performance of treatment processes, especially disinfection
Particle size distribution	PSD	To assess the performance of treatment processes
Turbidity	NTU ^b	Used to assess the quality of treated wastewater
Color	Light brown, grey, black	To assess the condition of wastewater (fresh or septic)
Transmittance	%T	To assess the suitability of treated effluent for UV disinfection
Odor	TON	To determine if odors will be a problem
Temperature	°C or °F	Important in the design and operation of biological processes in treatment facilities
Thermal energy content	J/g·°C	Important parameter in the recovery of heat from wastewater
Density	ρ	
Conductivity	EC	Used to assess the suitability of treated effluent for agricultural applications
Inorganic chemical characteristics		
Ammonia	NH ₃	
Ammonium	NH ₄ ⁺	
Nitrite	NO ₂ ⁻	
Nitrate	NO ₃ ⁻	
Organic nitrogen	Org N	
Phosphorus, inorganic	Inorg P	
Orthophosphate	PO ₄ ³⁻	Includes orthophosphates and polyphosphates
Organic phosphorus	Org P	Simplest of the phosphoric acids salts
pH	pH = $-\log [H^+]$	A measure of the acidity or basicity of an aqueous solution

(continued)

| **Table 2-1** (Continued)

Test ^a	Abbreviation/ definition	Use or significance of test results
Inorganic chemical characteristics (continued)		
Alkalinity	$\Sigma(\text{HCO}_3^- + \text{CO}_3^{2-} + \text{OH}^- - \text{H}^+)$	A measure of the buffering capacity of the wastewater
Chloride	Cl^-	To assess the suitability of wastewater for agricultural reuse
Sulfate	SO_4^{2-}	To assess the potential for the formation of odors and may impact the treatability of the waste sludge
Metals	As, Cd, Ca, Cr, Co, Cu, Pb, Mg, Hg, Mo, Ni, Se, Na, Zn	To assess the suitability of the wastewater for reuse and for toxicity effects in treatment. Trace amounts of metals are important in biological treatment
Specific inorganic elements and compounds		To assess presence or absence of a specific constituent
Various gases	$\text{O}_2, \text{CO}_2, \text{NH}_3, \text{H}_2\text{S}, \text{CH}_4$	The presence or absence of specific gases
Organic chemical characteristics		
Five-day biochemical oxygen demand	BOD ₅	A measure of the amount of oxygen required to stabilize a waste biologically over a 5-d period
Five-day carbonaceous biochemical oxygen demand	CBOD ₅	A measure of the amount of oxygen required to stabilize a waste biologically, over a 5-d period, in which nitrogen oxidation is suppressed
Ultimate carbonaceous biochemical oxygen demand	UBOD (also BOD _u , BOD _U)	A measure of the amount of oxygen required to stabilize a waste biologically
Nitrogenous oxygen demand	NOD	A measure of the amount of oxygen required to oxidize biologically the nitrogen in the wastewater to nitrate
Chemical oxygen demand	COD	Often used as a substitute for the BOD test
Total organic carbon	TOC	Often used as a substitute for the BOD test
Specific organic compounds and classes of compounds	MBAS ^c , CTAS ^d	To determine presence of specific organic compounds and to assess whether special design measures will be needed for removal
Chemical energy content	MJ/kg COD	To assess the chemical energy in wastewater
Biological characteristics		
Coliform organisms	MPN (most probable number)	To assess potential presence of pathogenic bacteria and effectiveness of disinfection process
Specific microorganisms	Bacteria, protozoa helminths, viruses	To assess presence of specific organisms in connection with plant operation and for reuse
Toxicity	TU _a ^e and TU _c ^f	To assess acute and chronic toxicity of various wastewater samples

^a Details on the various tests may be found in Standard Methods (2012).

^b NTU = Nephelometric turbidity unit.

^c MBAS = Methylene blue active substances.

^d CTAS = Cobalt thiocyanate active substances.

^e TU_a = toxic unit acute.

^f TU_c = toxic unit chronic.

Table 2–2
Principal constituents of concern in wastewater treatment

Constituent	Reason for importance
Suspended solids	Suspended solids can lead to the development of sludge deposits and anaerobic conditions when untreated wastewater is discharged in the aquatic environment.
Biodegradable organics	Composed principally of proteins, carbohydrates, and fats, biodegradable organics are measured most commonly in terms of BOD (biochemical oxygen demand) and COD (chemical oxygen demand). If discharged untreated to the environment, their biological stabilization can lead to the depletion of natural oxygen resources and to the development of septic conditions.
Pathogens	Communicable diseases can be transmitted by the pathogenic organisms that may be present in wastewater.
Nutrients	Both nitrogen and phosphorus, along with carbon, are essential nutrients for growth. When discharged to the aquatic environment, these nutrients can lead to the growth of undesirable aquatic life. When discharged in excessive amounts on land, they can also lead to the pollution of groundwater.
Priority pollutants	Organic and inorganic compounds selected on the basis of their known or suspected carcinogenicity, mutagenicity, teratogenicity, or high acute toxicity. Many of these compounds are found in wastewater.
Refractory organics	These organics tend to resist conventional methods of wastewater treatment. Typical examples include surfactants, phenols, and agricultural pesticides.
Heavy metals	Heavy metals are usually added to wastewater from commercial and industrial activities and may have to be removed if the wastewater is to be reused.
Dissolved inorganics	Inorganic constituents such as calcium, sodium, and sulfate are added to the original domestic water supply as a result of water use and may have to be removed if the wastewater is to be reused.

Sampling

Sampling programs are undertaken for a variety of reasons, such as to obtain (1) routine operating data on overall plant performance, (2) data that can be used to document the performance of a given treatment process, (3) data that can be used to implement proposed new programs, and (4) data needed for reporting regulatory compliance. To meet the goals of the sampling program, the data collected must be

1. *Representative:* The data must represent the wastewater or environment being sampled.
2. *Reproducible:* The data obtained must be reproducible by others following the same sampling and analytical protocols.
3. *Defensible:* Documentation must be available to validate the sampling procedures. The data must have a known degree of accuracy and precision.
4. *Useful:* The data can be used to meet the objectives of the monitoring plan (Pepper et al., 1996).

Because the data from the analysis of the samples will ultimately serve as a basis for implementing wastewater management facilities and programs, the techniques used in a

metals must be determined to assess any adverse affects that may occur. Calcium, magnesium, and sodium are of importance in determining the sodium adsorption ratio (SAR) which is used to assess the suitability of treated effluent for agricultural use (see Asano, et al., 2007). Where composted sludge is applied in agricultural applications, the concentration of arsenic, cadmium, copper, lead, mercury, molybdenum, nickel, selenium, and zinc must be determined.

Sampling and Methods of Analysis

Methods for determining the concentrations of these substances vary in complexity according to the interfering substances that may be present (Standard Methods, 2012). Metals are determined typically by flame atomic absorption, electrothermal atomic absorption, inductively coupled plasma, or ICP/mass spectrometry. Various classes of metals are defined as (1) *dissolved metals*, those metals present in unacidified samples that pass through a 0.45 μm membrane filter; (2) *suspended metals*, those metals present in unacidified samples that are retained on a 0.45 μm membrane filter; (3) *total metals*, the total of the dissolved and suspended metals or the concentration of metals determined on an unfiltered sample after digestion; and (4) *acid extractable metals*, those metals in solution after an unfiltered sample is treated with a hot dilute mineral acid (Standard Methods, 2012).

Typical Effluent Discharge Limits for Metals

Increasingly, metallic constituents in effluent discharges and in biosolids are being regulated. Typical discharge requirements for metals and other toxic constituents are reported in Table 2–13. In addition to complying with existing U.S. EPA requirements, many states have adopted more restrictive standards to protect specific beneficial uses.

2–6 AGGREGATE ORGANIC CONSTITUENTS

Organic compounds are normally composed of a combination of carbon, hydrogen, and oxygen, together with nitrogen in some cases. The organic matter in wastewater typically consists of proteins (40 to 60 percent), carbohydrates (25 to 50 percent), and oils and fats (8 to 12 percent). Urea, the major constituent of urine, is another important organic compound contributing to fresh wastewater. Because urea decomposes rapidly, urea is seldom found in other than very fresh wastewater. Because of the complex nature of wastewater, the organic characteristics of interest in wastewater are classified as aggregate and individual. Aggregate organic constituents are comprised of a number of individual compounds that cannot be or are not distinguished separately as opposed to constituents that are determined individually.

Sources of Aggregate Organic Constituents

Along with the proteins, carbohydrates, fats and oils, and urea, derived from food and human wastes, wastewater typically contains small quantities of a very large number of different synthetic organic molecules, with structures ranging from simple to extremely complex. Sources of synthetic organic molecules include unused medicine, personal care products, and household cleaning and maintenance products.

Measurement of Organic Content

In general, the analyses used to measure aggregate organic material may be divided into those used to measure gross concentrations of organic matter greater than about 1.0 mg/L and those used to measure trace concentrations in the range of 10^{-12} to 1 mg/L. Laboratory methods commonly used today to measure gross amounts of organic matter (typically

Table 2-13
Typical discharge limits for toxic constituents found in secondary effluent

Constituent	Units	Average value^a	
		Daily	Monthly
Arsenic	µg/L	20	
Cadmium	µg/L	1.1	
Chromium	µg/L	11	
Copper	µg/L	4.9	
Lead ^b	µg/L	5.6	
Mercury	µg/L	2.1	0.012
Nickel ^b	µg/L	7.1	
Selenium ^b	µg/L	5.0	
Silver	µg/L	2.3	
Zinc ^b	µg/L	58	
Dieldrin ^c	µg/L	0.0019	0.00014
Lindane	µg/L	0.16	0.063
Tributyltin	µg/L	0.01	0.005
PAHs ^{d,e}	µg/L	0.049	

^a Limits apply to the average concentration of all samples collected during the averaging period (daily–24-h period; monthly–calendar month).

^b Effluent limitation may be met as a 4-d average. If compliance is to be determined based on a 4-d average, then concentrations of four 24-h composite samples must be reported as well as the average of four.

^c Compliance will be based on the practical quantification level (PQL), 0.07 µg/L.

^d PAHs = polynuclear aromatic hydrocarbons.

^e Compliance will be based on the practical quantification level (PQL) for each PAH, 4 µg/L.

Source: Bay Area Regional Water Quality Control Board, Oakland, CA.

greater than 1 mg/L) in wastewater include (1) biochemical oxygen demand (BOD), (2) chemical oxygen demand (COD), and (3) total organic carbon (TOC). Complementing these laboratory tests is the theoretical oxygen demand (ThOD), which is determined from the chemical formula of the organic matter.

Biochemical Oxygen Demand (BOD)

The most widely used parameter of organic pollution applied to both wastewater and surface water is the 5-d BOD (BOD_5). This determination involves the measurement of the dissolved oxygen used by microorganisms in the biochemical oxidation of organic matter. Despite the widespread use of the BOD test, it has a number of limitations, as discussed later in this section. It is hoped that, through the continued efforts of workers in the field, one of the other measures of organic content, or perhaps a new measure, will ultimately be used in its place. Why, then, if the test suffers from serious limitations, is further space devoted to it in this text? The reason is that BOD test results are now used (1) to determine the approximate quantity of oxygen that will be required to biologically stabilize the organic matter present, (2) to determine the size of waste treatment facilities, (3) to measure the

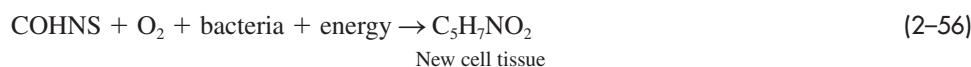
efficiency of some treatment processes, and (4) to determine compliance with wastewater discharge permits. Because it is likely that the BOD test will continue to be used for some time, it is important to know the details of the test and its limitations.

Basis for BOD Test. If sufficient oxygen is available, the aerobic biological decomposition of an organic waste will continue until all of the waste is consumed. Three more or less distinct activities occur. First, a portion of the waste is oxidized to end products to obtain energy for cell maintenance and the synthesis of new cell tissue. Simultaneously, some of the waste is converted into new cell tissue using part of the energy released during oxidation. Finally, when the organic matter is used up, the new cells begin to consume their own cell tissue to obtain energy for cell maintenance. This third process is called endogenous respiration. Using the term COHNS (which represents the elements carbon, oxygen, hydrogen, nitrogen, and sulfur) to represent the organic waste and the term $C_5H_7NO_2$ to represent cell tissue, the three processes are defined by the following generalized chemical reactions:

Energy reaction (oxidation)



Synthesis reaction



Endogenous respiration



If only the oxidation of the organic carbon that is present in the waste is considered, the ultimate BOD is the oxygen required to complete the three reactions given above. This oxygen demand is known as the ultimate *carbonaceous* or first-stage BOD and is usually denoted as UBOD.

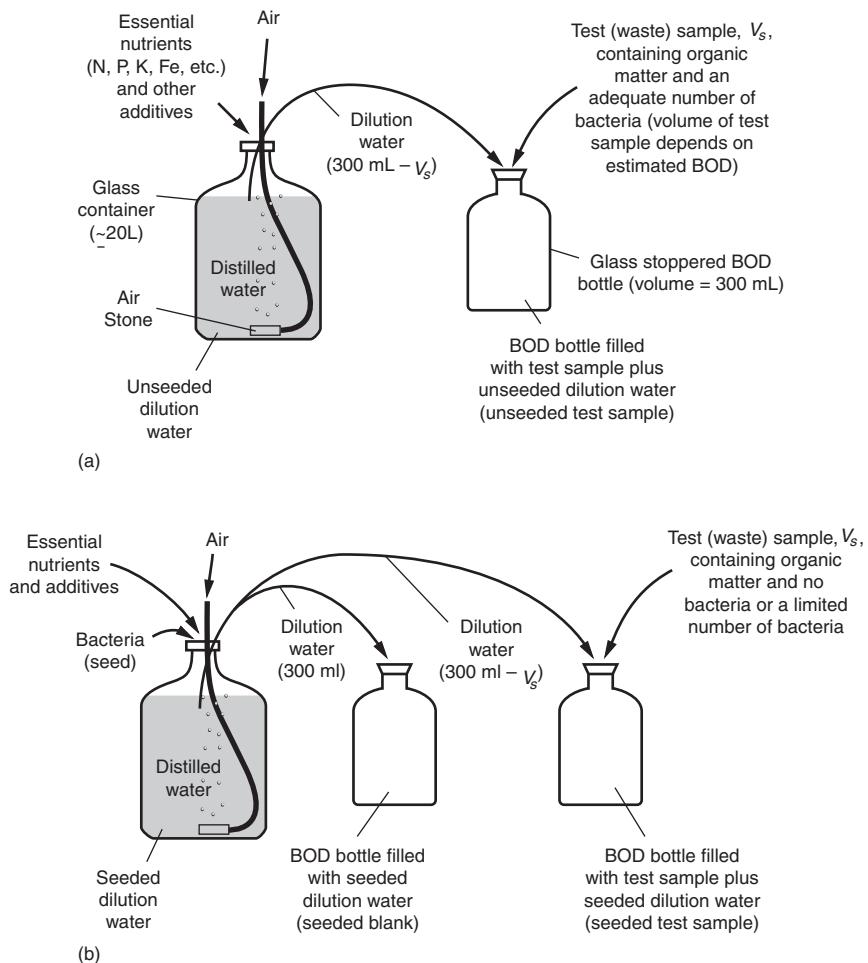
As will be discussed later, the ammonia produced in the energy reaction, Eq. (2-55), can be oxidized further to nitrite and nitrate. Thus, the BOD test only represents the amount of oxygen need to oxidize the carbonaceous material in a sample.

Description of BOD Test Procedure. The standard BOD test [see Fig. 2-21(a)] involves placing a small sample of the wastewater in a BOD bottle (volume = 300 mL). The bottle is then filled with dilution water saturated in oxygen and containing the nutrients required for biological growth. To ensure that meaningful results are obtained, the sample must be suitably diluted with a specially prepared dilution water so that adequate nutrients and oxygen will be available during the incubation period. Normally, several dilutions are prepared to cover the complete range of possible values. Before stoppering the bottle, the oxygen concentration in the bottle is measured (see Fig. 2-22). When testing wastewaters with low concentrations of microorganisms, a seeded BOD test is conducted [see Fig. 2-21(b)]. The organisms contained in the effluent from primary sedimentation facilities are used commonly as the seed for the BOD test. Seed organisms can also be obtained commercially. When the sample contains a large population of microorganisms (e.g., untreated wastewater), seeding is not necessary.

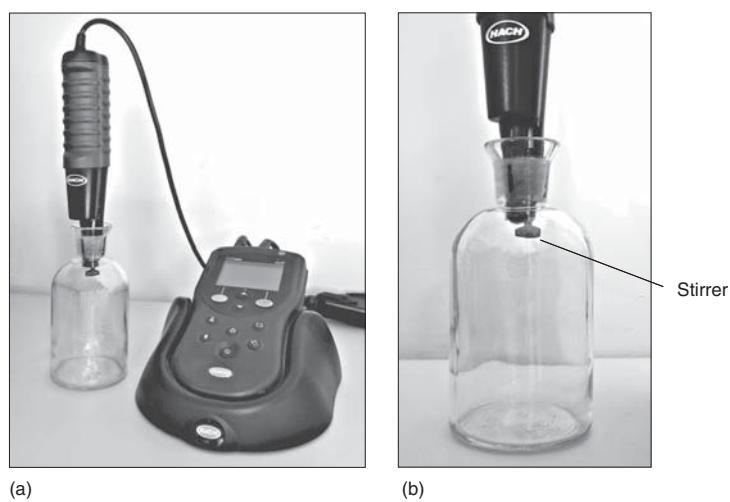
The standard incubation period is usually five days at 20°C, but other lengths of time and temperatures can be used. After incubating for a period of 5-d at 20°C, the dissolved oxygen concentration is measured again. The BOD of the sample is the difference in the

Figure 2-21

Procedure for setting up BOD test bottles: (a) with unseeded dilution water and (b) with seeded dilution water (Tchobanoglous and Schroeder, 1985).

**Figure 2-22**

Measurement of oxygen in BOD bottle: (a) with a DO probe equipped with a stirring mechanism and (b) close up of stirrer.



dissolved oxygen concentration values, expressed in milligrams per liter, divided by the decimal fraction of sample used (Standard Methods, 2012). The computed BOD value is known as the 5-d, 20°C biochemical oxygen demand. The 5-d incubation period dates back to the use of the BOD test to assess river pollution in England the late 1800s. Because the maximum time of flow of any river in England from the headwaters to the ocean is 5 days, the 5-d period was, and is, utilized for the test.

Longer time periods (typically seven days), which correspond to work schedules, are often used, especially in small plants where the laboratory staff is not available on the weekends. The temperature, however, should be constant throughout the test. The 20°C temperature used is an average value for slow-moving streams in temperate climates and is easily duplicated in an incubator. Different results would be obtained at different temperatures because biochemical reaction rates are temperature dependent.

Modeling of BOD Reaction. The rate of BOD oxidation (“exertion”) is modeled based on the assumption that the amount of organic material remaining at any time, t , is governed by a first order function (see Chap. 1).

$$\frac{d\text{BOD}_r}{dt} = k_1 \text{BOD}_r \quad (2-58)$$

Integrating between the limits of UBOD and BOD_r and $t = 0$ and $t = t$ yields:

$$\text{BOD}_r = \text{UBOD} (e^{-k_1 t}) \quad (2-59)$$

where BOD_r = amount of waste remaining at time t (d) expressed in oxygen equivalents, mg/L

UBOD = the total or ultimate carbonaceous BOD, mg/L

k_1 = first-order reaction rate constant, 1/d

t = time, d

Thus the BOD exerted up to time t is given by

$$\text{BOD}_t = \text{UBOD} - \text{BOD}_r = \text{UBOD} - \text{UBOD}(e^{-k_1 t}) = \text{UBOD}(1 - e^{-k_1 t}) \quad (2-60)$$

Equation (2-60) is the standard expression used to define the BOD for wastewater. The basis for this equation is discussed in Sec. 1–5 in conjunction with the analysis of a batch reactor. It should be noted that in the literature dealing with the characterization of wastewater, the terms L and BOD_u are often used to denote ultimate carbonaceous BOD (UBOD).

Biochemical oxidation theoretically takes an infinite time to go to completion because the rate of oxidation is assumed to be proportional to the amount of organic matter remaining. Within a 20-d period, the oxidation of the carbonaceous organic matter is about 95 to 99 percent complete, and in the 5-d period used for the BOD test, oxidation is from 60 to 70 percent complete.

BOD Reaction Rate Coefficients. The value of k_1 for untreated wastewater is generally about 0.12 to 0.46 d⁻¹ (base e), with a typical value of about 0.23 d⁻¹. The range of k_1 values for effluents from biological treatment processes is from 0.12 to 0.23 d⁻¹. For a given wastewater, the value of k_1 at 20°C can be determined experimentally by observing the variation with time of the dissolved oxygen in a series of incubated samples. If k_1 at 20°C is equal to 0.23 d⁻¹, the 5-d oxygen demand is about 68 percent of the ultimate first-stage demand. Occasionally, the first-order reaction rate constant will be expressed in log (base 10) units. The relationship between k_1 (base e) and K_1 (base 10) is as follows:

$$K_1(\text{base 10}) = \frac{k_1(\text{base } e)}{2.303} \quad (2-61)$$

As discussed above, the temperature at which the BOD of a wastewater sample is determined is usually 20°C. It is possible, however, to determine the reaction constant k at a temperature other than 20°C using the following relationship developed in the discussion on the effects of temperature in Chap. 1, Sec. 1-6:

$$\frac{k_2}{k_1} = \theta^{(T_2 - T_1)} \quad (1-44)$$

The value of the temperature coefficient θ has been found to vary from 1.056 in the temperature range between 20 and 30°C to 1.135 in the temperature range between 4 and 20°C (Schroepfer et al., 1964). A value of θ often quoted in the literature is 1.047 (Phelps, 1944), but it has been observed that this value does not apply at cold temperatures (e.g., below 20°C). Equation (2-60), along with Eq. (1-44), makes it possible to convert test results from different time periods and temperatures to the standard 5-d 20°C test, as illustrated in Example 2-9.

EXAMPLE 2-9 Calculation of Different BOD Values Determine the 1-d BOD and ultimate first-stage BOD for a wastewater whose 5-d, 20°C BOD is 200 mg/L. What would have been the 5-d BOD if the test had been conducted at 25°C? The reaction constant k (base e) = 0.23 d⁻¹, and $\theta = 1.047$.

Solution

- Determine the ultimate carbonaceous BOD.

$$\begin{aligned} \text{BOD}_5 &= \text{UBOD} - \text{BOD}_r = \text{UBOD}(1 - e^{-k_1 t}) \\ 200 &= \text{UBOD}(1 - e^{-0.23 \times 5}) = \text{UBOD}(1 - 0.317) \\ \text{UBOD} &= 293 \text{ mg/L} \end{aligned}$$

- Determine the 1-d BOD.

$$\begin{aligned} \text{BOD}_t &= \text{UBOD}(1 - e^{-k_1 t}) \\ \text{BOD}_1 &= 293(1 - e^{-0.23 \times 1}) = 293(1 - 0.795) = 60.1 \text{ mg/L} \end{aligned}$$

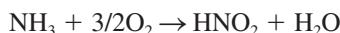
- Determine the 5-d BOD at 25°C.

$$\begin{aligned} k_{1_T} &= k_{1_{20}}(1.047)^{T-20} \\ k_{1_{25}} &= 0.23(1.047)^{25-20} = 0.29 \text{ d}^{-1} \\ \text{BOD}_5 &= \text{UBOD}(1 - e^{-k_1 t}) = 293(1 - e^{-0.29 \times 5}) = 224 \text{ mg/L} \end{aligned}$$

For polluted water and wastewater, a typical value of k_1 (base e at 20°C) is 0.23 d⁻¹ (K_1 , base 10, = 0.10 d⁻¹). The value of the reaction rate constant varies significantly, however, with the type of waste. The range may be from 0.05 to 0.3 d⁻¹ (base e) or more. For the same ultimate BOD, the oxygen uptake will vary with time and with different reaction rate constant values (see Fig. 2-23).

Nitrification in the BOD Test. Noncarbonaceous matter, such as ammonia, is produced during the hydrolysis of proteins. It is now known that a number of bacteria are capable of oxidizing ammonia to nitrite and subsequently to nitrate. The generalized reactions are as follows:

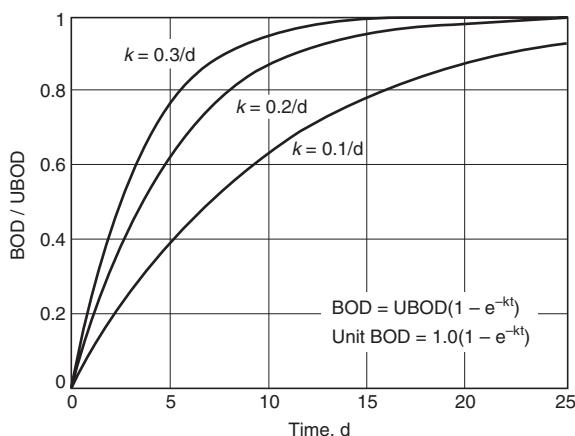
Conversion of ammonia to nitrite (as typified by *Nitrosomonas*):



(2-62)

Figure 2–23

Effect of the rate constant k_1 on BOD (for a unit UBOD value).



Conversion of nitrite to nitrate (as typified by *Nitrobacter*):



Overall conversion of ammonia to nitrate:

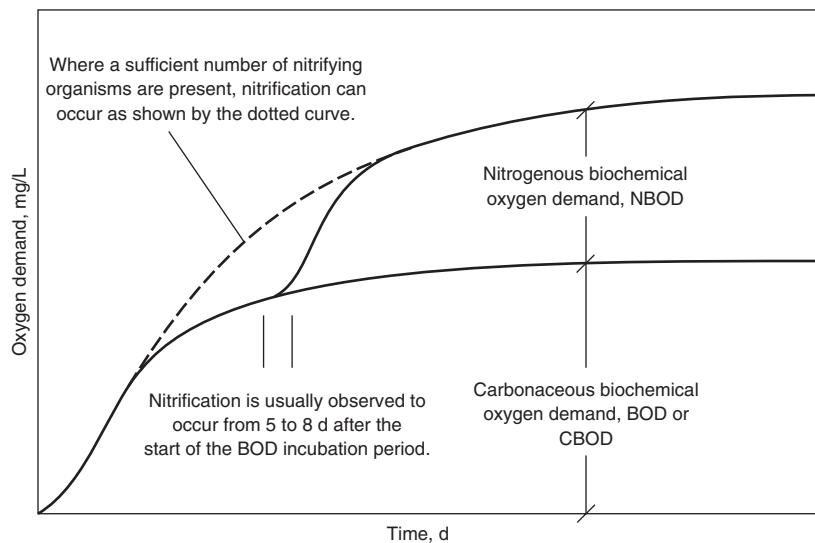


The oxygen demand associated with the oxidation of ammonia to nitrate is called the nitrogenous biochemical oxygen demand (NBOD). The normal exertion of the oxygen demand in a BOD test for a domestic wastewater is shown on Fig. 2–24. Because the reproductive rate of the nitrifying bacteria is slow, it normally takes from 6 to 10 d for them to reach significant numbers to exert a measurable oxygen demand. However, if a sufficient number of nitrifying bacteria is present initially, the interference caused by nitrification can be significant.

When nitrification occurs in the BOD test, erroneous interpretations of treatment operating data are possible. For example, assume the effluent BOD from a biological treatment process is 20 mg/L without nitrification and 40 mg/L with nitrification. If the influent BOD to the treatment process is 200 mg/L, then the corresponding BOD removal efficiency

Figure 2–24

Definition sketch for the exertion of the carbonaceous and nitrogenous biochemical oxygen demand in a waste sample.



3

Wastewater Flowrates and Constituent Loadings

3-1	WASTEWATER SOURCES AND FLOWRATES 185
	<i>Municipal Uses of Water</i> 185
	<i>Domestic Wastewater Sources and Flowrates</i> 186
	<i>Strategies for Reducing Interior Water Use and Wastewater Flowrates</i> 189
	<i>Water Use in Other Parts of the World</i> 194
	<i>Sources and Rates of Industrial (Nondomestic) Wastewater Flows</i> 194
	<i>Variations in Wastewater Flowrates</i> 195
	<i>Long-Term Multiyear Variations Due to Conservation</i> 198
	<i>Impact of Water Conservation on Future Planning</i> 200
3-2	IMPACT OF COLLECTION SYSTEM ON WASTEWATER FLOWRATES 200
	<i>Infiltration/Inflow</i> 200
	<i>Inflow into Collection Systems</i> 202
	<i>Exfiltration from Collection Systems</i> 204
	<i>Combined Collection System Flowrates</i> 205
	<i>Direct Measurement of Combined Sewer Flowrates and Wastewater Characteristics</i> 207
	<i>Calculation of Combined Sewer Flowrates</i> 207
3-3	ANALYSIS OF WASTEWATER FLOWRATE DATA 208
	<i>Statistical Analysis of Flowrate Data</i> 208
	<i>Developing Design Parameters from Flowrate Data</i> 211
	<i>Observed Variability in Influent Flowrates</i> 212
3-4	ANALYSIS OF WASTEWATER CONSTITUENTS 214
	<i>Wastewater Constituents Discharged By Individuals</i> 214
	<i>Constituent Concentrations Based on Individual Mass Discharges</i> 218
	<i>Mineral Increase Resulting from Water Use</i> 218
	<i>Composition of Wastewater in Collection Systems</i> 219
	<i>Variations in Constituent Concentrations</i> 219
	<i>Statistical Analysis of Constituent Concentrations</i> 225
	<i>Observed Variability in Influent Constituent Concentrations</i> 225
3-5	ANALYSIS OF CONSTITUENT MASS LOADING DATA 226
	<i>Simple Average</i> 226
	<i>Flow-Weighted Average</i> 226
	<i>Mass Loadings Rates</i> 229
	<i>Effect of Mass Loading Variability on Treatment Plant Performance</i> 231
3-6	SELECTION OF DESIGN FLOWRATES AND MASS LOADINGS 232
	<i>Design Flowrates</i> 234
	<i>Design Mass Loadings Rates</i> 240

3-7 FLOW AND CONSTITUENT LOAD EQUALIZATION	241
<i>Description/Application of Flow Equalization</i>	242
<i>The Benefits of Flow Equalization</i>	243
<i>Design Considerations</i>	243
<i>Equalization of Constituent Mass Loading Rates</i>	253
<i>Equalization of Sludge and Biosolids Processing Return Flows</i>	253
PROBLEMS AND DISCUSSION TOPICS	254
REFERENCES	260

WORKING TERMINOLOGY

Term	Definition
Combined sewer overflow (CSO)	A hydraulic relief structure within a combined collection system to allow excess wet-weather flow to be discharged ahead of the interceptor, pumping station or wastewater treatment plant.
Combined sewer system	A collection system which conveys both sanitary wastewater at all times and stormwater runoff during wet weather, as well as snowmelt.
Delayed inflow	Stormwater that may require several hours or days or more to drain through the collection system.
Direct inflow	Those types of inflow that have a direct stormwater runoff connection to the sanitary collection system and cause an almost immediate increase in wastewater flowrates.
Domestic wastewater	Wastewater discharged from residences and from commercial, institutional, and similar facilities including infiltration.
Exfiltration	Flow out of a collection system through breaks in the pipe wall, defective pipe joints or connections, or breaks in access port (manhole) walls.
Flow equalization	The dampening of flowrate variations to obtain a constant or nearly constant flowrate, usually by means of a storage (equalization) basin.
Industrial wastewater	Wastewater from nondomestic sources in which industrial wastes predominate.
Infiltration	Water entering a collection system from a variety of entry points including service connections and from the ground through defective pipe joints, connections, or breaks in access port (manhole) walls.
Inflow	Extraneous water that is discharged to the collection system such as from roof leaders, area drains, access port covers, cross connections from storm drains and catch basins, and combined systems.
Instantaneous peak flowrate	Highest recorded flowrate occurring for a period consistent with the recording equipment. In many situations the recorded peak flow may be considerably below the actual peak flow because of metering and recording equipment limitations.
Mass loading rate	The product of flowrate times constituent concentration.
Peaking factor	The ratio of the peak flowrate to the average flowrate.
Sanitary sewer overflow (SSO)	The release of wastewater from the sanitary sewer system caused by backups, clogging or hydraulic overloading.
Sanitary sewer system	A collection system in which primarily domestic wastewater is conveyed.
Steady inflow	Water discharged from cellar and foundation drains, cooling water discharges, and drains from springs and swampy areas.
Stormwater	Runoff resulting from rainfall and snowmelt.
Sustained flowrates	Flowrates that are equalled or exceeded for a specified number of consecutive days based on annual operating data.
Sustained mass loadings	The mass loading rate value sustained or exceeded for a given period of time (e.g., 1 h, 1 d, or 1 mo).

Determining wastewater flowrates and constituent mass loadings is a fundamental step in initiating the conceptual process design for upgrading existing and developing new wastewater treatment facilities. Reliable data for existing and projected flowrates are essential for determining the hydraulic characteristics, sizing, and operational considerations of the treatment system components. Constituent mass loading rate, the product of constituent concentration and flowrate, is necessary to determine the capacity and operational characteristics of the treatment facilities and ancillary equipment to ensure that treatment objectives are met.

Important factors and issues, typical to most planning and design projects, addressed in this chapter include (1) wastewater sources and flowrates, (2) impact of the collection system on wastewater flowrates, (3) analysis of wastewater flowrate data, (4) analysis of wastewater constituents, (5) analysis of constituent concentrations and mass loading rate data, (6) selection of design flowrates and mass loading rates, and (7) flow equalization.

3-1 WASTEWATER SOURCES AND FLOWRATES

The components that make up the wastewater flow from a community, identified previously in Chap. 1, and repeated here for convenience, are

Domestic (also called sanitary) wastewater. Wastewater discharged from residences and from commercial, institutional, and public facilities.

Industrial wastewater. Wastewater in which industrial process wastes predominate.

Infiltration/inflow (I/I). Water that enters the collection system through indirect and direct means. Infiltration is extraneous water that enters the collection system through leaking joints, cracks and breaks, or porous walls. Inflow is stormwater that enters the collection system from storm drain connections (catch basins), roof leaders, foundation and basement drains, or through access port (manhole) covers or breaks in the access port the walls.

Stormwater. Runoff resulting from rainfall and snowmelt.

Data that can be used to estimate average wastewater flowrates from various domestic, commercial, institutional, and industrial sources are presented in this section. The contributions associated with the collection system are considered in the following section.

Municipal Uses of Water

To understand the sources of wastewater it is helpful to review briefly the municipal use of water. Municipal uses of water may be divided into various categories as reported in Table 3-1. Domestic use includes water used indoors in private residences, apartment houses, etc., for drinking, cooking, hand washing, bathing, laundry, toilet flushing, and other uses, and outdoors for landscape irrigation, car washing, and other outdoor purposes.

Commercial and industrial use includes water used by commercial establishments and industries. In small residential communities the commercial and industrial use may be as low as 40 L/capita·d (10 gal/capita·d), but in industrial cities it may as high as 400 L/capita·d (100 gal/capita·d). Public use includes the water required for use in parks, civic buildings, schools, hospitals, churches, street washing, etc. Water that leaks from the system, meter slippage, unauthorized connections, and all other unaccounted-for water is classified as loss and waste. The loss and waste category is often estimated at about 75 L/capita·d (20 gal/capita·d), but with proper construction and careful maintenance it can be reduced to less than 20 L/capita·d (5 gal/capita·d).

Table 3-1
Municipal uses of water and typical quantities in the United States^a

Use	Flowrate, gal/capita·d		Flowrate, L/capita·d	
	Range	Typical	Range	Typical
Domestic				
Indoor use	40–80	65 ^b	150–300	250
Outdoor use	16–90	35 ^c	60–340	132
Commercial	10–75	40	40–300	150
Public	15–25	20	60–100	75
Loss and waste	15–25	20	60–100	75
Total	96–255	170	370–990	682

^a Data developed from numerous sources and the authors experience.

^b Based on current (mid-2013) level of water conservation.

^c In some parts of the country, outdoor water use is significantly higher than indoor use, depending on the season of the year.

Domestic Wastewater Sources and Flowrates

The principal sources of domestic wastewater in a community are the residential areas and commercial districts. Other important sources include public and private institutional facilities and public recreational facilities. Knowledge of the wastewater flowrates is of fundamental importance in the design and operation of wastewater treatment systems. Flowrates are measured both within the collection system and wastewater treatment plants.

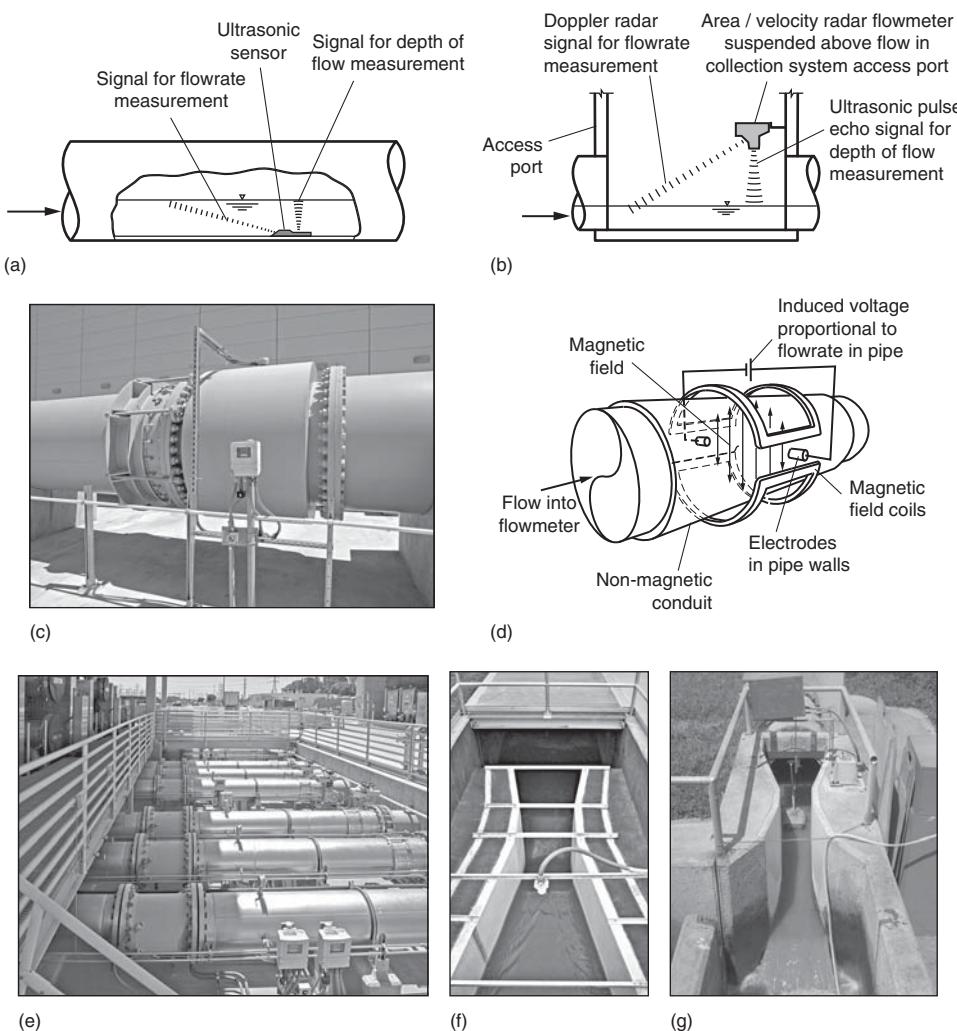
Flowrate Measurements in the Collection System. For areas served with collection systems, wastewater flowrates are determined commonly from existing records or by direct field measurements. Flowrate measurements can be made within gravity sewers by installing flow meters in access ports. In the past, measuring flumes (e.g., Palmer Bowlus) or weirs were installed with level measuring equipment calibrated to output direct flowrate readings. Although flumes and weirs are still used occasionally, most collection system flowrate measurements are now made using area-velocity meters. Area-velocity devices are used to measure simultaneously the depth of flow and the velocity in the collection system without restricting the flow path. Both ultrasonic and radar based devices are used [see Figs. 3-1(a) and (b)]. Unmetered pump stations in the collection system can also be used for gathering flowrate information by measuring the wet well volume and determining the time between pump starting and stopping.

Flowrate Measurements at Treatment Plants. Flowrate measurements at wastewater treatment plants are made with a variety of flowmeters. In the past, influent flowrate measurements were made with Venturi meters for measurements in force mains and Parshall flumes for open channel measurements. Today, the preference is for magnetic meters for force mains because they have a shorter laying length, no flow constrictions, and minimal headloss [see Figs. 3-1(c), (d) and (e)]. Ultrasonic meters are also used. Parshall flumes are the least complicated and flow can be measured manually in case the metering equipment is out of calibration or inoperative [see Figs. 3-1(f) and (g)]. The only problem with the Parshall flume is that to operate properly it requires a free discharge, and thus, the headloss is higher than other measuring devices.

Flowrate Estimates from Available Data. For new developments or newly seweried areas, wastewater flowrates are derived from an analysis of population data and estimates of per capita wastewater flowrates from similar communities. These records are

Figure 3-1

Examples of devices used for measurement of wastewater flowrates: (a) schematic of area/velocity ultrasonic flowrate meter sensor, (b) schematic of area/velocity radar flow meter sensor. Velocity is measured using Doppler Radar and ultrasonic echo plus is used to sense depth of flow, (c) view of magnetic flow meter and (d) schematic of magnetic flow meter. When a conductive fluid moves through a magnetic field a voltage is generated; the magnitude of the voltage is proportional to the flowrate, (e) view of multiple magnetic flow meters used to monitor flowrates to advanced oxidation facilities, and (f) Parshall flume equipped with a sonic water level indicator to determine the depth of flow that is correlated to the flowrate and (g) Parshall flume equipped with a float can also be used to determine depth. (Note: Parshall flumes are used commonly at smaller wastewater treatment plants.)



especially useful in other parts of the world where water use for landscape irrigation is limited and 90 percent or more of the water used becomes wastewater. In the United States, on average about 50 to as high as 90 percent (e.g., from high-rise apartments) of the per capita water consumption becomes wastewater. The higher percentages apply to the northern states during cold weather; the lower percentages are applicable to the semi-arid region of southwestern United States where landscape irrigation is used extensively. When water consumption records are used for estimating wastewater flowrates, the amount of water consumed for purposes such as landscape irrigation (that is not discharged to the collection system), leakage from water mains and service pipes, or product water that is used by manufacturing establishments must be evaluated carefully.

Residential Areas. For many residential areas, wastewater flowrates are commonly determined based on population and the average per capita contribution of wastewater. For residential areas where large residential development is planned, it is often advisable to develop flowrates on the basis of land-use areas and anticipated population densities. Where possible, these rates should be based on actual flow data from selected similar communities, preferably in the same locale. In the past, the preparation of population projections for use in

estimating wastewater flowrates was often the responsibility of the engineer, but today population projection data are usually available from local, regional, and state planning agencies.

Wastewater flowrates can vary depending on the quantity and quality of the water supply, the potable water billing rate structure, the extent of conservation measures, geographic location, rates of infiltration/inflow, and other economic and social characteristics of the community. In a report published by the U.S. Geological Survey (Kenny et al., 2009), the average domestic water use per capita by state varied from a high of 715 L/capita·d (189 gal/capita·d) to a low of 193 L/capita·d (51 gal/capita·d). The average for the country was 375 L/capita·d (99 gal/capita·d), which is consistent with the data reported in Table 3–1. If it is assumed that 50 to 90 percent of the water supply becomes wastewater, the average wastewater flowrates would vary from 188 to 338 L/capita·d (50 to 89 gal/capita·d). The reason that an average range is given is that no one has an accurate estimate of the extent of water conservation measures that have been implemented, which varies from community to community. The typical average range reflects an estimate of the current (2013) extent to which conservation measures have been implemented in different parts of the country.

Data on the typical flowrate values for residential sources in the United States as a function of the number of residents with the current (2013) level of conservation and with significant water conservation are given in Table 3–2. The data from Table 3–2 are plotted on Fig. 3–2. As shown on Fig. 3–2, as the number of persons per residence increases, the average wastewater flowrate per capita decreases. In the case with significant conservation measures, the per capita value approaches a value of about 150 L/capita·d (39.6 gal/capita·d). At the present time this asymptotic value represents a reasonable estimate of what can be achieved with extensive water conservation measures and fixtures and appliances. Assuming an occupancy rate of 3.3 persons per residence, the range of anticipated wastewater flowrates will vary from about 250 to 175 L/capita·d (66 to 46 gal/capita·d). The value of 250 L/capita·d (66 gal/capita·d) is within the range of typical per capita values reported above. With the passage of time, it is anticipated that the right-hand curve on Fig. 3–2 will move to the left. It is estimated that reaching significant conservation may take more than 20 y. Reduced household water use not only changes the quantity of wastewater generated, but, as discussed later in this chapter, the characteristics of wastewater as well.

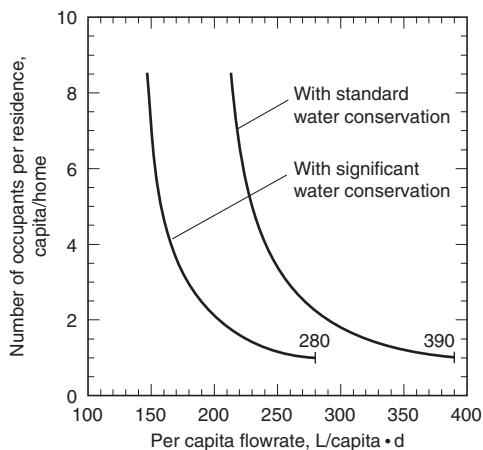
Commercial Districts. Depending on the function and activity, unit flowrates for commercial facilities can vary widely. Because of the wide variations that have been observed,

Table 3–2
Typical wastewater flowrates from urban residential sources in the United States

Household size, no. of persons	Flowrate, gal/capita·d		Flowrate, L/capita·d	
	With current level of conservation	With extensive conservation	With current level of conservation	With extensive conservation
1	103	74	390	280
2	77	54	290	205
3	68	48	257	180
4	63	44	240	168
5	61	42	230	160
6	59	41	223	155
7	58	40	218	151
8	57	39	215	149

Figure 3-2

Per capita flowrate from individual residences as a function of the number of residents.



every effort should be made to obtain records from actual or similar facilities. If no other records are available, estimates for selected commercial sources, based on function or persons served, may be made using the data presented in Table 3-3. In the past, commercial wastewater flowrates were often based on existing or anticipated future development or comparative data. Flowrates were generally expressed in terms of quantity of flow per unit area [i.e., $\text{m}^3/\text{ha} \cdot \text{d}$ (gal/ac \cdot d)]. Typical unit area flowrate allowances for commercial developments normally range from 7.5 to 14 $\text{m}^3/\text{ha} \cdot \text{d}$ (800 to 1500 gal/ac \cdot d). The latter approach can be used to check the values obtained from existing records or estimates made using Table 3-3.

Institutional Facilities. Typical flowrates from some institutional facilities are shown in Table 3-4. Again, it is stressed that flowrates vary with the region, climate, and type of facility. The actual records of institutions are the best sources of flow data for design purposes.

Recreational Facilities. Wastewater flowrates from many recreational facilities are subject to seasonal variations. Typical data on wastewater flowrates from recreational facilities are presented in Table 3-5.

Strategies for Reducing Interior Water Use and Wastewater Flowrates

Because of the importance of conserving both resources and energy, various means for reducing wastewater flowrates are available. The reduction of wastewater flowrates from domestic sources results directly from the reduction in interior water use. Therefore, the terms *interior water use* and *domestic wastewater flowrates* are sometimes used interchangeably. Representative water use rates for various devices and appliances are reported in Table 3-6. Information on the relative distribution of water use within a residence is reported in Table 3-7. Devices and appliances that can be used to reduce interior domestic water use and wastewater flows are described in Table 3-8.

Another method of achieving flow reduction that has been adopted by a number of communities is to restrict the water usage of certain appliances, such as automatic dishwashers and kitchen food-waste grinders (i.e., garbage disposal units), that tend to increase water consumption. The use of one or more of the flow-reduction devices is specified for all new residential dwellings in many communities; in others, the use of waste-food grinders has been limited in new housing developments. Further, many individuals concerned

Table 3–3**Typical wastewater flowrates from commercial sources in the United States^a**

Source	Unit	Flowrate, gal/unit·d		Flowrate, L/unit·d	
		Range	Typical	Range	Typical
Airport	Passenger	2.4–3.8	3	9–14	11
Apartment	Person	32–45	38	120–170	145
Automobile service station	Vehicle served	6–11	8	23–42	30
	Employee	7–11	10	26–42	38
Bar/cocktail lounge	Seat	8–15	11	30–57	43
	Employee	8–12	10	30–45	37
Boarding house	Person	20–45	30	76–170	115
Conference center	Person	5–8	6	20–30	24
Department store	Toilet room	280–450	300	1000–1700	1100
	Employee	6–11	8	23–42	30
Hotel	Guest	52–56	53	200–215	200
	Employee	6–11	8	23–42	30
Industrial building (sanitary waste only)	Employee	12–26	15	45–98	60
Laundry (self-service)	Machine	320–413	338	1210–1560	1280
	Load	36–41	38	136–155	145
Mobile home park	Unit	100–113	105	380–430	400
Motel (with kitchen)	Guest	36–60	38	135–230	145
Motel (without kitchen)	Guest	32–53	34	120–200	130
Office	Employee	6–12	10	23–45	38
Public lavatory	User	2.4–3.8	3	9–14	12
Restaurant:					
Conventional	Customer	6–8	6	23–30	24
With bar/cocktail lounge	Customer	6–9	7	23–34	26
Shopping center	Employee	6–10	8	23–38	30
	Parking space	0.8–2.3	1.5	3–9	6
Theater (Indoor)	Seat	1.6–3	2.3	6–11	9

^a Adapted in part from Tchobanoglou et al. (2003).

about conservation have installed such devices on their own as a means of reducing water consumption. New designs in front-loading clothes washers also offer significant reductions in water use, on the order of 50 to 75 percent of older models. A comparison of residential interior water use (and resulting per capita wastewater flows) is given in Table 3–9 for homes with the current levels of conservation and with extensive water conserving appliances and fixtures. The potential savings of employing selected water-efficient devices is illustrated in Example 3–1.

Table 3-4**Typical wastewater flowrates from institutional sources in the United States^a**

Source	Unit	Flowrate, gal/unit·d		Flowrate, L/unit·d	
		Range	Typical	Range	Typical
Assembly hall	Guest	1.6–3	2.3	6–11	9
Church	Seat	1.6–3	2.3	6–11	9
Hospital	Bed	128–240	150	480–900	570
	Employee	4–11	7.5	15–42	30
Institutions other than hospitals	Bed	60–94	75	230–360	285
	Employee	4–11	7.5	15–42	28
Prison	Inmate	60–110	90	240–430	340
	Employee	4–11	7.5	15–42	28
School, day					
With cafeteria, gym, and showers	Student	12–23	19	45–90	70
With cafeteria only	Student	8–15	11	30–60	42
School, boarding	Student	32–60	38	120–230	140

^a Adapted in part from Tchobanoglous et al. (2003).

Table 3-5**Typical wastewater flowrates from recreational facilities in the United States^a**

Facility	Unit	Flowrate, gal/unit·d		Flowrate, L/unit·d	
		Range	Typical	Range	Typical
Apartment, resort	Person	40–53	45	150–200	170
Cabin, resort	Person	6.4–38	30	24–145	115
Cafeteria	Customer	1.6–3	2.3	6–11	9
	Employee	6.4–9	7.5	24–34	28
Camp					
With toilets only	Person	12–23	18.8	45–87	70
With central toilet and bath facilities	Person	28–38	33.8	106–144	128
Day	Person	12–15	11.3	45–57	43
Cottages, (seasonal with private bath)	Person	32–45	37.5	120–170	142
Country club	Member present	16–30	18.8	60–115	70
	Employee	8–11	9.8	30–42	37
Dining hall	Meal served	3–7.5	5.3	11–28	20

(continued)

| Table 3-5 (Continued)

Facility	Unit	Flowrate, gal/unit·d		Flowrate, L/unit·d	
		Range	Typical	Range	Typical
Dormitory, bunkhouse	Person	16–38	30	120–200	115
Fairground	Visitor	0.8–2.3	1.5	3–9	6
Picnic park with flush toilets	Visitor	4–7.5	3.8	15–28	14
Recreational vehicle park					
With individual connection	Vehicle	60–113	75	230–430	284
With comfort station	Vehicle	32–38	33.8	120–145	128
Roadside rest areas	Person	2.4–4	2.5	9–15	11
Swimming pool	Customer	4–9	6.8	15–34	26
	Employee	6.4–9	7.5	24–34	28
Vacation home	Person	20–45	37.5	76–170	142
Visitor center	Visitor	2.4–4	2.5	9–15	11

^aAdapted in part from Tchobanoglous et al. (2003).

| Table 3-6

Typical rates of water use for various devices and appliances in the United States

Device or appliance	US Customary Units			SI Units		
	Units	Range	Typical	Units	Range	Typical
Bathtub	gal/use	25–35	30	L/use	95–130	114
Dishwasher	gal/load	5–15	10	L/load	19–57	38
Faucet, typical	gal/min·use	0.5–4	2.5	L/min·use	1.9–15	9
Kitchen food waste grinder	gal/d	1–2	1.5	L/d	4–8	6
Shower, standard	gal/min·use	4–7	5	L/min·use	15–26	19
Shower, low-flow	gal/min·use	2–2.5	2.5	L/min·use	8–9.5	9
Toilet, pre 1980s	gal/use	4–7	6	L/use	15–26	23
Toilet, 1980–1992-3	gal/use	3–4	3.5	L/use	11–15	13
Toilet, tank, low-flow	gal/use	0.9 ^a –1.6 ^b	1.6	L/use	3.4–6	6
Washbasin	gal/min·use	1–3	2	L/min·use	8–11	8
Washing machine						
Top loading, standard	gal/load	40–50	45	L/load	150–190	170
Front loading, low-flow	gal/load	12–25	20	L/load	45–95	76

^aDual flush type.

^bCurrently, some states have adopted regulations mandating the use of 1.28 gal/flush toilets. In the future, it is anticipated that the 1.28 gal/flush toilet may become a national standard and that in the future the allowable water usage per flush may be reduced further to 1.0 gal/flush.

Table 3-7

Typical distribution of residential indoor water use in the United States^a

Use	Percent of total		Typical water usage ^b	
	Range	Typical	gal/capita·d	L/capita·d
Bath	1.5–2	1.8	1.2	4.4
Clothes washing (Laundry)	20–24	23	15.0	56.6
Dishwashing	1–1.5	1.4	0.9	3.4
Faucet	15–18	16	10.4	39.4
Shower	16–20	18	11.7	44.3
Toilet flushing	24–30	28	18.2	68.9
Other domestic ^c	2–3	2.2	1.4	5.4
Leakage	8–12	9.6	6.2	23.6
Total		100	65.0	246.0

^a Typical range of indoor water use in the United States ranges from 150 to 300 L/capita·d (40 to 80 gal/capita·d).

^b Based on an indoor water usage rate of 246 L/capita·d (65 gal/capita·d).

^c Houseplant watering, water for pets, etc.

Table 3-8

Flow reduction devices and appliances

Device/appliance	Description and/or application
Faucet aerators	Increases the rinsing power of water by adding air and concentrating flow, thus reducing the amount of washwater used
Flow-limiting shower heads	Restricts and concentrates water passage by means of orifices that limit and divert shower flow for optimum use by the bather
Low-flush toilets	Reduces the amount of water per flush
Pressure reducing valve	Reduces home water pressure below that of the water distribution system, decreases the probability of leaks and dripping faucets
Pressurized shower	Water and compressed air are mixed together. Impact provides the sensation of conventional shower
Retrofit kits for bathroom	Kits may consist of shower flow restrictors, toilet dams or fixture displacement bags, and toilet leak detector tablets
Toilet dam	A partition in the toilet tank that reduces the amount of water per flush
Toilet leak detectors	Tablets that dissolve in the toilet tank and release dye to indicate leakage of the flush valve
Vacuum toilet	A vacuum along with a small amount of water is used to remove solids from toilet
Water efficient dishwasher	Reduces the amount of water used to wash dishes
Water efficient clothes washer	Reduces the amount of water used to wash clothes. New front-loading machines have been developed that not only use less water but are more energy efficient

EXAMPLE 3-1 Determine Water Savings by Employing Water Efficient Appliances

A new subdivision of 2000 homes is planned, and a condition of the building permit is to determine the potential savings in water consumption (and wastewater flows) if the following water-efficient appliances are used: front-loading washing machines, ultra-low flush toilets, and ultra-low flow shower heads. Use 3.5 residents per home and values for devices and appliances from Table 3-9 to determine the potential savings.

The estimated water use and percentage savings are illustrated in the following table.

Appliance/device	No. of Residents	Unit water use, L/capita·d		Water Use, L/d	
		With current level of conservation	With extensive conservation	With current level of conservation	With extensive conservation
Clothes washing	7000	56.6	36.0	396,200	252,000
Toilets	7000	68.9	31.0	482,300	217,000
Showers	7000	44.3	26.1	310,100	182,700
Total				1,188,600	651,700
Savings, %					45

Comment

Three of the largest water using appliances and devices utilized in the home are those described in this example. Interior water use and the generation of wastewater can be reduced significantly with the community-wide installation of water-efficient appliances and devices, thus reducing the flows that will have to be handled by the collection system and treatment plant. Where high infiltration rates occur within the collection system, it is difficult or impossible to assess the beneficial effects of using water conservation devices.

Water Use in Other Parts of the World

The typical flowrates and use patterns presented in Tables 3-1 through 3-7 and Table 3-9 are based on water use and wastewater flowrate data from communities and facilities in the United States. Many developed countries (e.g., Canada) have flowrates in similar ranges. Water use, and consequently the per capita wastewater generation rate in less developed countries, is significantly lower. In some cases, the water supply is only available for limited periods of the day. Water use data from other parts of the world are given in Table 3-10. The striking aspect of the data presented in Table 3-10 is the variability that exists between countries. In general, information such as presented in Table 3-10 is not very useful, other than for gross comparisons, as it is not presented in the context of the country dynamics and the basis for the collection and reporting of data is not uniform.

Sources and Rates of Industrial (Nondomestic) Wastewater Flows

Nondomestic wastewater flowrates from industrial sources vary with the type and size of the facility, the degree of water recycling within the facility, and the presence of any onsite wastewater pretreatment or final treatment methods. Extremely high peak flowrates may be reduced by the use of onsite detention tanks and equalization basins. Typical design values for estimating the flows from industrial areas that have no or little

Table 3-9

Typical comparisons of indoor water use with current level of conservation and with extensive conservation practices and devices in the United States

Use	Flow, gal/capita·d		Flow, L/capita·d	
	With current level of conservation ^{a,b,c}	With extensive conservation ^d	With current level of conservation	With extensive conservation
Bath	1.2 (30)	1.2 (30)	4.4	4.5
Clothes washing (Laundry)	15.0 (30)	9.5 (20)	56.6	36.0
Dishwashing	0.9 (10)	0.7 (8)	3.4	2.6
Faucet	10.4 (3)	6.9 (2)	39.4	26.1
Shower	11.7 (4)	6.9 (2.5)	44.3	26.1
Toilet flushing	18.2 (3.3) ^e	8.2 (1.6)	68.9	31.0
Other domestic	1.4 (3)	1.4 (3)	5.4	5.3
Leakage	6.2	6.0	23.6	22.7
Total	65.0	40.8	246.0	154.4

^aRates of indoor water use based on values given in Table 3-7.

^bCurrent level of conservation assumed to reflect mid-2013 values.

^cNumber in parenthesis is the assumed current water usage rate per use for various uses and devices as given in Table 3-6. For example, the number in parenthesis for the bath is 30 gal/use.

^dNumber in parenthesis is assumed to correspond to the water usage rate with extensive conservation based on the values given in Table 3-6.

^eEstimated average of currently installed toilets.

wet-process type industries are 7.5 to 14 m³/ha·d (1000 to 1500 gal/ac·d) for light industrial developments and 14 to 28 m³/ha·d (1500 to 3000 gal/ac·d) for medium industrial developments. For industries without internal water recycling or reuse programs, it can be assumed that about 85 to 95 percent of the water used in the various operations and processes will become wastewater. For large industries with internal water-recycling programs, separate estimates based on actual water consumption records must be made. Average domestic (sanitary) wastewater contributed from industrial facilities per employee may vary from 30 to 95 L/capita·d (8 to 25 gal/capita·d).

Variations in Wastewater Flowrates

Wastewater flowrates vary during the time of day, day of the week, season of the year, and year to year depending upon the nature of the discharges to the collection system and sources and rates of infiltration/inflow. Short-term, seasonal, multiyear, and industrial variations in wastewater flowrates are briefly discussed here.

Short-Term Variations. The typical current diurnal pattern of wastewater flows observed at treatment plants, such as shown on Fig. 3-3(a) for an intermediate size community, began in the latter half of the 20th century. In the early part of the twentieth century, the daily variation in wastewater flowrates was characterized by a single peak in the morning as illustrated on Fig. 3-3(a). The shift from a single to a dual peak reflects the changes that have occurred in the workplace, perhaps the most notable event being the employment of women in factories during the early 1940s as part of the war effort.

Table 3-10

Water consumption in various countries and the United States in 2000^a

Country	Per capita water consumption	
	gal/d	L/d
Argentina	93	350
Austria	113	430
Canada	196	742
Chile	63	238
Germany	41	156
Greece	93	350
Hungary	139	526
India	34	129
Kuwait	53	200
Libya	74	279
Nepal	8	30
Mexico	92	348
Mozambique	3	11
Norway	29	110
Russian Federation	72	274
Saudi Arabia	50	189
South Africa	59	224
United States	100	380

^a Adapted from United Nations (2005).

Referring to Fig. 3-3(a), minimum flows occur during the early morning hours when water consumption is lowest and when the base flow consists of infiltration and small quantities of sanitary wastewater. The first flowrate peak generally occurs in the late morning when wastewater from the peak morning water use reaches the treatment plant. A second flowrate peak generally occurs in the early evening between 7 and 9 p.m. In some bedroom communities, the amplitude of the second peak will exceed the morning peak. It should also be noted that a shift occurs on the weekends with respect to the morning peak [see Fig. 3-3(a)] as people tend to get up a bit later.

The time of occurrence and the amplitude of the flowrate peaks vary with the size of the community and the length and storage capacity of the collection system [see Fig. 3-3(b)]. In the curves shown on Fig. 3-3(b), there is essentially no late afternoon peak. The reason for this occurrence is that the collection system has excess storage capacity and there is a significant travel time to reach the treatment plant. The same phenomena will be observed where a centralized treatment plant for a larger community also serves a number of smaller communities located some distance from the plant. Because of the travel time, the flows from the outlying communities will arrive later than the peak flow from the main community. Arriving later, these flows tend to dampen the second peak that would have been observed had the treatment plant only received wastewater from the single community. Further, as a community increases in size, the diurnal variations tend to be reduced as shown on Fig. 3-3(c). The masking effect caused by high infiltration rates during a storm event is illustrated on Fig. 3-3(d), in

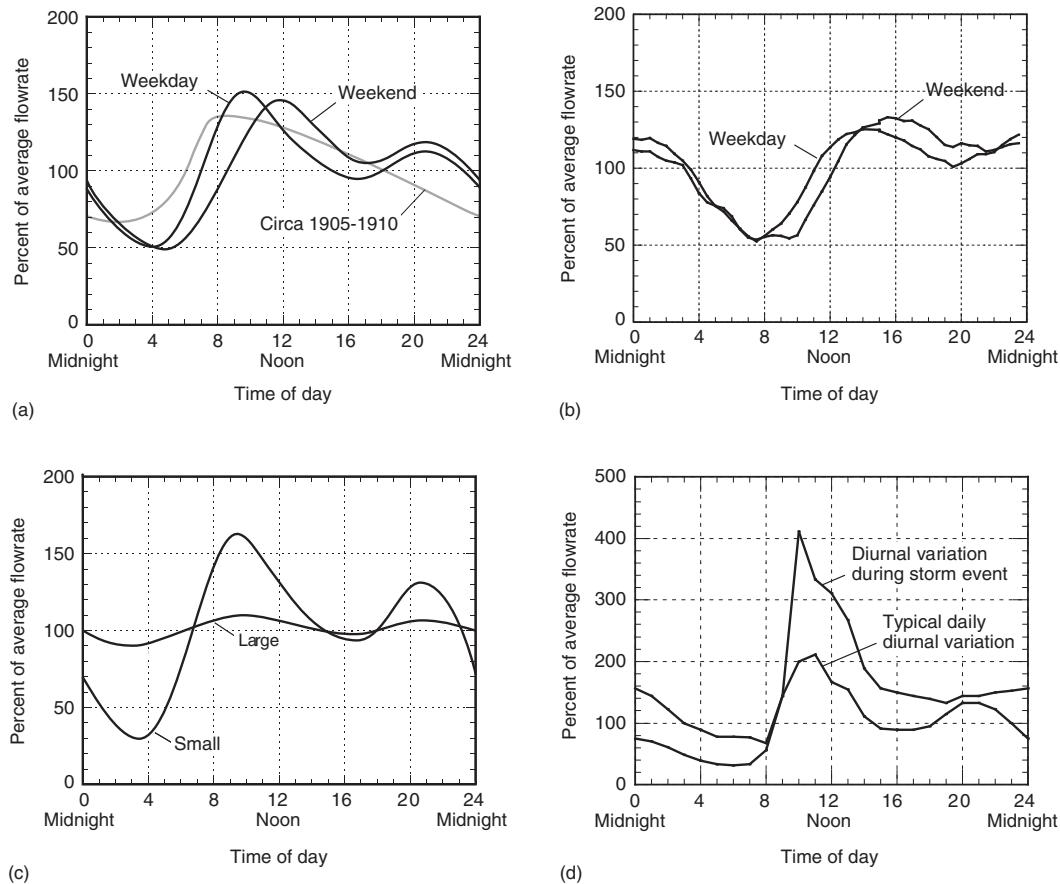


Figure 3-3

Typical variations in normalized influent flowrate data for domestic wastewater systems:

(a) generalized diurnal pattern for weekdays and weekends in the latter part of the 20th century for an intermediate sized community ($40,000\text{--}400,000 \text{ m}^3/\text{d}$), and the corresponding curve observed in the early part of the 20th century (ca. 1905–1910); (b) observed flowrate variations for the City of Davis (population 65,000), based on half hour flowrate measurements (courtesy of West Yost and Associates); (c) generalized flowrate variations for small ($4000\text{--}40,000 \text{ m}^3/\text{d}$) and large ($>400,000 \text{ m}^3/\text{d}$) communities; and (d) masking effect of excess stormwater infiltration, based on hourly flowrate measurements (note change in vertical scale).

which the late afternoon peak is masked by the continued flow resulting from the storm event.

When extraneous flows are minimal, wastewater discharge curves resemble water consumption curves, but with a lag of several hours. As the community size increases, the variations between the high and low flows decrease due to (1) the increased storage in the collection system of large communities that tends to equalize flowrates and (2) changes in the economic and social makeup of the community.

Industrial Variations. Industrial wastewater discharges are difficult to predict. Many manufacturing facilities generate relatively constant flowrates during production, but the flowrates change markedly during cleanup and shutdown. While internal process

changes may lead to reduced discharge rates, plant expansion and increased production may lead to increased wastewater generation. Where joint treatment facilities are to be constructed, special attention should be given to industrial flowrate projections, whether they are prepared by the industry or jointly with the city's staff or engineering consultant. Industrial discharges are most troublesome in smaller wastewater treatment plants where there is limited capacity to absorb shock loadings.

Seasonal Variations. Seasonal variations depend on location and the nature of the community. In the eastern part of the United States, where it tends to rain throughout the year, there is less seasonal variation in observed flowrates as compared to the western United States where there are distinct wet (November through April) and dry (May through October) periods. Snowmelt is a significant factor in the areas with high rates of infiltration observed each spring, due to seasonably high groundwater levels in the Northeast and other snow-belt states. The difference in observed flowrates is illustrated on Fig. 3–4. It is important to note that the general patterns and the magnitude of the flowrates shown on Fig. 3–4 will vary considerably with increased or decreased rainfall patterns resulting from global climate change.

Seasonal variations in domestic wastewater flows are commonly observed at resort areas, in small communities with college campuses, and in communities that have seasonal commercial and industrial activities. The magnitude of the variations to be expected depends on both the size of the community and the seasonal activity.

Long-Term Multiyear Variations Due to Conservation

In addition to the daily and seasonal changes described above, some significant trends have been observed in the long-term flowrates measured at wastewater treatment plants at many large cities in the United States. In general, three major trends in flowrates are observed based on cities with (1) increasing population, (2) relatively constant population, and (3) decreasing population as illustrated on Fig. 3–5. In all cases, the recent and ongoing implementation of low-flow appliances and fixtures and practices that result in water conservation will result in an increase in wastewater constituent concentrations.

Figure 3–4

Seasonal flowrate variations observed in the Western United States in the dry period (May through October) and wet period (November through April). In some locations, the distinction between dry and wet periods is becoming blurred due to global climate change.

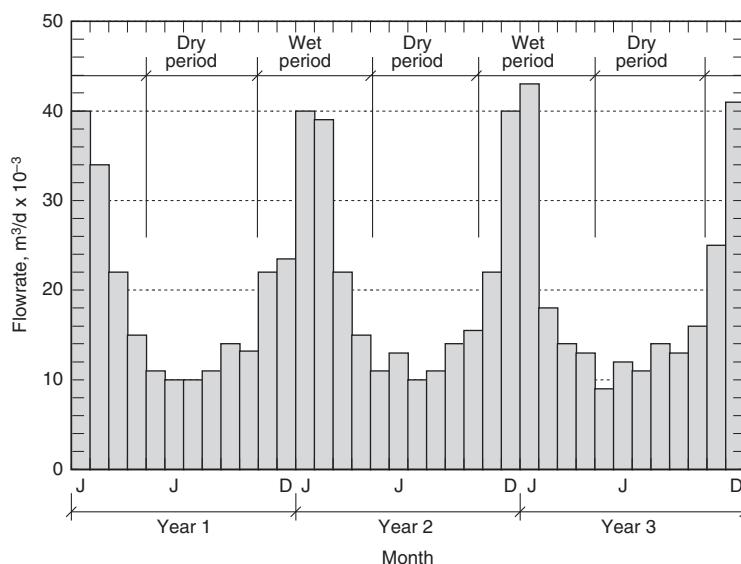
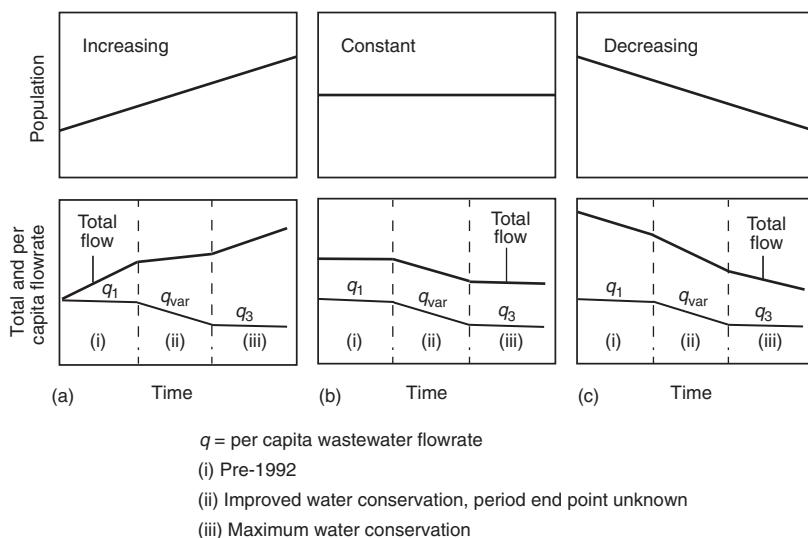


Figure 3-5

Flowrate variations observed in cities in the United States with (a) increasing population, (b) constant population, and (c) decreasing population. The three time periods depicted in these plots are as follows: (i) time period up to about 1990 with relatively limited implementation of water conservation measures, (ii) period following 1990 when the per capita flowrate is decreasing as a result of greater implementation of conservation devices and public awareness, and (iii) period after the time that maximum water conservation has been implemented. The time when maximum conservation has been implemented will vary with each community.



Increasing Population. In cities where the population is increasing, the per capita wastewater flowrates were relatively constant up to the early nineties and the total wastewater flowrate increased in proportion to the population [see Fig. 3-5(a), period (i)]. After the early nineties, the per capita flowrate started to decrease reflecting changes in the plumbing code and the installation of water conserving appliances (e.g., low flush toilets). As more water conservation measures are adopted and water conservation devices and appliances become the norm [Note: the per capita flowrate, q_{var} , is variable in period (ii)], the total flowrate will continue to increase, but at a slower rate of increase [see Fig. 3-5(a), period (ii)]. At some point in the future, water conservation practices and low-flow devices and appliances will be installed in essentially all homes. At that point, the total flow will continue to increase with population, but at a stable and reduced per capita flowrate [see Fig. 3-5(a), period (iii)]. The result of these practices will be that while the constituent mass loading to the wastewater treatment plant will be expected to increase in proportion to the population increase, the concentration of the constituents will be higher than before.

Relatively Constant Population. In cities where the population has been relatively constant, the per capita flowrate was also more or less constant up to the early nineties [see Fig. 3-5(b), period (i)]. After the nineties, the total per capita flowrate started to decrease reflecting the installation of water conserving appliances (e.g., low flush toilets) [see Fig. 3-5(b), period (ii)]. The total flowrate will continue to decrease as more water conservation measures are adopted and low-flow devices and appliances are installed. As discussed above, at some point in the future water conservation devices and appliances will be installed in essentially all of the homes. At that point, the total per capita flowrate will again become more-or-less constant, but at a reduced per capita flowrate [see Fig. 3-5(b), period (iii)]. The constituent mass loading to the wastewater treatment plant is expected to remain relatively constant, but, as above, the concentration of the constituents will increase to reflect the reduced per capita flowrate.

Decreasing Population. In cities with a decreasing population, the per capita wastewater flowrate was relatively constant up to the nineties and the total flowrate decreased in proportion to the population [see Fig. 3-5(c), period (i)]. With the adoption

of water conservation measures, total flowrate has decreased along with the per capita wastewater flowrate [see Fig. 3–5(c), period (ii)]. When water conservation devices and appliances will be installed in essentially all of the homes, the total flow will again decrease with decreasing population, but at a reduced per capita flowrate [see Fig. 3–5(c), period (iii)]. The constituent mass loadings to the wastewater treatment plant are expected to decrease in proportion to the population decrease, while the concentration of the constituents will increase to reflect the reduced per capita flowrate.

Impact of Water Conservation on Future Planning

Because of the variations in flowrates now observed, great care must be taken in the selection of future flowrates for planning purposes. An important aspect in the selection of future flowrates will be to try to estimate the degree to which conservation measures will have already been implemented, the projected maximum level of water conservation that is expected, and the possible future use of satellite water reuse systems. Satellite water reuse systems, including onsite greywater systems, divert wastewater flow to onsite or local reuse applications and effectively reduce the flow of wastewater to the centralized wastewater treatment plant (Tchobanoglous and Leverenz, 2013).

3–2 IMPACT OF COLLECTION SYSTEM ON WASTEWATER FLOWRATES

In addition to the domestic and industrial sources of wastewater, as discussed above, other significant variables in estimating wastewater flowrates are the contributions from infiltration/inflow and from stormwater runoff and snowmelt, where combined collection systems are used. Exfiltration from collection systems can also affect the total quantity of wastewater. The flowrate contributions from these sources are considered in this section.

Infiltration/Inflow

Extraneous flows in collection systems, described as infiltration and inflow, are illustrated on Fig. 3–6 and are defined as follows:

Infiltration. Water entering a collection system from a variety of entry points, including service connections and from the ground through such means as defective pipes, pipe joints, connections, or access port (manhole) walls and joints.

Steady inflow. Water discharged from cellar and foundation drains, cooling-water discharges, and direct connections from springs and swampy areas. This type of inflow is steady and is identified and measured along with infiltration.

Direct inflow. Inflow sources that result from direct stormwater runoff connections to the sanitary or combined collection system and cause an almost immediate increase in wastewater flowrates. Possible sources are roof leaders, yard and areaway drains, access port covers, cross connections from storm drains and although typically confined to combined systems, catch basins. River or tidal intrusion from faulty tide or backwater gates is another source of direct inflow.

Total inflow. The sum of the direct inflow at any point in the system plus any flow leaving from the system upstream through sanitary sewer overflow (SSO) and combined sewer overflow (CSO) discharges, pumping station bypasses, and the like.

Delayed inflow. Delayed inflow, which is not caused by the direct connections listed above, can include foundation drains, the discharge of sump pumps from cellars, and the slowed entry of surface water through access ports (manholes) in ponded areas.

systems are typically separated, but existing combined systems may require evaluation and design relative to increased sanitary wastewater inputs and, mostly, relative to CSO control. The models used for combined sewer systems need to be transient to simulate flow variations during storms. There are a number of models available, with ever-increasing capabilities such as graphical user interfaces, GIS interaction (for model development), multi-user functionality, semi-automatic calibration, graphical results displays, animations, and two-dimensional above-ground flow simulation.

The USEPA Stormwater Management Model (SWMM) is a public domain model that has seen widespread use and upgrades since its development in the 1970s. Several proprietary models or user interfaces are based on SWMM, including PC-SWMM, XP-SWMM, and InfoSWMM. A number of independent proprietary models have also been used widely, including Mike Urban, InfoWorks, and SewerGEMS. These models are also applicable to separate systems where infiltration and inflow (I/I) is similar to runoff, except that it can only be simulated empirically based on flow monitoring data. Software capabilities are now such that large, complex systems can be simulated for extended periods of time. But, for any application, model calibration with flow monitoring data is essential to ensure reliability of the results, which are often the basis of large capital investments.

Model Calibration and Verification. The process of calculating flows in a combined system using any of the computer models normally involves comparison of measured versus predicted flows at selected locations in the system. During model calibration, the model is run with rainfall data collected from one storm, and the calculated results are compared with the observed field results. Estimated input parameters are then adjusted within reasonable bounds to obtain best fit between predictions and measurements. During verification, data sets from other storms are used, and no adjustments of parameters are allowed. This calibration and verification process is essential in assessing the predictive capability of the model.

3-3 ANALYSIS OF WASTEWATER FLOWRATE DATA

Because the hydraulic design of both collection and treatment facilities is affected by variations in wastewater flowrates, the flowrate characteristics have to be analyzed carefully from existing records. In cases where only flowrate data in the collection system are available, it must be recognized that the flowrates may differ somewhat from the flowrate entering the treatment plant because of the flow dampening effect of the wastewater collection system. The statistical analysis of flowrate data and the determination of design parameters are considered in this section.

Statistical Analysis of Flowrate Data

The statistical analysis of wastewater flowrate and constituent concentration data involves the determination of statistical parameters used to quantify a series of measurements. The statistical parameters and graphical techniques used for the analysis of wastewater management data are summarized in Appendix D.

Type of Probability Distribution. The two most common types of probability distributions encountered in the statistical analysis of flowrate data are (1) the standard normal distribution and (2) the log-normal distribution, in which log of the values is distributed normally. If the data are distributed normally, the statistical measure used to define the distribution include the mean, variance, skewness, and coefficient of kurtosis. Skewness and coefficient of kurtosis are needed to quantify the nature of a given distribution. If a distribution is highly skewed, as determined by the coefficient of skewness, normal statistics cannot be used. Another term used to assess the relative variability in the data is *coefficient of*

variation (see Appendix D). If the data are distributed log-normally, the statistical measure used to define the distribution include the geometric mean and the geometric standard deviation (see Appendix D). In general, most flowrate data are distributed log-normally because of the wide range of wet-weather flowrates as compared to the average dry-weather flowrates. The statistical analysis of flowrate data is illustrated in Example 3-4.

EXAMPLE 3-4 Statistical Analysis of Wastewater Flowrate Data Using the following weekly flowrate data obtained, from an industrial discharger, for the dry (May–October) and wet (November–April) periods, determine the statistical characteristics and estimate the maximum weekly flowrate that will occur during each period.

Week	Flowrate, m ³ /wk		Week	Flowrate, m ³ /wk	
	Dry period	Wet period		Dry period	Wet period
1	13,500 ^a	20,000 ^b	14	37,000	51,600
2	25,900	16,250	15	30,100	41,250
3	28,750	40,350	16	21,250	35,000
4	10,750	18,600	17	23,500	30,750
5	12,500	18,300	18	16,750	23,900
6	9850	18,750	19	8350	16,350
7	13,900	21,800	20	18,100	30,200
8	15,100	20,200	21	9250	21,100
9	23,400	23,750	22	9900	21,750
10	21,900	42,500	23	8750	20,800
11	23,700	32,000	24	15,500	24,500
12	18,000	28,300	25	7600	14,400
13	26,400	28,300	26	8700	15,200

^aFirst wk of May.

^bFirst wk of November.

Solution

- Determine the nature of the distribution by plotting the data on arithmetic and log-probability paper.
- Set up a data analysis table with four columns as described below.
 - In column 1, enter the rank serial number starting with number 1
 - In column 2, enter the probability plotting position (see Appendix D)
 - In columns 3 and 4, arrange the weekly flowrate data in ascending order

No.	Plotting position, % ^a	Flowrate, m ³ /wk		No.	Plotting position, % ^a	Flowrate, m ³ /wk	
		Dry period	Wet period			Dry period	Wet period
1	3.7	7600	14,400	14	51.9	16,750	23,750
2	7.4	8350	15,200	15	55.6	18,000	23,900
3	11.1	8700	16,250	16	59.3	18,100	24,500
4	14.8	8750	16,350	17	63.0	21,250	28,300

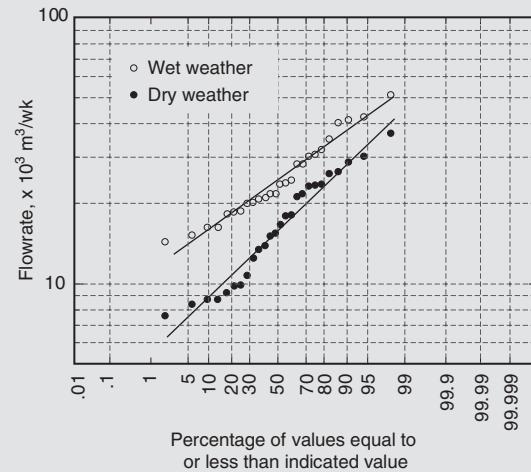
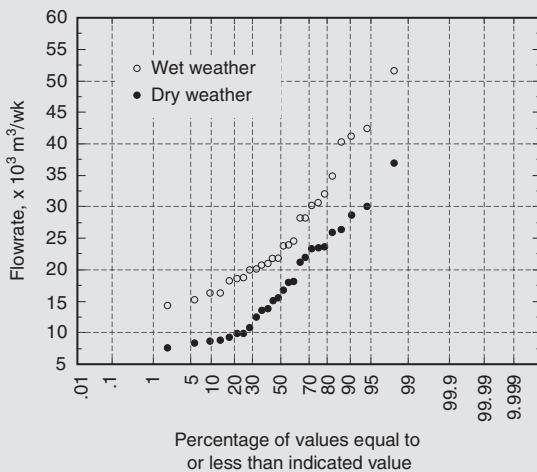
(continued)

(Continued)

No.	Plotting position, % ^a	Flowrate, m ³ /wk		No.	Plotting position, % ^a	Flowrate, m ³ /wk	
		Dry period	Wet period			Dry period	Wet period
5	18.5	9250	18,300	18	66.7	21,900	28,300
6	22.2	9850	18,600	19	70.4	23,400	30,200
7	25.9	9900	18,750	20	74.1	23,500	30,750
8	29.6	10,750	20,000	21	77.8	23,700	32,000
9	33.3	12,500	20,200	22	81.5	24,600	35,000
10	37.0	13,500	20,800	23	85.2	25,900	40,350
11	40.7	13,900	21,100	24	88.8	28,750	41,250
12	44.4	15,100	21,750	25	92.6	30,100	42,500
13	48.2	15,500	21,800	26	96.3	37,000	51,600

^a Plotting position, % = $\left(\frac{m}{n+1} \right) \times 100$, where $n = 26$ [Equation (D-10), Appendix D].

- b. Plot the weekly flowrates expressed in m³/wk versus the plotting position. The resulting plots are presented below. Because the data fall on a straight line on log-probability paper, they are log-normally distributed.



2. Determine the geometric mean for the dry and wet periods and the corresponding geometric standard deviation using Eq. (D-9) from Appendix D.

$$s_g = \frac{P_{84.1}}{M_g} = \frac{M_g}{P_{15.9}}$$

Period	M_g	$P_{84.1}$	s_g
Dry	15,948	25,198	1.58
Wet	24,504	34,391	1.40

Based on the value of the geometric standard deviation, it can be concluded that the observed variation is large.

3. Estimate the probable annual maximum weekly flowrate during the dry and wet periods.
 - a. Determine the probability factor

$$\text{Peak week} = \left(\frac{m}{n + 1} \right) \times 100 = \left(\frac{26}{26 + 1} \right) \times 100 = 96.3$$

- b. Determine the flowrate from the figure given in Step 1b at the 96.3 percentile

Peak dry week flowrate = 35,948 m³/wk

Peak wet week flowrate = 44,900 m³/wk

Comment The statistical analysis of data is important in establishing the design conditions for wastewater treatment plants. The application of statistical analysis to the selection of design flowrates and mass loadings is considered in the following section.

Developing Design Parameters from Flowrate Data

Quantifying the variations in flowrates is important in the design and operation of wastewater treatment plants. Before considering the quantification of flowrate variations, it will be helpful to define some of the parameters that are used commonly to quantify the observed variations.

Flowrate Parameters. The principal parameters used to quantify the observed variations in flowrate are defined in Table 3–11. As will be discussed in Sec. 3–7, these terms are also of importance in the selection and sizing of individual unit treatment processes and operations. As reported in Table 3–11 the parameters used to describe flowrate variations can be grouped into five categories: (1) average values including wet- and dry-weather and annual flows, (2) various peak values, (3) maximum values, (4) minimum values, and (5) sustained values. Using a 1-d record for the purposes of illustration, the following parameters are identified on Fig. 3–8: average daily flowrate, instantaneous peak, peak hour, minimum hour, minimum, and sustained 13.5-h flowrate. Obviously, a two- or three-year record is needed to establish the parameters shown on Fig. 3–8 and the long-term parameters identified in Table 3–11.

Flowrate Ratios. Because it is difficult to compare numerical peak flow values from different wastewater treatment plants, peak flowrate values are normalized by dividing by the long-term average flowrate. The resultant ratio, known as a peaking factor, is defined as follows.

$$\text{Sustained peaking factor, PF} = \frac{\text{peak flowrate (e.g., hourly, daily, monthly)}}{\text{long-term average flowrate}} \quad (3-1)$$

Peaking factors are applied most frequently to determine the peak hourly flowrate. For example, the sustained hourly peaking factor relative to the average flowrate for the daily flowrate record given on Fig. 3–8 is 1.72 [(183 m³/d)/(106 m³/d)]. Where flowrate records are available, at least three years of data should be analyzed to define the peak to average day peaking factor. The peaking factor is particularly useful in estimating the maximum hydraulic conditions that might occur and have to be accommodated. Peaking factors can also be applied to mass loadings.

Table 3-11

Parameter	Description^b
Average dry weather flow (ADWF)	The average of the daily flow data for dry weather periods ^c
Average wet weather flow (AWWF)	The average of the daily flow for wet weather periods ^c
Average daily flow	The average flow occurring over a 24-h period based on total annual flowrate data
Instantaneous peak flowrate	Highest recorded flowrate occurring for a period consistent with the reporting period for the metering equipment. In many situations the recorded peak flow may be considerably below the actual peak flow because of metering and recording equipment limitations
Peak hourly flowrate	The peak sustained hourly flowrate occurring during a 24-h period based on annual operating data (see also Instantaneous peak flow)
Maximum day flow	The maximum flow occurring over a 24-h period based on annual operating data
Maximum monthly flowrate	The maximum daily flow sustained for a period of 1 mo in the record examined. In practice, the maximum month is typically the maximum 30-d value that occurs in the reporting period per the NPDES permit. Although not statistically correct, it is the value that is reported
Minimum hourly flowrate	The minimum sustained hourly flowrate occurring over a 24-h period based on annual operating data
Minimum daily flow	The minimum flow that occurs over a 24-h period based on annual operating data
Minimum monthly flow	The minimum daily flow that occurs over a period of 1 mo based on annual operating data (see Maximum month above)
Sustained flow or load	The flow or mass loading rate sustained or exceeded for a specified period of time (e.g., 1 h, 1 d, or 1 mo) based on annual operating data

^a Adapted in part from Crites and Tchobanoglous (1998).

^b The definitions can also be applied to constituent mass loadings.

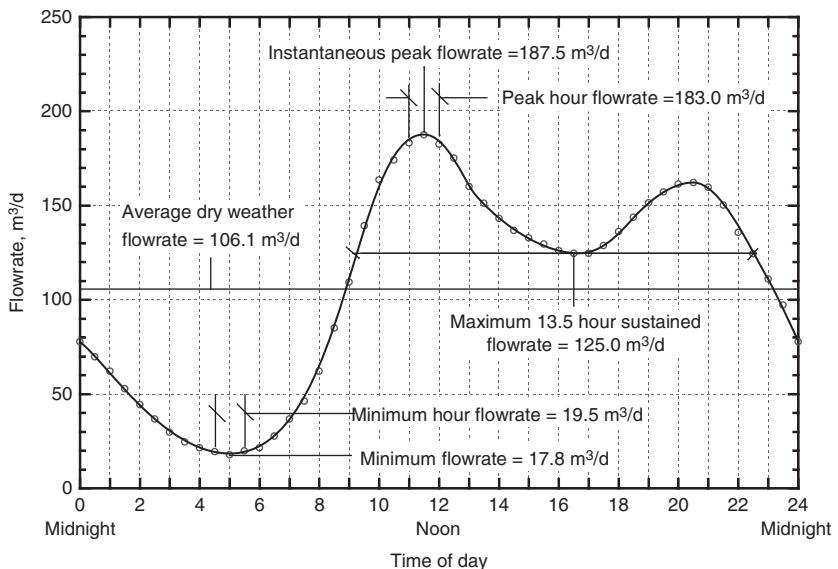
^c In some parts of the country, the distinction between wet and dry periods is becoming blurred due to climate change effects.

Observed Variability in Influent Flowrates

The influent flowrate to a treatment facility, as noted previously, is dependent on factors such as the time of day, season, size and characteristics of the contributing population, and infiltration/inflow to and exfiltration from the collection system. In large cities the wastewater flow is distributed more evenly because there is a greater diversity of lifestyles and a high amount of activity at night. In contrast, wastewater treatment plants used for small residential communities are more likely to experience higher peak flow relative to mean flow values. Peaking factors, as discussed above, are used to estimate the maximum values that would be expected. One method used to characterize the variability of wastewater parameters and treatment processes is the use of the geometric standard deviation, s_g , as computed in Example 3–5. The value of s_g can be used to approximate an entire distribution of all expected values if a mean value is known or can be estimated. As discussed in

Figure 3-8

Identification of flowrate parameters based on variations observed on a typical weekday.



Appendix D, the greater the numerical value of s_g , the greater the observed range in the measured values.

Peaking factors are also related to s_g by specification of a frequency value. The peaking factor is calculated as the value at a given frequency divided by the mean value. For example, the peak day value, which corresponds to one event per year, is the value that occurs at a frequency of 99.7 percent $[(364/365) \times 100]$. The typical ranges of observed values for s_g for influent flowrates for small, medium, and large capacity wastewater treatment plants are given in Table 3-12. The relationship between s_g values and the peaking factors for peak day, week, and month can be determined using the curves given on Fig. 3-9. An example of the use of the s_g value and the curves given on Fig. 3-9 is illustrated subsequently in Example 3-9.

Figure 3-9

Relationship of s_g values to peaking factors for peak day, week and month: (a) diagram for facilities with large peak to mean variations and (b) expanded portion of diagram (a) for facilities with small peak to mean variations.

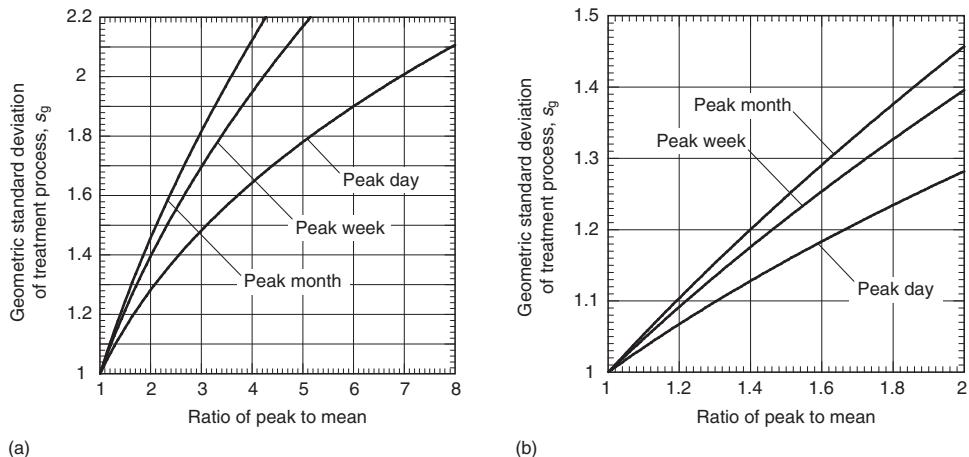


Table 3-12

Ranges of geometric standard deviations (s_g) for influent wastewater flowrates and selected constituents observed at small, intermediate, and large wastewater treatment facilities

Parameter	Ranges of s_g values for typical wastewater treatment facilities ^a					
	Small ^b		Medium ^c		Large ^d	
	Range	Typical	Range	Typical	Range	Typical
Flowrate	1.4–2.0	1.6	1.1–1.5	1.25	1.1–1.2	1.15
BOD	1.4–2.1	1.6	1.3–1.6	1.3	1.1–1.3	1.27
COD	1.5–2.2	1.7	1.4–1.8	1.4	1.1–1.5	1.30
TSS	1.4–2.1	1.6	1.3–1.6	1.3	1.1–1.3	1.27

^a Excluding systems with large amounts of infiltration in the collection system.

^b Flowrate of 4000–40,000 m³/d (1–10 Mgal/d).

^c Flowrate of 40,000–400,000 m³/d (10–100 Mgal/d).

^d Flowrate >400,000 m³/d (>100 Mgal/d).

3-4 ANALYSIS OF WASTEWATER CONSTITUENTS

The complete analysis of wastewater data involves the determination of the flowrate, constituent concentrations, and constituent mass loading and their variations. The analysis may involve determining the concentrations of specific constituents, mass loadings (flowrate times concentration), or sustained mass loadings (loadings that occur over a defined period of time). From the standpoint of treatment processes, one of the most serious deficiencies results when the design of a treatment plant is based on average flowrates and average BOD and TSS loadings, with little or no recognition of peak conditions and other factors that may affect the average values.

In many communities, peak influent flowrates and BOD and TSS loading rates can reach two or more times average values. It must also be emphasized that, in nearly all cases, peak flowrates and BOD and TSS loading rates do not occur at the same time. Analysis of current records is the best method of arriving at appropriate peak and sustained mass loadings. The principal factors responsible for loading variations are (1) the established habits of community residents, which cause short-term (hourly, daily, and weekly) variations; (2) seasonal conditions, which usually cause longer-term variations; and (3) industrial activities, which cause both long- and short-term variations.

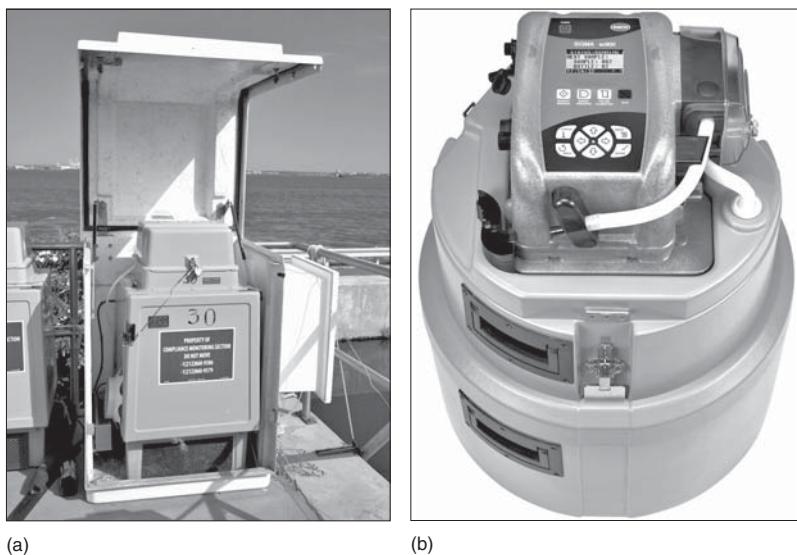
Wastewater Constituents Discharged By Individuals

The physical, chemical, and biological characteristics of wastewater vary throughout the day. An adequate determination of the waste characteristics will result only if the sample tested is representative. Typically, composite samples made up of portions of samples collected at regular intervals during a day are used (see Fig. 3–10, see also Fig. 2–2 in Chap 2). The amount of liquid collected for each sample is proportional to the flowrate at the time the sample was collected. Adequate characterization of mass of the various constituents discharged by individuals is of fundamental importance in the design of treatment and disposal processes.

In the past, it was quite common practice to report measured constituent concentrations, most commonly as mg/L. In the twenty-first century this practice is less useful because, as noted previously in the discussion of flowrates, the per capita flowrates are decreasing as a result of conservation and the use of water conserving appliances and fixtures. Use of current measured concentration values for future projections can be misleading because future concentration values will be understated, due to the impacts of

Figure 3-10

Samplers used to collect wastewater samples:
 (a) refrigerated, locked, and enclosed sampler used for the collection of treatment plant effluent compliance samples (see Fig. 2-2 for view of internal workings of sampler) and
 (b) portable sampler used to collect individual hourly samples throughout a day to assess process performance and to study potential benefits of process modifications (courtesy of Hach Company, www.hach.com). Portable samplers are also used to conduct collection system evaluations.



continued water conservation. A more rational approach is to consider the constituent mass discharge rate on a per capita basis. With the availability of such data, the corresponding wastewater concentrations can be determined easily. In what follows typical constituent mass data are presented for the United States and for foreign countries.

Per Capita Mass Constituent Discharges in the United States. Typical data on the total mass of waste discharged per person per day (dry weight basis) from individual residences are reported in Table 3-13. The data presented in Table 3-13 have been gathered from numerous sources. The values reported in Table 3-13 represent the combined total for feces and urine. The reason that the range of values for the individual constituents is much greater than that observed in other countries (see Table 3-14) is due to the fact that about 37.5 percent of all adults in the United States are classified as obese (Ogden et al., 2012). The total number of pathogenic organisms discharged will depend on whether an individual is ill and is shedding pathogens. If one or more members of a family are ill and shedding pathogens, the number of measured organisms can increase by several orders of magnitude. The data given in Table 3-13 will be used subsequently to determine constituent concentrations based on various volumes of water. In the future, it is anticipated that constituent mass per capita data, as given in Table 3-13, will be used as a basis for determining expected constituent concentrations for the design of treatment plant upgrades as well as for new plants.

Per Capita Mass Constituent Discharges in Countries Outside of the United States. The amounts of waste discharged by individuals in other countries can vary significantly due to cultural and socioeconomic differences. Constituent data for 12 other countries as compared to the United States are reported in Table 3-14. The difference in the range of constituent values for other countries as compared to the United States is striking and, as described above, is largely cultural. Water use by individuals also differs significantly in other countries; in most cases the quantities used are significantly less. Consequently, the strength of the wastewater may be much higher than found in the United States and can affect significantly wastewater treatability. In some cases, the wastewater composition may be high in organic content but low in alkalinity and thus cannot be nitrified fully. Concentrations of TSS and BOD for another culture compared to the United States are illustrated in Example 3-5.

Table 3-13**Quantity of waste discharged by individuals on a dry weight basis in the United States^a**

Constituent (1)	Range (2)	Value, lb/capita·d		Value, g/capita·d		
		Typical without ground up kitchen waste (3)	Typical with ground up kitchen waste (4)	Range (5)	Typical without ground up kitchen waste (6)	Typical with ground up kitchen waste (7)
BOD ₅	0.11–0.26	0.15	0.20	50–120	70	93
COD	0.30–0.65	0.40	0.50	110–295	180	230
TSS	0.13–0.33	0.15	0.19	60–150	70	87
NH ₃ as N	0.011–0.026	0.017	0.017	5–12	7.6	7.9
Organic N as N	0.009–0.022	0.012	0.013	4–10	5.4	6.0
TKN ^{b,c} as N	0.020–0.040	0.029	0.031	9–18	13	13.9
Organic P as P	0.002–0.004	0.0026	0.0029	0.9–1.8	1.2	1.3
Inorganic P as P	0.001–0.006	0.0020	0.0020	0.50–2.7	0.90	0.90
Total P ^c as P	0.003–0.010	0.0046	0.0048	1.5–4.5	2.1	2.2
Potassium, K ^c	0.009–0.015	0.013	0.014	4–7	6.0	6.2
Oil and grease	0.022–0.077	0.062	0.070	10–35	28.0	32

^a Adapted in part from Tchobanoglous et al. (2003).^b TKN is total Kjeldahl nitrogen.^c The percentage distribution of the total nitrogen, phosphorus, and potassium in feces and urine is 14 to 17, 28 to 35, and 12 to 18 percent, respectively.**Table 3-14****Typical wastewater constituent data for various countries^a**

Country/ constituent	BOD, g/capita·d	TSS, g/capita·d	TKN, g/capita·d	NH ₃ -N, g/capita·d	Total P, g/capita·d
Brazil	55–68	55–68	8–14	ND	0.6–1
Denmark	55–68	82–96	14–19	ND	1.5–2
Egypt	27–41	41–68	8–14	ND	0.4–0.6
Germany	55–68	82–96	11–16	ND	1.2–1.6
Greece	55–60	ND	ND	8–10	1.2–1.5
India	27–41	ND	ND	ND	ND
Italy	49–60	55–82	8–14	ND	0.6–1
Japan	40–45	ND	1–3	ND	0.15–0.4
Palestine ^b	32–68	52–72	4–7	3–5	0.4–0.7
Sweden	68–82	82–96	11–16	ND	0.8–1.2
Turkey	27–50	41–68	8–14	9–11	0.4–2
Uganda	55–68	41–55	8–14	ND	0.4–0.6
United States ^c	50–120	60–150	9–18	5–12	1.5–4.5

^a Adapted from Tchobanoglous et al. (2003).^b West Bank and Gaza Strip.^c From Table 3-13.

Characteristics of Urine. Because there is increased interest in urine separation and its implications for wastewater treatment and recovery for use as a fertilizer, typical composition data for urine are reported in Table 3-15. The values in the table are identified as approximate because the composition of urine will vary for each individual depending on the amount of water ingested, the amount and composition of the food ingested, the time of day, general health, blood pressure, and the temperature. As noted earlier the bulk of the nutrients (i.e., nitrogen, phosphorus, and potassium) in wastewater are present in urine.

Table 3-15
Approximate properties and overall composition of urine^{a,b}

Constituent	Formula	Value, mg/L ^c	
		Range	Typical
Organic molecules			
Urea	CON ₂ H ₄	9000–23,000	20,000
Creatinine	C ₄ H ₉ N ₃ O ₂	900–1200	1000
Uric acid	C ₅ H ₄ N ₄ O ₃	200–400	300
Trace organics			
Inorganic elements			
Ammonium	NH ₄ ⁺	400–600	500
Bicarbonate	HCO ₃ ⁻	20–600	300
Calcium	Ca ²⁺	100–300	150
Chloride	Cl ⁻	1600–8000	1900
Magnesium	Mg ²⁺	80–120	100
Potassium	K ⁺	1200–1700	1500
Sodium	Na ⁺	5000–7000	6000
Inorganic compounds			
Sulphates	SO ₄ ²⁻	1600–2000	1800
Phosphates	H ₂ PO ₄ ⁻ , HPO ₄ ²⁻ PO ₄ ³⁻	1000–1500	1200
Overall composition			
Urea	CON ₂ H ₄	16,000–24,000	20,000
Organic compounds	various	4000–8000	6000
Organic ammonium salts	various	4000–6000	5000
Inorganic salts	various	12,000–16,000	15,000
Other	various	2500–6000	4000
Solids	%	4–7	5
Water	%	93–96	95
Density	g/mL	1.002–1.030	1.010

^a Adapted in part from Putnam (1971); Ryan (1966); Gotaas (1956); and numerous other sources.

^b Values in the table are identified as approximate because the composition of urine will vary for each individual depending on the amount of water ingested, the amount and composition of the food ingested, the time of day, general health, blood pressure, and the temperature.

^c Per capita amount of urine discharged per day will vary from 0.8 to 1.3 L/capita·d.

Constituent Concentrations Based on Individual Mass Discharges

The corresponding constituent concentrations for the United States, assuming the per capita mass quantities of waste constituents given in Table 3–13, are reported in Table 3–16. In preparing Table 3–16, it was assumed that (1) 25 percent of the homes were equipped with kitchen food waste grinders and (2) the constituent mass quantities were diluted in 190 and 460 L (50 and 120 gal) of water. Two different dilutions are used to illustrate the impact that the dilution has on the resulting constituent concentrations.

The method used to determine the constituent concentration values is illustrated in Example 3–5.

EXAMPLE 3–5

Estimate Waste Constituent Concentration Using data from Table 3–14, determine the BOD, TSS, and ammonia nitrogen concentrations for the West Bank and the Gaza Strip assuming the water supply is intermittent and the wastewater flowrate is 60 L/capita·d.

Solution

1. From Table 3–15, use the following average constituent contributions:
 - a. BOD = 50 g/capita·d
 - b. TSS = 62 g/capita·d
 - c. NH₃-N = 4 g/capita·d
2. Compute BOD concentration.

$$\text{BOD} = \left[\frac{(50 \text{ g/capita}\cdot\text{d})}{(60 \text{ L/capita}\cdot\text{d})} \right] \left(\frac{10^3 \text{ L}}{1 \text{ m}^3} \right) = 833 \text{ g/m}^3$$

3. Compute TSS concentration.

$$\text{TSS} = \left[\frac{(62 \text{ g/capita}\cdot\text{d})}{(60 \text{ L/capita}\cdot\text{d})} \right] \left(\frac{10^3 \text{ L}}{1 \text{ m}^3} \right) = 1033 \text{ g/m}^3$$

4. Compute the NH₃-N concentration.

$$\text{NH}_3\text{-N} = \left[\frac{(4 \text{ g/capita}\cdot\text{d})}{(60 \text{ L/capita}\cdot\text{d})} \right] \left(\frac{10^3 \text{ L}}{1 \text{ m}^3} \right) = 66.7 \text{ g/m}^3$$

Comment

In many parts of the world where water usage is low, constituent concentrations for BOD and TSS may range up to 1000 g/m³ (mg/L). In the above example, the concentrations of BOD and TSS are nearly 2 to 4 times the BOD and TSS concentrations found typically in the United States (see Table 3–16). Ammonia nitrogen (NH₃-N) concentrations may also be higher, perhaps on the order of two times or more of those in the United States.

Mineral Increase Resulting from Water Use

Data on the increase in the mineral content of wastewater resulting from water use and the variation of the increase within a collection system are especially important in evaluating the reuse potential of wastewater. Typical data on the incremental increase in mineral content that can be expected in municipal wastewater resulting from domestic use are reported in Table 3–17. Increases in the mineral content of wastewater may be due in part from addition of highly mineralized water from private wells and groundwater and from

Table 3-16

Typical unit loading factors and expected wastewater constituent concentrations from individual residences in the United States^a

Constituent	Typical value ^b , g/capita·d	Concentration, mg/L	
		Volume, L/capita·d (gal/capita·d)	190(50) 380(100)
BOD ₅	76.0	399.0	199.0
COD	193.0	1013.0	507.0
TSS	74.0	391.0	195.0
NH ₃ as N	7.7	40.0	20.0
Org. N as N	5.5	29.0	14.0
TKN as N	13.2	70.0	35.0
Org. P as P	1.2	6.4	3.2
Inorg. P as P	0.9	4.7	2.4
Total P as P	2.1	11.0	5.6
Potassium	6.1	32.0	16.0
Oil and grease	29.0	153.0	76.0

^a Adapted from Tchobanoglou et al. (2003).

^b Data from Table 3-13, Columns 6 and 7, assuming 25 percent of the homes have kitchen waste food grinders.

For example, BOD₅ = [70 + (93 - 70)(0.25)] mg/L = 76 mg/L.

industrial use. Domestic and industrial water softeners also contribute significantly to the increase in mineral content and, in some areas, may represent the major source. Occasionally, water added from private wells and groundwater infiltration will (because of its high quality) serve to dilute the mineral concentration in wastewater.

Composition of Wastewater in Collection Systems

Typical data on the composition of untreated domestic wastewater as found in wastewater collection systems (in the United States) are reported in Table 3-18. The data presented in Table 3-18 for medium strength wastewater are based on an average flow of 460 L/capita·d (120 gal/capita·d) and include constituents added by commercial, institutional, and industrial sources. Typical concentrations for low-strength and high strength wastewater, which reflect different amounts of infiltration, are also given. Because there is no “typical” wastewater, it must be emphasized that the typical data presented in Table 3-18 *should only be used as a guide*.

Variations in Constituent Concentrations

Several types of constituent concentrations can occur depending upon the characteristics of the contributors to the wastewater collection system.

Short-Term Variation in Constituent Concentrations. Constituent concentrations variations change significantly during the course of a day. An example of typical

Table 3-17
Typical mineral increase from domestic water use

Constituent	Typical increment range^{a, b}	
	Mass, g/capita·d	Concentration, mg/L^c
Anions:		
Bicarbonate (HCO_3^-)	23–46	60–121
Carbonate (CO_3^{2-})	0–5	0–13
Chloride (Cl)	9–23	24–60
Sulfate (SO_4^{2-})	7–14	18–37
Cations:		
Calcium (Ca)	3–7	8–18
Magnesium (Mg)	2–5	4–13
Sodium (Na)	18–32	47–84
Other constituents		
Aluminum (Al)	0.04–0.09	0.11–0.24
Boron (B)	0.04–0.09	0.11–0.24
Fluoride (F)	0.09–0.2	0.24–0.53
Manganese (Mn)	0.09–0.2	0.24–0.53
Silica (SiO_2)	0.9–5	2.4–13
Total alkalinity (as CaCO_3)	28–55	74–145
Total dissolved solids (TDS)	69–175	182–460

^a Values do not include commercial and industrial additions.

^b Excluding the addition from domestic water softeners.

^c Based on 380 L/capita·d (100 gal/capita·d) as used in Tables 3-16 and 3-18.

Note: mg/L = g/m³.

variations in BOD and TSS in domestic wastewater are shown on Fig. 3-11. In general, the variations in BOD and TSS follow the flowrate variation. The peak BOD (organic matter) concentration typically occurs in the evening, whereas the peak TSS concentration occurs in the morning. The amplitude of the BOD and TSS variations depends on the size and characteristics of the community. Also, the time that the peak value occurs will depend on whether it is a weekday or the weekend [see Fig. 3-3(b)].

Seasonal Variation in Constituent Concentrations. For domestic flow only, and neglecting the effects of infiltration, the unit (per capita) loadings and the strength of the wastewater from most seasonal sources, such as resorts, will remain about the same on a daily basis throughout the year even though the total flowrate varies. The total mass of BOD and TSS of the wastewater, however, will increase directly with the population served.

Infiltration/inflow, as discussed earlier in this chapter, is another source of water flow into the collection system. In most cases, the presence of this extraneous water tends to decrease the concentrations of BOD and TSS, depending on the characteristics of the water entering the sewer. In some cases, concentrations of some inorganic constituents may actually increase where the groundwater contains high levels of dissolved constituents.

Table 3-18
Typical composition of untreated domestic wastewater

Constituent	Unit	Concentration^b		
		Low strength	Medium strength	High strength
Solids, total (TS)	mg/L	537	806	1612
Dissolved, total (TDS)	mg/L	374	560	1121
Fixed	mg/L	224	336	672
Volatile	mg/L	150	225	449
Suspended solids, total (TSS)	mg/L	130	195	389
Fixed	mg/L	29	43	86
Volatile	mg/L	101	152	304
Settleable solids	mL/L	8	12	23
Biochemical oxygen demand, 5-d, 20°C BOD	mg/L	133	200	400
Total organic carbon (TOC)	mg/L	109	164	328
Chemical oxygen demand (COD)	mg/L	339	508	1016
Nitrogen (total as N)	mg/L	23	35	69
Organic	mg/L	10	14	29
Free ammonia	mg/L	14	20	41
Nitrites	mg/L	0	0	0
Nitrates	mg/L	0	0	0
Phosphorus (total as P)	mg/L	3.7	5.6	11.0
Organic	mg/L	2.1	3.2	6.3
Inorganic	mg/L	1.6	2.4	4.7
Potassium	mg/L	11	16	32
Chlorides ^c	mg/L	39	59	118
Sulfate ^c	mg/L	24	36	72
Oil and grease	mg/L	51	76	153
Volatile organic compounds (VOCs)	µg/L	<100	100–400	>400
Total coliform	No./100 mL	10 ⁶ –10 ⁸	10 ⁷ –10 ⁹	10 ⁷ –10 ¹⁰
Fecal coliform	No./100 mL	10 ³ –10 ⁵	10 ⁴ –10 ⁶	10 ⁵ –10 ⁸
<i>Cryptosporidium</i> oocysts	No./100 mL	10 ⁻¹ –10 ¹	10 ⁻¹ –10 ²	10 ⁻¹ –10 ³
<i>Giardia lamblia</i> cysts	No./100 mL	10 ⁻¹ –10 ²	10 ⁻¹ –10 ³	10 ⁻¹ –10 ⁴

^a Adapted from Tchobanoglous et al. (2003).

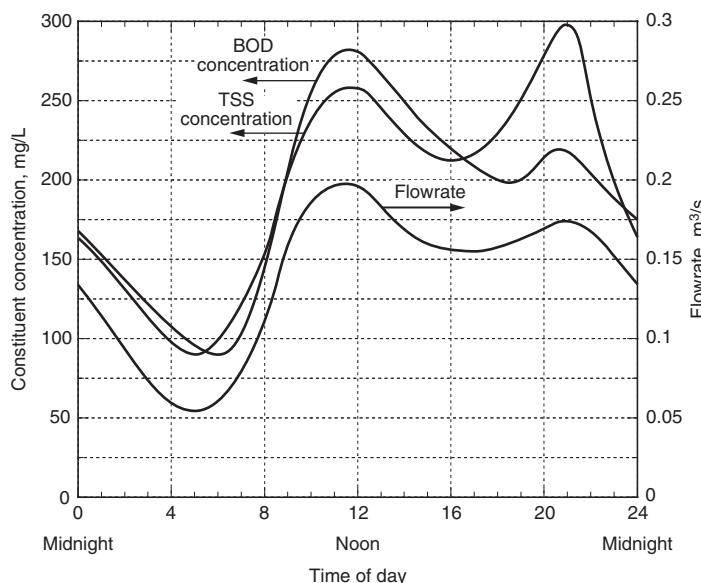
^b Low strength is based on an approximate wastewater flowrate of 570 L/capita·d (150 gal/capita·d). Medium strength is based on an approximate wastewater flowrate of 380 L/capita·d (100 gal/capita·d). High strength is based on an approximate wastewater flowrate of 190 L/capita·d (50 gal/capita·d).

^c Values should be increased by amount of constituent present in domestic water supply.

Note: mg/L = g/m³.

Figure 3-11

Typical hourly variations observed in flow and BOD and TSS concentrations in domestic wastewater.



Variations in Industrial Wastewater. The composition of wastewater from industrial operations varies widely depending on the function and activity of the particular industry. Examples of the constituent concentration variability are illustrated in Table 3–19. From these examples, it can be observed that flow values and water quality measurements may vary by several orders of magnitude over a period of a year. Because of this variation, it is often difficult to define “typical operating conditions” for industrial activities.

The concentrations of both BOD and TSS, as well as a wide range of other constituents, in industrial wastewater can vary significantly throughout the day. For example, the BOD and TSS concentrations contributed from vegetable-processing facilities during the noon wash-up period may far exceed those contributed during working hours. Problems with high short-term loadings most commonly occur in small treatment plants that have limited reserve capacity to handle these so-called “shock loadings.” The seasonal impact of industrial wastes such as canneries can cause both the flow and BOD loading to increase from two to five times average conditions.

If industrial wastes are to be discharged to the collection system for treatment in a municipal wastewater facility, it will be necessary to characterize the wastes adequately to identify the ranges in constituent concentrations and mass loadings. Such characterization is also needed to determine if pretreatment is required before the waste is permitted to be discharged into the collection system. If pretreatment is needed, the effluent from the pretreatment facilities must also be characterized. Further, any proposed future process changes should also be assessed to determine what effects they might have on the wastes to be discharged. Where data are not available, every effort should be made to obtain information from similar facilities. With sufficient characterization of the wastewater from industrial discharges, suitable pretreatment facilities can be provided and plant upsets can be avoided.

Variations in Constituent Concentrations in Combined Collection Systems. Flowrates, constituent concentrations, and mass loads emanating from combined collection systems can vary widely from community to community, season to season,

Table 3-19

Typical examples of the range of effluent flowrates and constituent concentrations for two industrial activities^a

Constituent	Unit	Wool textile mill		Tomato cannery	
		Annual average	Daily maximum	Peak season ^b	Off season ^c
Flowrate	m ³ /d	—	—	4164–22,300	1140–6400
pH	—	5.92 ^d	—	7.2–8.0	7.2–8.0
BOD	mg/L	90.7	169	460–1100	29–56
COD	mg/L	529	1240	—	—
SS	mL/L	—	—	6–80	0.5–2.2
TSS	mg/L	93.4	860	270–760	69–120
TDS	mg/L	—	—	480–640	360–520
Nitrate-N	mg/L	—	—	0.4–5.6	0.1–2.2
Ammonia-N	mg/L	8.1	54	—	—
Phosphorus	mg/L	—	—	1.5–7.4	0.3–3.9
Sulfate	mg/L	—	—	15–23	7.1–9.9
DO	mg/L	—	—	0.9–3.8	1.6–9.8
Oil and grease	mg/L	27.4	45.2	—	—
Temperature	°C	—	—	18–23	13–19

^a Adapted from Tchobanoglous et al. (2003).

^b Peak season is from early July to late September, when fresh-harvest tomatoes are canned.

Treatment consists of screening and sedimentation.

^c Off-season is from November to June, when canned tomatoes are remanufactured into tomato paste, tomato sauce, and other tomato products (e.g., salsa, ketchup, spaghetti sauce).

Treatment typically consists of screening, aeration, and sedimentation.

^d Median value.

Note: m³/d × 0.264 × 10⁻³ = Mgal/d.

storm to storm and within a storm. Typical factors influencing the characteristics of wastewater from combined collection systems are shown in Table 3–20. Example variations of BOD, TSS, and fecal coliform measured in a combined system are shown on Fig. 3–12, during and after a storm event. As shown, the BOD and fecal coliform bacteria concentrations are low during the storm when runoff flows are high. After the storm, when runoff subsides and the flow consists primarily of wastewater, concentrations rise significantly. When this rise occurs, it can be concluded that the BOD and fecal coliform concentrations in the stormwater are significantly lower than in the wastewater component. The characteristics of combined wastewater and stormwater as compared to municipal wastewater are shown in Table 3–21.

Unlike BOD and fecal coliform bacteria, TSS concentrations rise slightly during the storm and remain unchanged after the storm, indicating that TSS concentrations from stormwater runoff and wastewater are similar. The slight rise in the TSS concentration during the peak flow may be due to a phenomenon common to many combined sewer systems known as the “first flush.” The first flush has often been observed following the initial phase of a rainfall event in which much of the accumulated surface contaminants are washed into the combined system. In combined collection systems, the increased flows may be capable of resuspending material deposited previously during low flow periods.

Table 3–20
Typical factors influencing the characteristics of combined wastewater

Parameter	Quantity-related factors	Quality-related factors
Precipitation	Rainfall depth (volume) Storm intensity Storm duration	Antecedent dry weather Atmospheric quality
Wastewater sources	Peak flowrate and variability Land use	Contributing sources (domestic, industrial, commercial, etc.)
Drainage basin characteristics	Size, shape, slope, time of concentration Land use Impervious area Soil characteristics Stormwater management practices	Effectiveness of best management practices/minimum controls with respect to street cleaning and litter control Pollutant buildup and wash-off Stormwater management practices
Sewer system, interceptor design and condition	Pipe size, slope and shape Quantity of base infiltration Number and location of inflow sources (catch basins, roof leaders, etc.) Surcharging or backwater conditions Type of flow regulation or diversion structures including pumping station overflows Capacity reduction from sediment build-up River or tidal inflow	Effectiveness of best management practices/minimum controls with respect to catch basin and sewer cleaning Pollutant buildup and resuspension Chemical and biological transformations

Figure 3–12

Typical variations of flowrate, BOD, TSS, and fecal coliform in a combined collection system during a storm event.

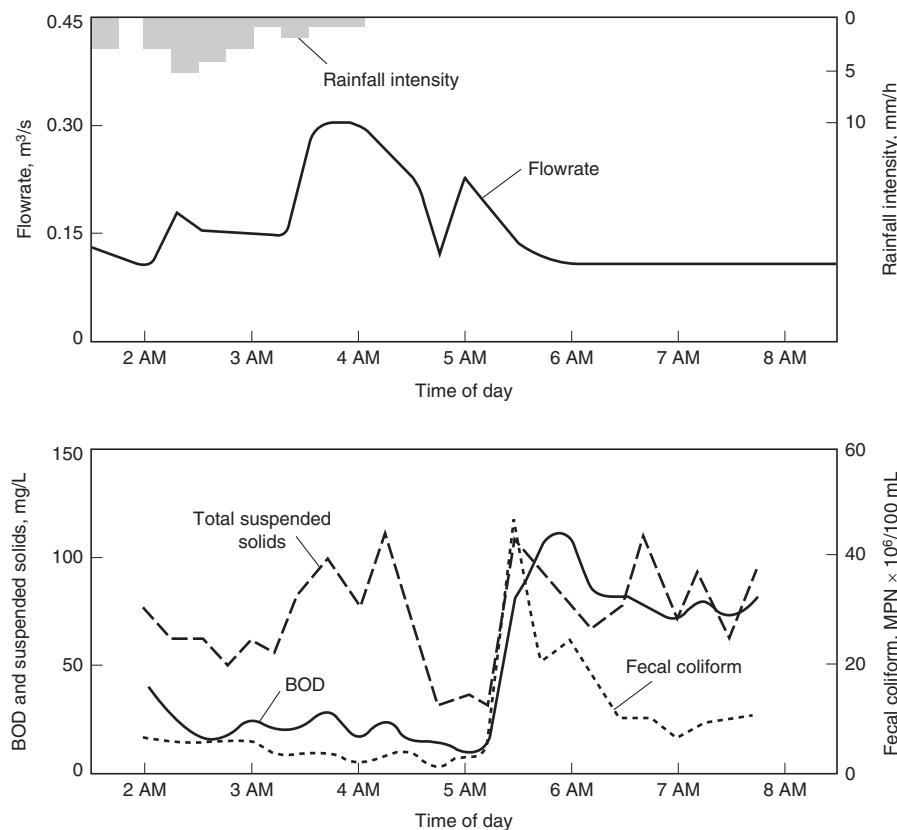


Table 3-21**Comparison of characteristics of combined wastewater with other sources^a**

Parameter	Unit	Rainfall	Stormwater runoff	Combined wastewater	Municipal wastewater
Total suspended solids, TSS	mg/L	<1	67–101	270–550	120–400
Biochemical oxygen demand, BOD	mg/L	1–13	8–10	60–220	110–350
Chemical oxygen demand, COD	mg/L	9–16	40–73	260–480	250–800
Nitrogen (as N)					
Total Kjeldahl nitrogen, TKN	mg/L		0.40–1.00	4–17	20–70
Nitrate, NO ₃	mg/L		0.05–1.0	0.48–0.91	0
Phosphorus (total as P)	mg/L	0.02–0.15	0.67–1.66	1.2–2.8	4–12
Metals					
Copper, Cu	µg/L		27–33		
Lead, Pb	µg/L	30–70	30–144	140–600	
Zinc, Zn	µg/L		135–226		
Fecal coliform bacteria	MPN/100 mL		10 ³ –10 ⁴	10 ⁵ –10 ⁶	10 ⁵ –10 ⁸

^a Adapted from Tchobanoglous et al. (2003).

Together, the resuspended material and contaminants washed off surfaces result in high contaminant concentrations. Factors known to contribute to the magnitude and frequency of the first flush effect include combined sewer slopes, street and catch basin cleaning frequency and design, rainfall intensity, duration, and surface buildup of debris and contaminants.

Wastewater from combined collection systems usually contains more inorganic matter than wastewater from sanitary collection systems because of the larger quantities of stormwater runoff that enter the combined sewer system. This observation is particularly true in snowbelt states where sand is used to treat the streets during snow and ice conditions.

Statistical Analysis of Constituent Concentrations

The statistical analysis of constituent concentration data is essentially the same as described previously for flowrate data in Sec. 3-3 and illustrated in Example 3-4. The most common type of probability distribution encountered in the statistical analysis of constituent concentration data is the log-normal distribution, in which log of the constituent concentration is normally distributed. In general, wastewater constituents that can be altered biologically are distributed log-normally. Inorganic constituents, such as chloride and sulfate, can be distributed both normally or log-normally.

Observed Variability in Influent Constituent Concentrations

The influent constituent concentrations to a treatment facility, as noted above, and illustrated on Fig. 3-12, are also quite variable depending on the time of day, season, size and

characteristics of the contributing population, and infiltration to and exfiltration from the collection system. The variability observed in constituent concentrations can be quantified using the geometric standard deviation, s_g , as computed in Example 3–4. The value of s_g can be used to approximate an entire distribution of all expected values if a mean value is known or can be estimated. For example, the peak day value, which corresponds to one event per year, is the value that occurs at a frequency of 99.7 percent $[(364/365) \times 100]$. The typical range of observed values for s_g for influent BOD, TSS, and COD concentrations for small-, medium-, and large-capacity wastewater treatment plants are given in Table 3–12, presented previously. The application of the s_g values given in Table 3–12 is illustrated subsequently in Example 3–9.

3–5 ANALYSIS OF CONSTITUENT MASS LOADING DATA

The analysis of wastewater constituent data involves the determination of simple average or flow-weighted average concentrations, mass loadings rates, and sustained peak mass loading rates.

Simple Average

The simple or arithmetic average of a number of individual measurements is given by

$$\bar{x} = \frac{1}{n} \sum_{i=1}^n x_i \quad (3-2)$$

Where \bar{x} = arithmetic average of individual measurements

n = number of measurements

x_i = average value of measurement during the i th time period

To analyze the BOD and TSS data given on Fig. 3–6, for example, the usual procedure is to divide the day's record into 24 1-h increments, sum the 24 individual average hourly values, and divide by 24. Although arithmetic averages are still used, they are of little value because the magnitude of the flow at the time of the measurement is not taken into account. If the flowrate remains constant, the use of a simple average is acceptable.

Flow-Weighted Average

Flow-weighted constituent concentrations are obtained by multiplying the flow (typically hourly values over a 24-h period) times the corresponding constituent concentration, summing the results, and dividing by the summation of the flows as given by Eq. 3–3.

$$C_w = \frac{\sum_{i=1}^n q_i C_i}{\sum_{i=1}^n q_i} \quad (3-3)$$

where C_w = flow-weighted average constituent concentration

n = number of observations

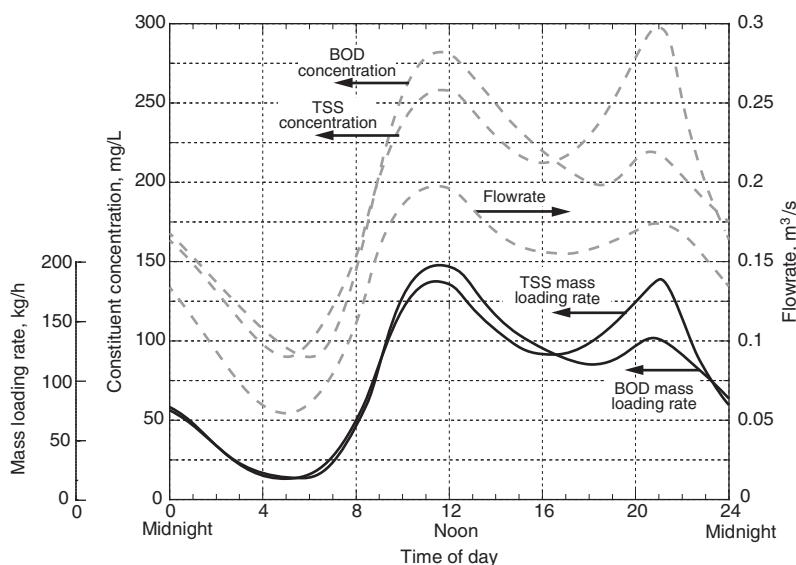
q_i = average flowrate during i th time period

C_i = average concentration of the constituent during i th time period

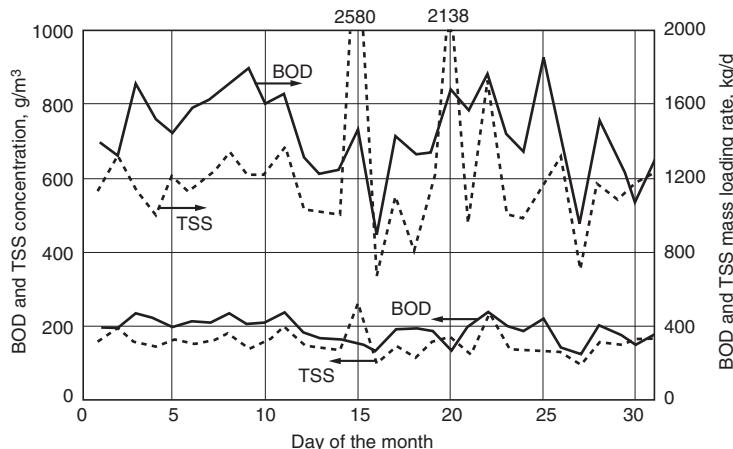
Whenever possible, flow-weighted constituent concentrations should be used because they are a more accurate representation of the actual wastewater strength that must be treated.

Figure 3-14

Illustration of typical hourly variations observed in flow and BOD and TSS concentrations and the corresponding BOD and TSS and mass loading rates. (Typical hourly variations are from Fig. 3-11.)

**Figure 3-15**

Example variations of TSS and BOD concentrations and mass loading rates over a monthly period.



biological treatment operating conditions. The maximum hourly BOD loading rate may vary as much as 3 to 4 times the minimum hourly BOD loading rate in a 24-h period. Over longer periods of time, the mass loading rates can also vary widely (see Fig. 3-15). These types of variations have to be accounted for in the design of the biological treatment system. In extreme cases, flow or load equalization may be required.

3-6 SELECTION OF DESIGN FLOWRATES AND MASS LOADINGS

The rated capacity of wastewater treatment plants is normally based on the average annual daily flowrate at the design year plus an allowance for future growth. As a practical matter, however, wastewater treatment plants have to be designed to meet a number of

conditions that are influenced by flowrates, wastewater characteristics and constituent concentrations, and a combination of both flowrate and concentration (mass loading). Conditions that must be considered include peak and minimum hydraulic flowrates and the maximum, minimum, and sustained process constituent mass loading rates. The importance of wastewater flowrates in process design and operation is considered in this section; mass loadings are discussed in a following section.

Additionally, periods of initial operation and low flows and loads must be taken into consideration in design. Typical flowrate and mass loading factors that are important in the design and operation of wastewater treatment facilities are described in Table 3-22. The overall objective of wastewater treatment is to provide a wastewater treatment system that

Table 3-22
Typical flowrate and mass loading factors used for the design and operation of wastewater treatment facilities^a

Flowrate factor	Purpose for design and operation
Flowrate	
Average daily flow	Development of flowrate ratios and for estimating pumping and chemical costs. Also used to estimate operator staffing requirements
Peak hour	Sizing of pumping facilities and conduits; sizing of physical unit processes: grit chambers, sedimentation tanks, and filters; sizing chlorine contact tanks. Getting the peak hourly flowrate through the treatment plant without flooding channels and weirs is a major design challenge. Also important in developing process control strategies.
Maximum day	Sizing of equalization basins, sedimentation tank and clarifier effluent troughs and weirs, chlorine contact tanks, and sludge pumping system. Also important in developing process control strategies for managing high flows
Maximum month	Record keeping and reporting; selection of maximum number of operating units required during high flow periods; sustained high flow operational strategies; and sizing of chemical storage facilities
Minimum hour	Sizing turndown of pumping facilities and chemical feed systems and determining low range of plant flowmeter(s)
Minimum day	Sizing of influent channels to control solids deposition; sizing effluent recycle requirements for trickling filters
Minimum month	Selection of minimum number of operating units required during low flow periods; scheduling shutdown for maintenance
Mass loading	
Minimum month	Process turndown requirements
Minimum day	Sizing of trickling filter recycle rates
Maximum day	Sizing of selected process units
Maximum month	Sizing of sludge storage facilities; sizing of composting requirements
Maximum 15-d	Sizing anaerobic and aerobic digesters
Sustained loading	Sizing of selected process units and ancillary process equipment

^a Adapted from Tchobanoglous et al. (2003).

is capable of coping with a wide range of probable wastewater conditions while complying with the overall performance requirements.

Design Flowrates

The development and forecasting of flowrates is necessary to determine the design capacity as well as the hydraulic requirements of the treatment system. Flowrates need to be developed both for the initial period of operation and for the future (design) period. Consideration of the flowrates during the early years of operation is often overlooked, and oversizing of equipment and inefficient operation can result. The focus of the following discussion is on the development of various design flowrates.

Rationale for the Selection of Flowrates. The rationale for selecting flowrates is based on hydraulic and process considerations. As stated, the process units and hydraulic conduits must be sized to accommodate the anticipated peak flowrates that will pass through the treatment plant. Provisions have to be made to ensure bypassing of wastewater does not occur either in the collection system or at the treatment plant. Many of the process units are designed based on detention time or overflow rate (flowrate per unit of surface area) to achieve the desired removal rates of BOD and TSS. Because the performance of these units can be affected significantly by varying flowrate conditions and mass loadings, minimum and peak flowrates must be considered in design.

Forecasting Flowrates. In determining the design flowrate, elements to be considered are (1) the existing base flows; (2) estimated future flows for residential, commercial, institutional, and industrial sources; and (3) non-excessive infiltration/inflow. Existing base flows equal actual metered flowrates minus excessive infiltration/inflow (defined as infiltration/inflow that can be controlled by cost-effective improvements to the collection system).

A yardstick by which total dry-weather base flow can be measured where infiltration is not excessive is 420 L/capita·d (110 gal/capita·d). This base flow includes 230 L/capita·d (60 gal/capita·d) for domestic flows, 40 L/capita·d (10 gal/capita·d) for commercial and small industrial flows, and 150 L/capita·d (40 gal/capita·d) for infiltration.

A useful technique in forecasting flowrates is probability analysis, discussed earlier in this chapter. Where flowrate data are available, preferably for at least two years, future flowrates for design can be predicted with a reasonable certainty. An example of a probability analysis of flowrates, as well as BOD and TSS concentrations and mass loadings, is shown on Fig. 3–16. The probability analysis can be used to estimate occurrences of peak flows and loads, and to establish a basis for selecting design flows and loads. For example, a maximum one-day occurrence can be determined based on a 99.7 percent probability; the value will not be equaled or exceeded in the time period analyzed. A probability value, such as the 95th percentile, can also be established for forecasting the design loadings to meet permit requirements.

Minimum Flowrates. As noted in Table 3–22, low flowrates are also of concern in treatment plant design, particularly during the initial years of operation when the plant is operating well below the design capacity, and in designing pumping stations. In cases where very low nighttime flow is expected, provisions for recycling treated effluent may have to be included to sustain the process, (e.g., biological treatment processes such as trickling filters, and to maintain optimal flowrates through ultraviolet disinfection systems). In absence of measured flowrate data, minimum daily flowrates may be assumed to range from 30 to 90 percent of average flowrates for small to large size communities, respectively [see also Fig. 3–3(c)] (WEF, 2010).

Using the s_g values determined in Step 1, the corresponding peaking factors for peak day and peak month can be found on Fig. 3-9(a) for a small facility and Fig. 3-9(b) for a large facility. The peaking factors are summarized in the following table.

Parameter	Small facility			Large facility		
	s_g	Peaking factor		s_g	Peaking factor	
		d	mo		d	mo
Flowrate	1.6	3.70	2.35	1.15	1.48	1.29
BOD	1.6	3.70	2.35	1.27	1.95	1.55
COD	1.7	4.40	2.65	1.30	2.20	1.62
TSS	1.6	3.70	2.35	1.27	1.95	1.55

3. To obtain the maximum values for a specified frequency, multiply the peaking factors determined in step 2 by the mean value from the table given in the problem statement.
 - a. For the peak day flowrate for the small facility the peaking factor is 3.70 and the mean design value is $10,000 \text{ m}^3/\text{d}$:
 $(3.70)(10,000 \text{ m}^3/\text{d}) = 37,000 \text{ m}^3/\text{d}$
 - b. The design values for the two facilities are summarized in the following table.

Parameter	Unit	Design values					
		Small facility			Large facility		
		Mean	Peak d	Peak mo	Mean	Peak d	Peak mo
Flowrate	m^3/d	10,000	37,000	23,500	500,000	740,000	645,000
BOD	mg/L	250	925	587.5	250	487.5	387.5
COD	mg/L	600	2640	1590	600	1320	972
TSS	mg/L	200	740	470	200	390	310

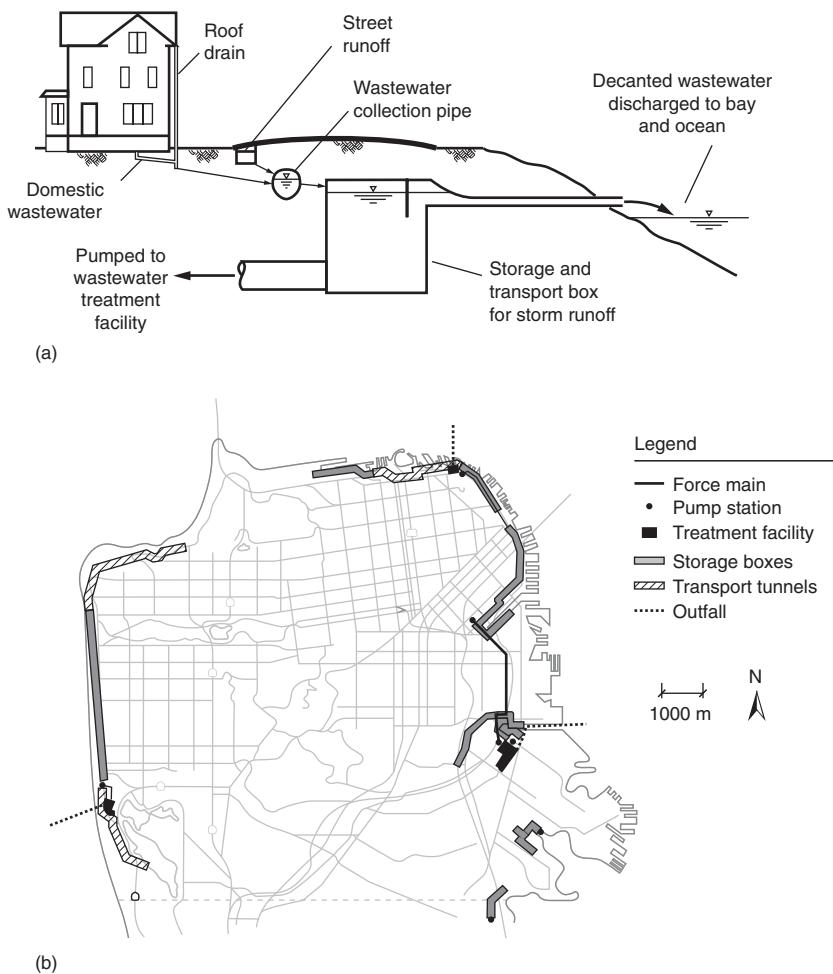
Comment As shown in the summary table presented in Step 3, the smaller facility must be designed to accommodate a larger range in influent wastewater parameters relative to the large facility.

3-7 FLOW AND CONSTITUENT LOAD EQUALIZATION

The variations of influent wastewater flowrate and characteristics at wastewater treatment facilities were discussed in Secs. 3-4 and 3-6. Flow equalization is a method used to overcome the operational problems caused by flowrate variations, to improve the performance of the downstream processes, and to reduce the size and cost of downstream treatment facilities. Similarly, load equalization is a method used to reduce capital and operating cost of downstream treatment facilities. A variation of flow equalization involves the use of large offline storage basins or tunnels that are built specifically to capture and temporally store excess stormwater flow that would otherwise be discharged through CSO outfalls (see Fig. 3-19). The discussion below, however, focuses primarily on flow equalization as it pertains to wastewater treatment plant design.

Figure 3-19

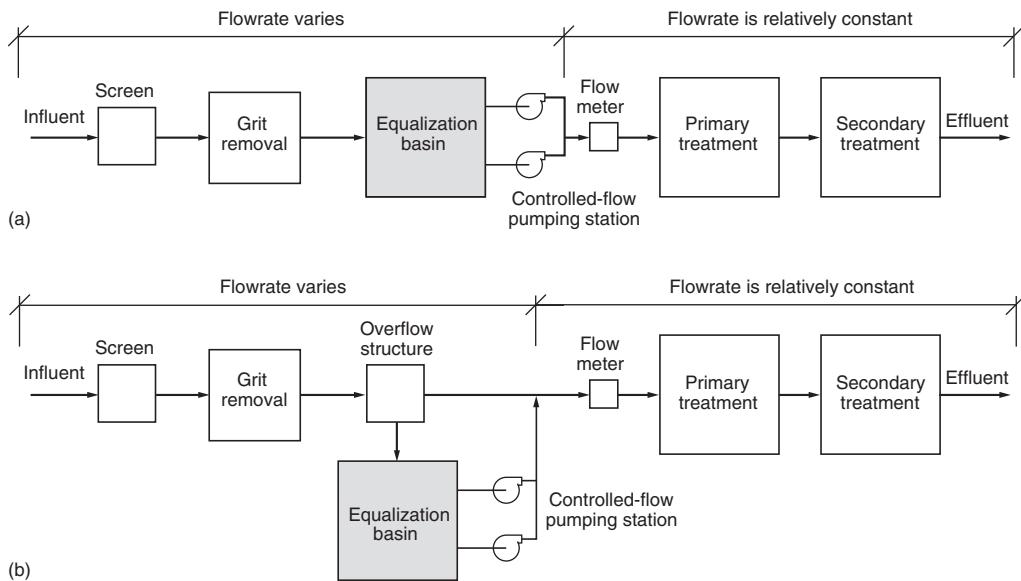
Details of combined collection system in the City of San Francisco, CA. (a) domestic wastewater, water from roof drains, and street runoff are collected in a combined collection system. Excess stormwater is discharged to large stormwater transport/storage boxes and tunnels. Once the stormwater flow has receded and treatment capacity becomes available, the wastewater from the storage boxes is treated before being discharged to San Francisco Bay or the Pacific Ocean, and (b) location of transport/storage boxes and tunnels around the periphery of the city (courtesy of City of San Francisco, CA).



Description/Application of Flow Equalization

Flow equalization simply is the dampening of flowrate variations to achieve a constant or nearly constant flowrate and can be applied in a number of different situations, depending on the characteristics of the collection system. The principal applications are for the equalization of (1) dry-weather flows to reduce peak flows and loads, (2) wet-weather flows in sanitary collection systems experiencing inflow/infiltration, or (3) combined stormwater and sanitary system flows as briefly noted above.

The application of flow equalization in wastewater treatment is illustrated in the two flow diagrams given on Fig. 3-20. In the inline arrangement [see Fig. 3-20(a)], all of the flow passes through the equalization basin. This arrangement can be used to achieve a considerable amount of constituent concentration and flowrate dampening. In the offline arrangement [see Fig. 3-20(b)], only the flow above some predetermined flow limit is diverted into the equalization basin. Although pumping requirements are minimized in this arrangement, the amount of constituent concentration dampening is considerably reduced. Offline equalization is sometimes used to capture the “first flush” from combined collection systems.

**Figure 3-20**

Typical wastewater treatment plant flow diagram incorporating flow equalization (a) inline equalization and (b) offline equalization.

The Benefits of Flow Equalization

The principal benefits that are cited as deriving from application of flow equalization are (1) biological treatment is enhanced, because shock loadings are eliminated or can be minimized, inhibiting substances can be diluted, and pH can be stabilized; (2) the effluent quality and thickening performance of secondary sedimentation tanks following biological treatment is improved through improved consistency in solids loading and the elimination of flow surges; (3) effluent filtration, or other tertiary treatment systems, surface area or volumetric requirements are reduced, performance is improved, and, for filtration systems, more uniform filter-backwash cycles are possible by lower hydraulic loading; and (4) in chemical treatment, dampening of mass loading improves chemical feed control and process reliability. Apart from improving the performance of most treatment operations and processes, flow equalization is an attractive option for upgrading the performance of overloaded treatment plants. Disadvantages of flow equalization include (1) relatively large land areas or sites are needed, (2) additional operation and maintenance is required, (3) potential for odors, and (4) increased capital cost. The use of, benefits, advantages, and disadvantages of flow equalization in municipal wastewater treatment were reviewed in an early U.S. EPA report (Ongерth, 1979).

Design Considerations

The design of flow equalization facilities is concerned with the following questions:

1. Where in the treatment process flowsheet should the equalization facilities be located?
2. What type of equalization flowsheet should be used, inline or offline?
3. What is the required equalization volume?
4. What are the features that should be incorporated into design?
5. How can the deposition of solids and potential odors be controlled?

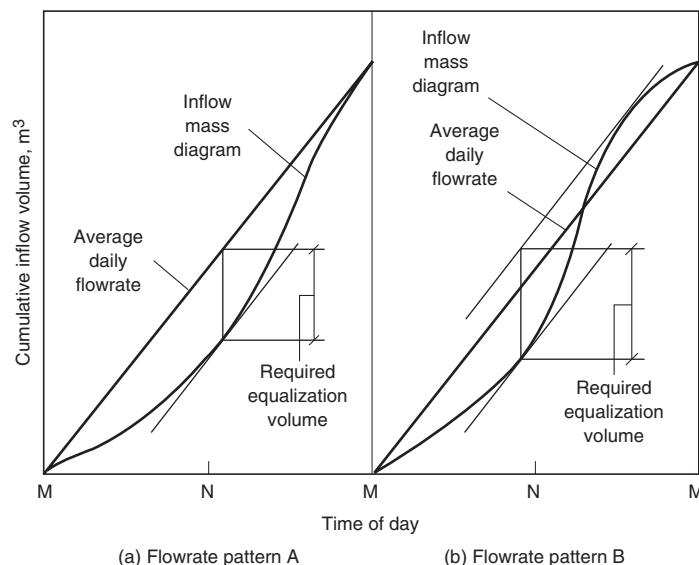
Location of Flow Equalization Facilities. The optimal location for siting equalization facilities must be determined for each system. Because the location will vary depending upon the characteristics of the collection system and the wastewater to be handled, land requirements and availability, and the type of treatment required, detailed studies should be performed for several locations throughout the system. Where equalization facilities are considered for location adjacent to the wastewater treatment plant, it is necessary to evaluate how they could be integrated into the treatment process flowsheet. In some cases, equalization after primary treatment and before biological treatment may be appropriate. Equalization after primary treatment causes fewer problems with solids deposits and scum accumulation. If flow equalization systems are to be located ahead of primary settling and biological systems, the design must provide for sufficient mixing to prevent solids deposition and concentration variations, and aeration to prevent odor problems.

Determination of Volume Requirements for Equalization Basin. The volume required for flowrate equalization is determined by using an inflow cumulative volume diagram in which the cumulative inflow volume is plotted versus the time of day. The average daily flowrate, also plotted on the same diagram, is the straight line drawn from the origin to the end point of the diagram. Diagrams for two typical flowrate patterns are shown on Fig. 3-21.

To determine the required volume, a line parallel to the coordinate axis, defined by the average daily flowrate, is drawn tangent to the mass inflow curve. The required volume is then equal to the vertical distance from the point of tangency to the straight line representing the average flowrate [see Fig. 3-21(a)]. If the inflow mass curve goes above the line representing the average flowrate [see Fig. 3-21(b)], the inflow mass diagram must be bounded with two lines that are parallel to the average flowrate line and tangent to extremities of the inflow mass diagram. The required volume is then equal to the vertical distance between the two lines. The determination of the required volume for equalization is also illustrated in Example 3-10. The procedure is exactly the same as if the average hourly volume were subtracted from the volume flow occurring each hour, and the resulting

Figure 3-21

Schematic mass diagrams for the determination of the required equalization basin storage volume for two typical flowrate patterns.



cumulative volumes were plotted. In this case, the low and high points of the curve would be determined using a horizontal line.

The physical interpretation of the diagrams shown on Fig. 3-21 is as follows. At the low point of tangency (flowrate pattern A) the storage basin is empty. Beyond this point, the basin begins to fill because the slope of the inflow mass diagram is greater than that of the average daily flowrate. The basin continues to fill until it becomes full at midnight. For flowrate pattern B, the basin is filled at the upper point of tangency.

In practice, the volume of the equalization basin will be larger than that theoretically determined to account for the following factors:

1. Continuous operation of aeration and mixing equipment will not allow complete drawdown, although special structures can be built.
2. Volume must be provided to accommodate the concentrated plant recycle streams that are expected, if such flows are returned to the equalization basin (a practice that is not recommended because of the potential to create odors).
3. Some contingency should be provided for unforeseen changes in diurnal flow.

Although no fixed value can be given, the additional volume will vary from 10 to 20 percent of the theoretical value, depending on the specific conditions.

EXAMPLE 3-10 Determination of Flowrate Equalization Volume Requirements and Effects on BOD Mass Loading For the flowrate and BOD concentration data given in following table, determine (1) the inline storage volume required to equalize the flowrate graphically (Note: the analytical spreadsheet solution is left to the reader) and (2) the effect of flow equalization on the BOD mass loading rate.

Time period	Given data		Derived data	
	Average flowrate during time period, m ³ /s	Average BOD concentration during time period, mg/L	Cumulative volume of flow at end of time period, m ³	BOD mass loading during time period, kg/h
M-1	0.275	150	990	149
1-2	0.220	115	1782	91
2-3	0.165	75	2376	45
3-4	0.130	50	2844	23
4-5	0.105	45	3222	17
5-6	0.100	60	3582	22
6-7	0.120	90	4014	39
7-8	0.205	130	4752	96
8-9	0.355	175	6030	223
9-10	0.410	200	7506	295
10-11	0.425	215	9036	329
11-N	0.430	220	10,584	341
N-1	0.425	220	12,114	337

(continued)

(Continued)

Time period	Given data		Derived data	
	Average flowrate during time period, m ³ /s	Average BOD concentration during time period, mg/L	Cumulative volume of flow at end of time period, m ³	BOD mass loading during time period, kg/h
1–2	0.405	210	13,572	306
2–3	0.385	200	14,958	277
3–4	0.350	190	16,218	239
4–5	0.325	180	17,388	211
5–6	0.325	170	18,558	199
6–7	0.330	175	19,746	208
7–8	0.365	210	21,060	276
8–9	0.400	280	22,500	403
9–10	0.400	305	23,940	439
10–11	0.380	245	25,308	335
11–M	0.345	180	26,550	224
Average	0.307		213	

Note: m³/s × 35.3147 = ft³/sm³ × 35.3147 = ft³mg/L = g/m³.**Solution**

- Determine the volume of the inline basin required for the flow equalization.
 - The first step is to develop a cumulative volume curve of the wastewater flow-rate expressed in cubic meters. The cumulative volume curve is obtained by converting the average flowrate (q_i) during each hourly period to cubic meters, using the following expression, and then cumulatively by summing the hourly values to obtain the cumulative flow volume.

$$\text{Volume, m}^3 = (q_i, \text{m}^3/\text{s})(3600 \text{ s/h})(1.0 \text{ h})$$

For example, for the first three time periods shown in the data table, the corresponding hourly volumes are as follows:

For the time period M–1:

$$\begin{aligned} V_{M-1} &= (0.275 \text{ m}^3/\text{s})(3600 \text{ s/h})(1.0 \text{ h}) \\ &= 990 \text{ m}^3 \end{aligned}$$

For the time period 1–2:

$$\begin{aligned} V_{1-2} &= (0.220 \text{ m}^3/\text{s})(3600 \text{ s/h})(1.0 \text{ h}) \\ &= 792 \text{ m}^3 \end{aligned}$$

The cumulative flow, expressed in m³, at the end of each time period is determined as follows:

At the end of the first time period M–1:

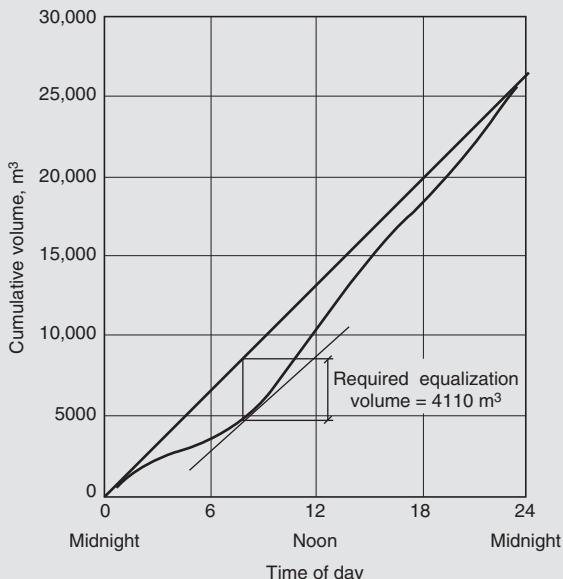
$$V_1 = 990 \text{ m}^3$$

At the end of the second time period 1–2:

$$V_2 = 990 + 792 = 1782 \text{ m}^3$$

The cumulative flows for all the hourly time periods are computed in a similar manner (see derived data in data table)

- The second step is to prepare a plot of the cumulative flow volume, as shown in the following diagram. As will be noted, the slope of the line drawn from the origin to the end point of the inflow mass diagram represents the average flow-rate for the day, which in this case is equal to $0.307 \text{ m}^3/\text{s}$.



- The third step is to determine the required inline storage volume. The required storage volume is determined by drawing a line parallel to the average flowrate tangent to the low point of the inflow mass diagram. The required volume is represented by the vertical distance from the point of tangency to the straight line representing the average flowrate. Thus, the required volume is equal to

$$\text{Volume of equalization basin, } V \sim 4100 \text{ m}^3 (144,790 \text{ ft}^3)$$

- Determine the effect of the equalization basin on the BOD mass loading rate. Although there are alternative computation methods, perhaps the simplest way is to perform the necessary computations starting with the time period when the equalization basin is empty. Because the equalization basin is empty at about 8:00 a.m., the necessary computations will be performed starting with the 8–9 time period.
 - The first step is to compute the liquid volume in the equalization basin at the end of each time period. The volume required is obtained by subtracting the equalized hourly flowrate expressed as a volume from the inflow flowrate also expressed as a volume. The volume corresponding to the equalized flowrate for a period of 1 h is $0.307 \text{ m}^3/\text{s} \times 3600 \text{ s/h} = 1106 \text{ m}^3$. Using this value, the volume in storage is computed using the following expression:

$$V_{sc} = V_{sp} + V_{ic} - V_{oc}$$

where V_{sc} = volume in the equalization basin at the end of current time period

V_{sp} = volume in the equalization basin at the end of previous time period

V_{ic} = volume of inflow during the current time period

V_{oc} = volume of outflow during the current time period

4

Wastewater Treatment Process Selection, Design, and Implementation

4-1	PLANNING FOR NEW AND UPGRADING EXISTING WASTEWATER TREATMENT PLANTS 265
	<i>Need to Upgrade Existing Wastewater Treatment Plants</i> 265
	<i>Planning for New Wastewater Treatment Plants</i> 266
	<i>Treatment Process Design Considerations</i> 266
	<i>Owner Needs</i> 270
	<i>Environmental Considerations</i> 270
	<i>Compatibility with Existing Facilities</i> 271
	<i>Energy and Resource Requirements</i> 271
	<i>Cost Considerations</i> 272
	<i>Other Design Considerations</i> 273
4-2	CONSIDERATIONS IN PROCESS SELECTION 274
	<i>Important Factors in Process Selection</i> 274
	<i>Process Selection Based on Reaction Kinetics</i> 276
	<i>Process Selection Based on Mass Transfer</i> 277
	<i>Process Design Based on Loading Criteria</i> 277
	<i>Bench-Scale Tests and Test-Bed Pilot-Scale Studies</i> 277
	<i>Wastewater Discharge Permit Requirements</i> 278
4-3	TREATMENT PROCESS RELIABILITY AND SELECTION OF DESIGN VALUES 279
	<i>Variability in Wastewater Treatment</i> 280
	<i>Selection of Process Design Parameters to Meet Discharge Permit Limits</i> 286
	<i>Performance of Combined Processes</i> 289
4-4	ELEMENTS OF PROCESS DESIGN 291
	<i>Design Period</i> 291
	<i>Treatment Process Flow Diagrams</i> 291
	<i>Process Design Criteria</i> 291
	<i>Preliminary Sizing</i> 292
	<i>Solids Balance</i> 293
	<i>Plant Layout</i> 294
	<i>Plant Hydraulics</i> 295
	<i>Energy Management</i> 297
4-5	IMPLEMENTATION OF WASTEWATER MANAGEMENT PROGRAMS 297
	<i>Facilities Planning</i> 297
	<i>Design</i> 298
	<i>Value Engineering</i> 298
	<i>Construction</i> 298
	<i>Facilities Startup and Operation</i> 299

4-6 FINANCING	299
<i>Long-Term Municipal Debt Financing</i>	300
<i>Non-Debt Financing</i>	300
<i>Leasing</i>	300
<i>Privatization</i>	300
PROBLEMS AND DISCUSSION TOPICS 301	
REFERENCES 303	

WORKING TERMINOLOGY

Term	Definition
Bench-scale tests	Tests conducted in the laboratory with small quantities of samples to answer specific treatment questions.
Biosolids	Sludge from wastewater treatment processes that has been stabilized to meet the criteria in the U.S. EPA's 40 CFR 503 regulations and, therefore, can be used beneficially.
CCA	Critical component analysis. A method used to determine which mechanical components in the wastewater treatment plant will have the most immediate impact on effluent quality should failure occur.
Class A Biosolids	Biosolids that contain less than 1000 most probable number (MPN)/g of fecal coliforms and less than 3 MPN/4g of <i>Salmonella</i> bacteria and meet one of six stabilization alternatives given in 40 CFR 503. The material also must meet the pollutant limits and vector attraction reduction requirements set forth in 40 CFR 503.
ENRCCI	Engineering News-Record Construction Cost Index used to adjust construction cost information.
Flow diagram	The graphical representation of a particular combination of unit processes used to achieve specific treatment objectives.
Hydraulic profile	A graphical representation of the elevation of the free surface and hydraulic grade line of wastewater as it flows through the various treatment units.
NPDES	National Pollution Elimination Discharge System (NPDES), established based on uniform technological minimums with which each point source discharger has to comply.
Peaking factor	A factor that is applied customarily to average long term (typically flowrate and mass loading rate values) to determine maximum values.
Plant layout	The spatial arrangement of the physical facilities of the treatment plant identified in the flow diagram.
Pilot plant studies	Studies conducted at test beds at a scale larger than bench-scale, to establish the suitability of a process in the treatment of a specific wastewater under specific environmental conditions and to obtain data that can be used for full-scale design.
POTW	Publicly Owned Treatment Works.
Priority pollutants	Organic and inorganic compounds defined under Section 307 of Clean Water Act and listed in the Code of Federal Regulations at 40 CFR 401.15. Priority pollutants are selected on the basis of their known or suspected carcinogenicity, mutagenicity, teratogenicity, or high acute toxicity.
Privatization	Private sector ownership and operation of facilities and services used by government entities in performing their public function.
Process design criteria	The criteria used as the bases for sizing individual unit processes and their support systems.
Reliability, treatment process	Probability of adequate performance for a specified period of time under specified conditions or the percent of the time that effluent concentrations meet specified permit requirements.

Solids balance	The identification of the quantities of solids entering and leaving each unit process.
Value engineering	An intensive review of a project to determine best value, or value improvement, which may or may not result in cost reduction. Typically conducted at the 20 to 30 percent design stage of the project.
Variability, inherent	Based on the laws of chance, all physical, chemical, and biological treatment processes exhibit some measure of variability with respect to the performance that can be achieved. Variability is inherent in biological treatment processes.

Since the early 1900s, when the field of environmental engineering was in its infancy in the United States, there has been a steady evolution and development in the methods used for wastewater treatment. Descriptions of the many methods and variations that have been tried to date would fill several large volumes. The approach followed in this text is to identify and discuss basic principles and their application to wastewater treatment. The purpose of this chapter is to provide perspective on how the principles presented in Chaps. 1 through 3 and 5 through 18, along with other subjects not discussed in detail in this textbook, fit into the overall design, construction, operation and maintenance, and implementation of new and existing wastewater management projects. The following topics are covered: (1) planning considerations for wastewater treatment plants, (2) process selection, (3) treatment process reliability and selection of design values, (4) elements of process design, (5) implementation of wastewater management programs, and (6) financing.

4-1 PLANNING FOR NEW AND UPGRADING EXISTING WASTEWATER TREATMENT PLANTS

Most of the wastewater plants implemented using federal grant funding were designed using empirical design guidelines and were sized to accommodate long-term population growth (a 20 to 30 y timeline was the norm). Plants were located typically in remote areas close to water bodies that provided natural buffer zones between the treatment facilities and the communities they served. As sanitary sewer collection systems use gravity flow to collect wastewater, treatment plants were often located in lowlands adjacent to the receiving waters. In many cases, municipal growth has encroached on the natural buffer zones so that treatment facilities that once were considered remote are now in close proximity to the community. Going forward, new wastewater treatment plants will be required (1) to support population growth; (2) to deal with changing demographics; (3) to deal with changing wastewater characteristics, especially increasing wastewater concentrations as discussed in Chap. 3; (4) to meet new and more stringent effluent discharge standards; (5) to meet water reuse needs, including potable reuse; (6) to meet new stormwater management objectives; and (7) to replace existing aging infrastructure.

Need to Upgrade Existing Wastewater Treatment Plants

Over the next decade, many existing treatment plants, built during the 1970s and 1980s, will need to be upgraded to maintain treatment efficiency, to provide higher levels of treatment when discharging to environmentally sensitive receiving waters, to meet new water reuse opportunities, and to meet new stormwater management objectives. As more wastewater reuse is being implemented, additional treatment processes, including advanced oxidation, may have to be implemented. Treatment plants must also be upgraded aesthetically to minimize odors, noise, and visual impacts to the neighboring community.

4-4 ELEMENTS OF PROCESS DESIGN

Treatment process design is undertaken simultaneously with the selection of the treatment process and appropriate design coefficients based on the wastewater flowrate and characteristics and treatment objectives and goals. The principal elements of process design are (1) establishing the design period for facilities, (2) development of the process flow diagram, (3) establishing process design criteria, (4) preliminary sizing of treatment units, (5) preparation of solids balances, (6) site layout considerations, (7) evaluation of plant hydraulics (hydraulic profile), and (8) energy management. Each of these elements are introduced and discussed in this section.

Design Period

The design period corresponds to the target date when the design capacity of the facilities is reached. Design periods may vary for individual components, depending upon the ease or difficulty of expansion. Typical design periods for various types of facilities are given in Table 4-9. Longer periods are preferred for structures and hydraulic conduit systems, which cannot be expanded easily. The selection of the design period depends upon growth characteristics, environmental considerations, and the availability and source of construction funds.

Treatment Process Flow Diagrams

Treatment process flow diagrams are graphical representations of particular combinations of unit operations and processes. Depending on the constituents that must be removed, an almost limitless number of different flow diagrams can be developed by combining various unit processes. Apart from the analysis of the suitability of the types of individual treatment units, the exact configuration of process units selected will also depend on factors such as (1) the designer's past experience, (2) design and regulatory agency policies on the application of specific treatment methods, (3) the availability of suppliers of equipment for specific treatment methods, (4) the maximum use that can be made of existing facilities, (5) initial construction costs, and (6) future operation and maintenance costs. A typical process flow diagram for the treatment of wastewater to meet secondary treatment standards, as defined by the U.S. EPA is shown on Fig. 4-7(a).

Process Design Criteria

After one or more preliminary process flow diagrams have been developed, the next step is to determine the process design criteria for the selected treatment processes so that the size of the physical facilities can be determined. The hydraulic detention time would be an

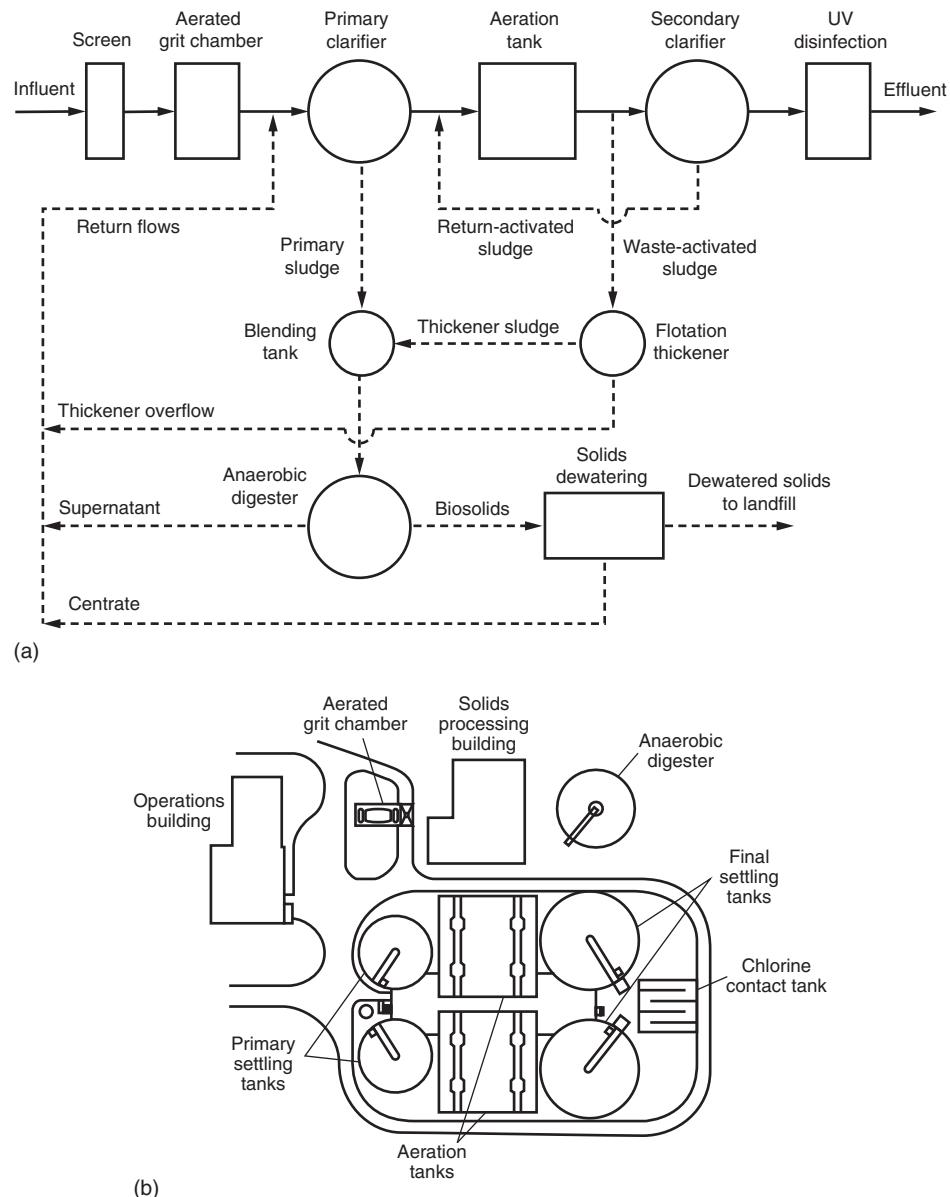
Table 4-9

**Typical design periods
for wastewater
treatment facilities**

Facility	Planning period range, y
Collection Systems	20–40
Pump stations	
Structures	20–40
Pumping equipment	10–20
Treatment plants	
Process structures	20–40
Process equipment	10–20
Hydraulic conduits	20–40

Figure 4–7

Wastewater treatment plant designed to meet US EPA secondary wastewater treatment standards (see Table 1–2)
 (a) schematic flow diagram and
 (b) schematic layout.



example of the process design criteria for the grit chamber. Similar procedures are followed for each unit process. All the key design criteria should be listed in a summary table. Because most treatment plants are designed to be effective for some time in the future (up to 40 y), design criteria are given generally for the time when the facilities will first be put into operation, and for the end of the design period. The latter will be influenced by projections of the population to be served and the economic studies of cost effectiveness for various design periods.

Preliminary Sizing

After the design criteria have been established, the next step is to determine the number and size of the physical facilities needed. For example, if the hydraulic detention time in the

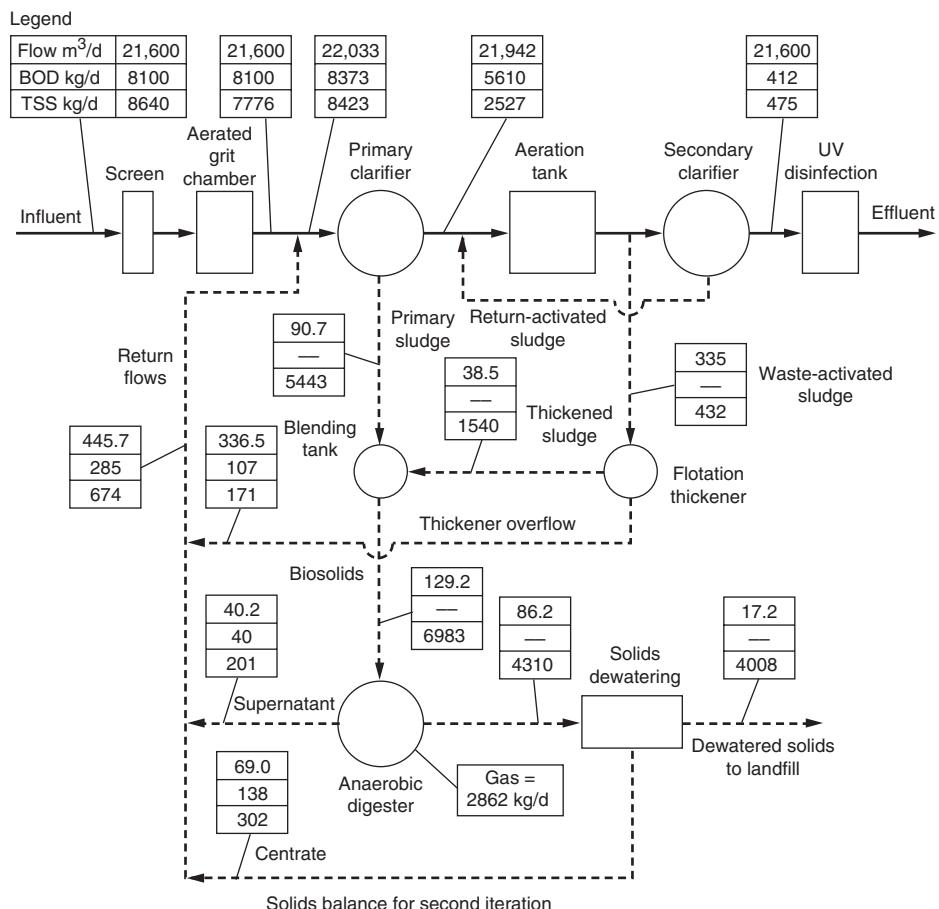
aerated grit chamber shown on Fig. 4-7(a) is to be 3.5 min at a peak flowrate, the corresponding grit chamber volume required would be calculated. In considering sizing, physical site constraints need to be considered: for example, will the site accommodate the use of round sedimentation tanks or will rectangular tanks have to be used? Operational considerations such as flow splitting and load balancing will have to be evaluated, particularly in process trains that combine different numbers of unit processes—for example, two primary clarifiers and three aeration tanks. Maintenance factors have to be considered in selecting the number of units so that provisions are included for taking a unit out of service for maintenance and repair. In small plants where a single unit is being considered, maintenance of that unit may be a particular problem unless special provisions, such as temporary storage, are included.

Solids Balance

After the design criteria are established and the preliminary sizing is completed, solids balances should be prepared for each process flow diagram. They should be prepared for the average load with appropriate peaking factors applied for maximum loads. Such information must be available to size (1) sludge thickening and storage facilities, (2) sludge digesters, (3) sludge dewatering facilities, (4) thermal reduction systems, (4) composting facilities, and (5) sludge piping and pumping equipment and other appurtenant facilities. The solids balance for the flow diagram shown on Fig. 4-7(a) is presented on Fig. 4-8. The details involved in the preparation of the solids balance are illustrated in Chap. 14.

Figure 4-8

Results of solid mass balance for the treatment process flow diagram shown on Fig. 4-7. Computational details are presented in Sec. 14-7 in Chap. 14.



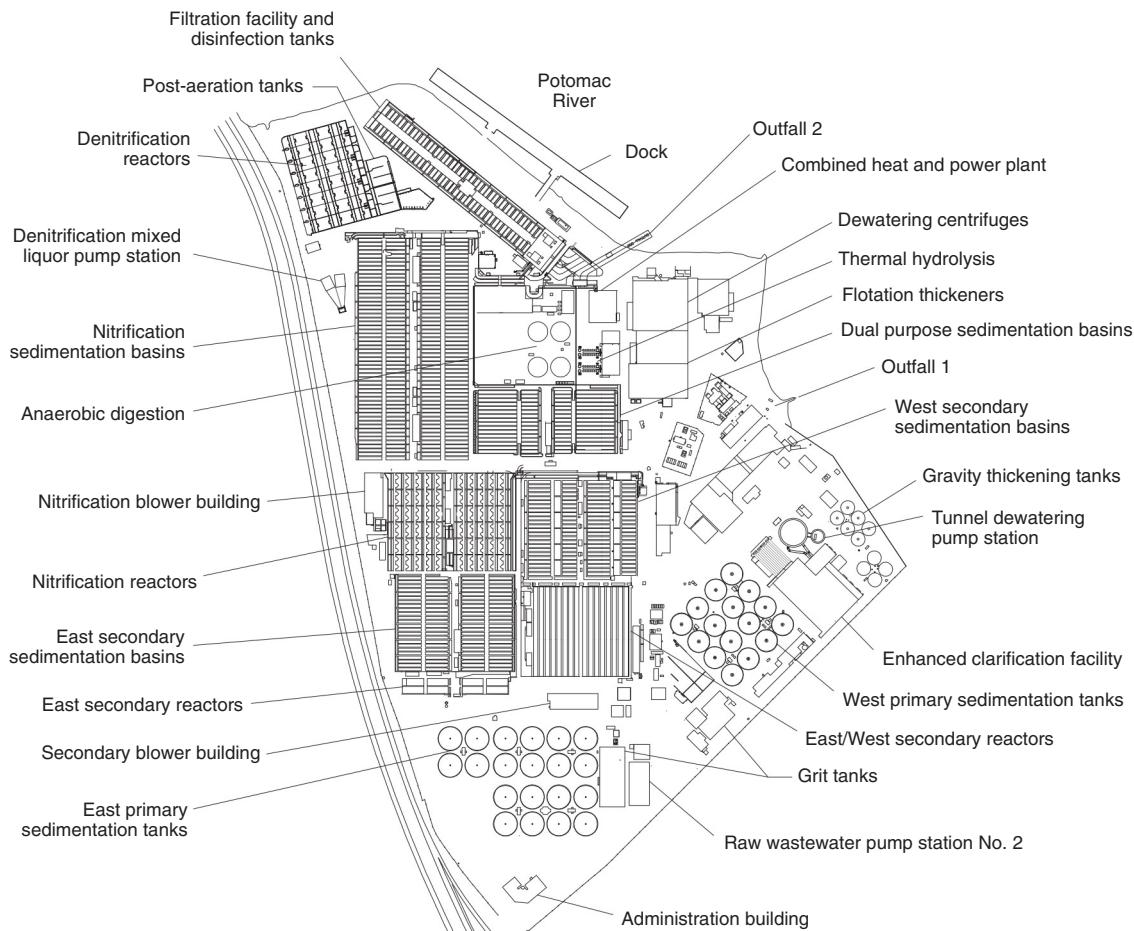


Figure 4-9

Layout of the Blue Plains Advanced Wastewater Treatment Plant with a capacity $16.2 \text{ m}^3/\text{s}$ (370 Mgal/d) serving the Washington, DC area and environs. (Coordinates: 38.8178 N, 77.0220 W, view at altitude 4 km, also shown on the cover of this book.)

Plant Layout

Plant layout refers to the spatial arrangement of the physical facilities required to achieve a given treatment objective. The overall plant layout includes the location of the control and administrative buildings and any other necessary structures. Several different layouts, using computer generated overlays, are normally evaluated before a final selection is made. Among the factors that must be considered when laying out a treatment plant are the following: (1) geometry of the available treatment plant sites, (2) topography, (3) soil and foundation conditions, (4) location of the influent sewer, (5) location of the point of discharge, (6) plant hydraulics, preferably with straight flow paths between units to minimize headloss and provide symmetry for flow splits, (7) types of processes involved, (8) process performance and efficiency, (9) transportation access, (10) accessibility to operating personnel, (11) reliability and economy of operation, (12) aesthetics, (13) environmental control, and (14) provisions for future plant expansion including additional area. The physical layouts for a small and large wastewater treatment plant are shown on Figs. 4-7(b) and 4-9,

respectively. The layout shown on Fig. 4-7(b) corresponds to the flow diagram shown on Fig. 4-7(a). The plant layout shown on Fig. 4-9 is of the large Blue Plains Wastewater Treatment Plant, Washington, DC, which includes a number of advanced treatment processes.

Plant Hydraulics

After the process flow diagram has been selected and the size of the corresponding physical facilities is determined, hydraulic computations and profiles are prepared for both average and peak flowrates. Hydraulic computations are made to size the interconnecting conduits and channels and to compute the headlosses through the plant. Typical ranges of headlosses through treatment units are given in Table 4-10. In designing the plant hydraulic system, consideration needs to be given to (1) equalizing the flow splitting between the treatment units, (2) making provisions for bypassing secondary treatment units at extreme peak flows to prevent loss of biomass, (3) provision for removing treatment facilities during periods of sustained low flow, and (4) minimizing the number of changes in direction of wastewater flow in conduits and channels.

Hydraulic profiles are prepared for three reasons: (1) to ensure that the hydraulic gradient is adequate for the wastewater to flow through the treatment facilities by gravity, (2) to establish the head requirement for the pumps where pumping will be needed, and (3) to ensure that the plant facilities will not be flooded or backed up during periods of peak flow. The hydraulic profile for the flow diagram given on Fig. 4-7 is shown on Fig. 4-10. In preparing a hydraulic profile, distorted vertical and horizontal scales are commonly used to depict the physical facilities.

Hydraulic profile computations involve the determination of the headloss as the wastewater flows through each of the physical facilities in the process flow diagram. Specific computational procedures may vary depending on local conditions. For example, if a downstream discharge condition is the control point, some designers prepare the hydraulic profile by working backward from the control point. Other designers prefer to

Table 4-10
Typical headlosses across various treatment units^a

Treatment unit	Headloss range	
	ft	m
Bar Screen	0.5–1.0	0.2–0.3
Grit chambers		
Aerated	1.5–4.0	0.1–1.2
Velocity controlled	1.5–3.0	0.5–0.9
Primary sedimentation	1.5–3.0	0.5–0.9
Aeration tank	0.7–2.0	0.2–0.6
Trickling filter		
Low-rate	10.0–20.0	3.0–6.1
High-rate, rock media	6.0–16.0	1.8–4.9
High-rate, plastic media	16.0–40.0	4.9–12.2
Secondary sedimentation	1.5–3.0	0.5–0.9
Filtration	10.0–16.0	3.0–4.9
Carbon adsorption	10.0–20.0	3.0–6.1
Chlorine-contact tank	0.7–6.0	0.2–1.8

^aThe reported values do not reflect designs optimized for minimum energy usage.

5–1 SCREENING

The first unit process generally encountered in wastewater treatment plants is screening. A screen is a device with openings, generally of uniform size, that is used to retain large solids found in the influent wastewater to the treatment plant or in combined wastewater collection systems subject to overflows, especially from stormwater. The principal role of screening is to remove coarse materials from the flow stream that could (1) damage or clog subsequent process equipment, (2) reduce overall treatment process reliability and effectiveness, or (3) contaminate waterways. Fine screens are sometimes used in place of or following coarse screens where greater removals of solids are required to (1) protect equipment which may be more sensitive to solids such as membrane bioreactors or (2) eliminate materials that may inhibit the beneficial reuse of biosolids.

All aspects of screenings removal, transport, and disposal must be considered in the application of screening devices, including (1) the degree of screenings removal required because of potential effects on downstream treatment processes and equipment, (2) health and safety of the operators as screenings contain pathogenic organisms and attract insects, (3) odor potential, and (4) requirements for handling, transport, prior to disposal (i.e., removal of organics by washing and reduced water content by pressing), and (5) disposal options. Thus, an integrated approach is required to achieve effective screenings management.

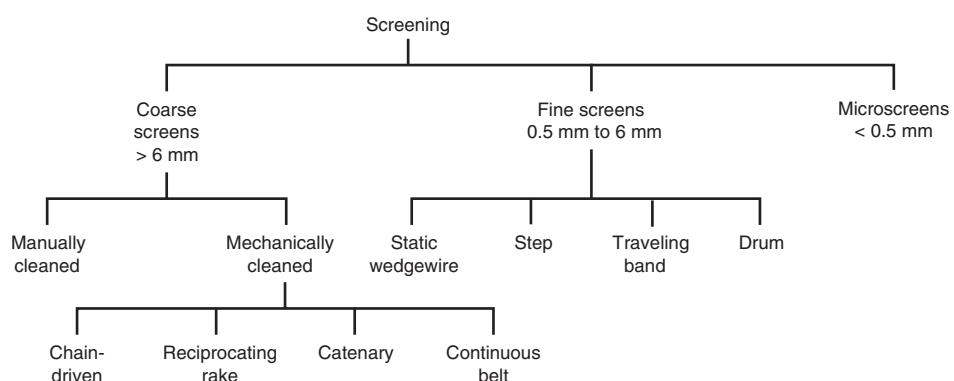
Classification of Screens

Types of screens commonly used in wastewater treatment are identified on Fig. 5–2. Two general types of screens, coarse and fine screens, are used in preliminary treatment. Fine screens may also be used as an alternative primary treatment process or for removal of additional organic solids from sludge streams prior to sludge processing. Microscreens are used principally for removing residual solids from treated effluents.

Screening elements may consist of parallel bars, rods or wires, grating, wire mesh, or perforated plate, and the openings may be of any shape, but generally are circular or rectangular slots. A screen composed of parallel bars or rods is often called a “bar rack” or a coarse screen and is used for the removal of coarse solids. Fine screens are devices consisting of perforated plates, wedge wire elements, and wire cloth that have smaller openings. The materials removed by these devices are known as screenings.

Figure 5–2

General classification for the types of screens used in wastewater treatment.



Screenings Characteristics and Quantities

Screenings are the material retained on bar racks and screens. The smaller the screen opening, the greater will be the quantity of collected screenings. While no precise definition of screenable material exists, and no recognized method of measuring quantities of screenings is available, screenings exhibit some common properties.

Screenings Retained on Coarse Screens. Coarse screenings greater than 6 mm (0.25 in.) consist of debris such as rocks, branches, pieces of lumber, leaves, paper, tree roots, plastics, bottles, cans, and rags. Organic matter associated with screenings are also removed with organic content increasing as the spacing decreases. The accumulation of oil and grease can be a serious problem, especially in cold climates. The quantity and characteristics of screenings collected for disposal vary, depending on the type of bar screen, the size of the bar screen opening, the type of sewer system, and the geographic location. Typical data on the characteristics and quantities of coarse screenings to be expected at wastewater treatment plants served by conventional gravity sewers are reported in Table 5–2.

The characteristics of the collection system will also impact the volume of screenings received. Where the collection system has many lift stations with their own screening equipment, the volume of screenings received at the treatment plant will be less. Combined sewer systems typically produce more screenings than separate sanitary sewers and may produce volumes of screenings at up to 20 times the average rate during the “first flush” of peak wet weather flow event (WEF, 2009).

Screenings Retained on Fine Screens. Fine screenings consist of materials retained on screens with openings of 0.5 mm (0.02 in.) to 6 mm (0.25 in.). The materials retained on fine screens include small rags, paper, plastic materials of various types, razor blades, grit, undecomposed food waste, feces, etc. Typical data on characteristics and quantities of screenings removed from various types of screens are reported in Table 5–3. Compared to coarse screenings, the specific weight of the fine screenings is lower and the moisture content is higher than for coarse screens. Fine screenings contain putrescible matter (including fecal material), substantial amounts of grease and scum and may be or become extremely odorous. Consequently, manual handling of collected fine screenings should be minimized and screening should be conveyed using enclosed conveyors and include grit washing and compacting equipment.

Table 5–2

Typical information on the characteristics and quantities of screenings removed from wastewater with coarse screens

Size of opening between bars, mm	Moisture content, %	Specific weight, kg/m ³	Volume of screenings			
			ft ³ /Mgal Range	ft ³ /Mgal Typical	L/1000 m ³ Range	L/1000 m ³ Typical
6.0 ^a	60–90	700–1100	7–13.5	9.5	51–100	67
12.5	60–90	700–1100	5–10	7.0	37–74	50
25.0	50–80	600–1000	2–5	3.0	15–37	22
37.5	50–80	600–1000	1–2	1.5	7–15	11
50.0	50–80	600–1000	0.5–1.5	0.8	4–11	6

^aInformation on fine screen added for the purpose of comparison.

Note: mm × 0.3937 = in.

kg/m³ × 8.3492 = lb/1000 gal.

Table 5–3

Typical information on the characteristics and quantities of screenings removed from wastewater with various types of screens

Operation	Size of opening, mm	Moisture content, %	Specific weight, kg/m³	Volume of screenings			
				ft³/Mgal	Range	Typical	L/1000 m³
Fine bar screens	12.5	80–90	900–1100	6–15	10	44–110	75
Static wedge wire	9.0	80–90	900–1100	5–12	8	37–85	60
Rotary drum ^a	6.0	80–90	900–1100	4–8	6	30–60	45

^aFollowing coarse screening.

Note: mm × 0.3937 = in.

kg/m³ × 8.3492 = lb/1000 gal.

Coarse Screens (Bar Racks)

In wastewater treatment, coarse screens are used to protect pumps, valves, pipelines, and other appurtenances from damage or clogging by rags and large objects. Typically coarse screens precede fine screens to protect them from damage from large debris. Industrial waste treatment plants may not need coarse screens, depending on the character of the wastes. According to the method used to clean them, coarse screens are designated as either manually cleaned or mechanically cleaned. Screenings, handling, processing, and disposal are discussed following the discussion of the types of screens.

Manually Cleaned Coarse Screens. Manually cleaned coarse screens are used frequently ahead of pumps in small wastewater pumping stations and sometimes at the headworks of small- to medium-sized wastewater treatment plants. Often they are used for standby screening in bypass channels for service during high-flow periods, when mechanically cleaned screens are being repaired, or in the event of a power failure. Normally, mechanically cleaned screens are provided in lieu of manually cleaned screens to minimize labor required to clean the screens and to reduce flooding resulting from clogging of infrequently cleaned screens.

Where used, the length of the manually cleaned bar rack should not exceed the distance that can be conveniently raked, approximately 3 m (10 ft). The screen bars are welded to spacing bars located at the rear face, out of the way of the tines of the rake. A perforated drainage plate should be provided at the top of the rack where the rakes may be stored temporarily for drainage.

The screen channel should be designed to prevent the accumulation of grit and other heavy materials in the channel ahead of the screen and following it. The channel floor should be level or should slope downward through the screen without pockets to trap solids. Fillets may be desirable at the base of the sidewalls. The channel preferably should have a straight approach, perpendicular to the bar screen, to promote uniform distribution of screenable solids throughout the flow and on the screen. Typical design information for manually cleaned bar screens is provided in Table 5–4.

Mechanically Cleaned Bar Screens. The design of mechanically cleaned bar screens has evolved over the years to reduce the operating and maintenance problems and

Table 5-4**Typical design information for manually and mechanically cleaned bar racks**

Parameter	Unit	U.S. customary units		SI units		Cleaning method	
		Cleaning method		Unit	Manual	Mechanical	
		Manual	Mechanical				
Bar size							
Width	in.	0.2–0.6	0.2–0.6	mm	5–15	5–15	
Depth	in.	1.0–1.5	1.0–1.5	mm	25–38	25–38	
Clear spacing between bars	in.	1.0–2.0	0.6–3.0	mm	25–50	15–75	
Slope from vertical	deg	30–45	0–30	deg	30–45	0–30	
Approach velocity							
Maximum	ft/s	1.0–2.0	2.0–3.25	m/s	0.3–0.6	0.6–1.0	
Minimum	ft/s		1.0–1.6	m/s		0.3–0.5	
Allowable headloss	in.	6	6–24	mm	150	150–600	

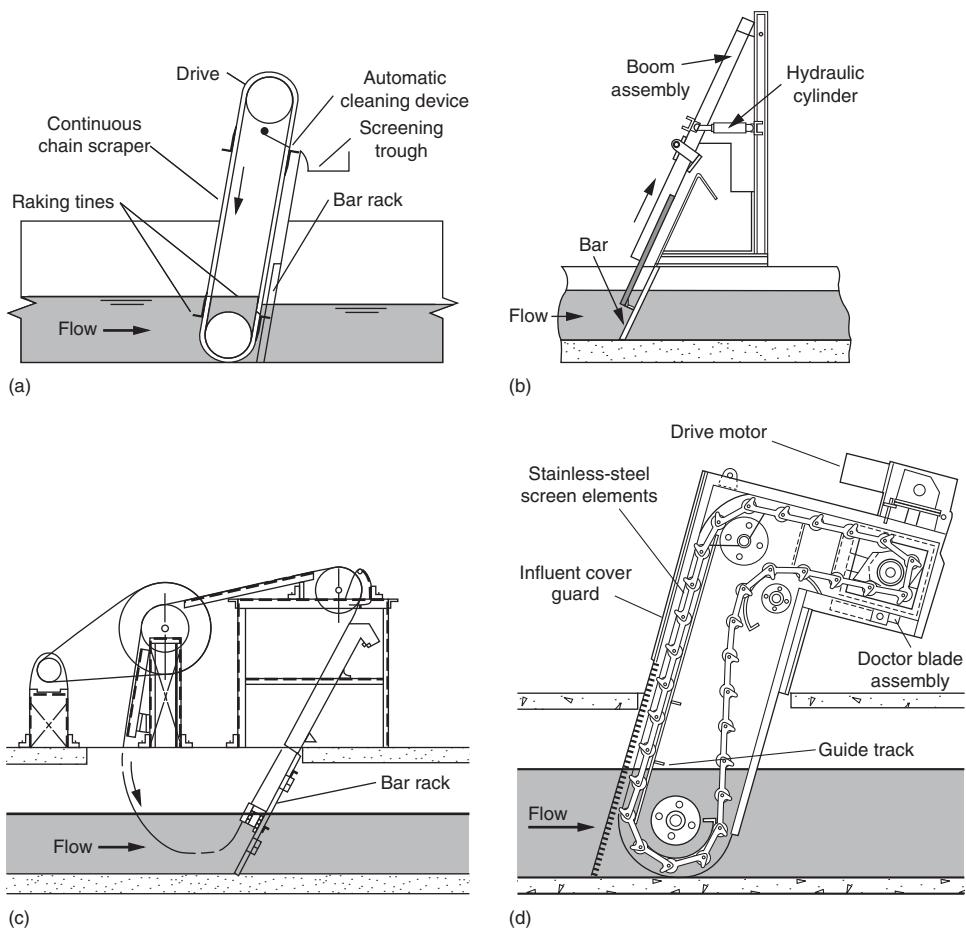
to improve the screenings removal capabilities. Many of the newer designs include extensive use of corrosion-resistant materials including stainless steel and plastics. Mechanically cleaned bar screens are divided into four principal types: (1) chain-driven, (2) reciprocating rake, (3) catenary, and (4) continuous belt. Cable-driven bar screens were used extensively in the past but have been replaced in most wastewater applications by the other types of screens. However, cable-driven raking mechanisms are still used in specific applications such as screens ahead of tunnel dewatering pump stations where deep tunnels are used for storage or conveyance of wet weather flow from combined sewer systems. Typical design information for mechanically cleaned screens is also included in Table 5–4. Examples of the different types of mechanically cleaned bar screens are shown on Fig. 5–3 and the advantages and disadvantages of each type are presented in Table 5–5.

Chain-Driven Screens. Chain-driven mechanically cleaned bar screens can be divided into categories based on whether the screen is raked to clean from the front (upstream) side or the back (downstream) side and whether the rakes return to the bottom of the bar screen from the front or back. Each type has its advantages and disadvantages, although the general mode of operation is similar. In general, front-cleaned, front-return screens [see Fig. 5–3(a)] are more efficient in terms of retaining captured solids, but they are less rugged and are susceptible to jamming by solids that collect at the base of the rake. In front-cleaned, back-return screens, the cleaning rakes return to the bottom of the bar screen on the downstream side of the screen, pass under the bottom of the screen, and clean the bar screen as the rake rises. The potential for jamming is minimized, but a hinged plate, which is also subject to jamming, is required to seal the pocket under the screen.

In back-cleaned screens, the bars protect the rake from damage by the debris. However, a back-cleaned screen is more susceptible to solids carryover to the downstream side, particularly as rake wipers wear out. The bar rack of the back-cleaned, back-return screens is less rugged than the other types because the top of the rack is unsupported so the rake

Figure 5–3

Typical mechanically-cleaned coarse screens: (a) front-cleaned, front-return chain-driven, (b) reciprocating rake, (c) catenary, and (d) continuous belt.



tines can pass through. Most of the chain-operated screens share the disadvantage of submerged sprockets that require frequent operator attention and are difficult to maintain. Additional disadvantages include the adjustment and repair of the heavy chains, and the need to dewater the channels for inspection and repair of submerged parts.

Reciprocating Rake (Climber) Screen. The reciprocating-rake-type bar screen [see Fig. 5–3(b)] imitates the movements of a person raking the screen. The rake moves to the base of the screen, engages the bars, and pulls the screenings to the top of the screen, where they are removed. Most screen designs utilize a cogwheel drive mechanism for the rake. The drive motors are either submersible electric or hydraulic type. A major advantage is that all parts requiring maintenance are above the waterline and can be easily inspected and maintained without dewatering the channel. The front-cleaned, front-return feature minimizes solids carryover. The screen uses only one rake instead of multiple rakes that are used with other types of screens. As a result, the reciprocating rake screen may have limited capacity in handling heavy screening loads, particularly in deep channels where a long “reach” is necessary. For example, heavy leaf loadings in the autumn on screens serving combined sewer systems can lead to clogging of the screens due to the long cycle time between raking. The high overhead clearance required to accommodate the rake mechanism can limit its use in retrofit applications.

Table 5-5**Advantages and disadvantages of various types of bar screens**

Type of screen	Advantages	Disadvantages
Chain-driven screens		
Front clean/back return	Multiple cleaning elements (short cleaning cycle) Used for heavy duty applications	Unit has submerged moving parts that require channel dewatering for maintenance Less efficient screenings removal, i.e., carryover of residual screenings to screened wastewater channel
Front clean/front return	Multiple cleaning elements (short cleaning cycle) Very little screenings carryover	Unit has submerged moving parts that require channel dewatering for maintenance Submerged moving parts (chains, sprockets, and shafts) are subject to fouling Heavy objects may cause rake to jam
Back clean/back return	Multiple cleaning elements (short cleaning cycle) Submerged moving parts (chains, sprockets, and shafts) are protected by bar rack	Unit has submerged moving parts that require channel dewatering for maintenance Long rake teeth are susceptible to breakage Some susceptibility to screenings carryover
Reciprocating rake	No submerged moving parts; maintenance and repairs can be done above operating floor Can handle large objects (bricks, tires, etc.) Effective raking of screenings and efficient discharge of screenings Relatively low operating and maintenance costs Stainless steel construction reduces corrosion	Requires more headroom than other screens Long cycle time; raking capacity may be limiting Grit accumulation in front of bar may impede rake movement Relatively high cost due to stainless steel construction
Catenary	Sprockets are not submerged; most maintenance can be done above the operating floor Required headroom is relatively low	Because design relies on weight of chain for engagement of rakes with bars, chains are very heavy and difficult to handle Because of the angle of inclination of the screen (45 to 75-deg), screen has a large footprint Misalignment and warpage can occur when rakes are jammed May emit odors because of open design
Continuous belt	Multiple cleaning elements (short cleaning cycle) Can handle large objects Very little screenings carryover Most maintenance can be done above operating floor Unit is difficult to jam	Overhaul or replacement of the screening elements is a time-consuming and expensive operation

Catenary Screen. A catenary screen is a type of front-cleaned, front-return, chain-driven screen, but it has no submerged sprockets. In the catenary screen [see Fig. 5–3(c)], the rake is held against the rack by the weight of the chain. If heavy objects become jammed in the bars, the rakes pass over them instead of jamming. The screen, however, has a relatively large “footprint” and requires greater space for installation.

Continuous Belt Screen. The continuous belt screen is a continuous, self-cleaning screening belt that can be used to remove both fine and coarse solids [see Fig. 5–3(d)]. A large number of screening elements (rakes) are attached to the drive chains; the number of screening elements depends on the depth of the screen channel. Because the screen openings can range from 0.5 to 30 mm (0.02 to 1.18 in.), it can be used as either a fine or a coarse screen. When used as a fine screen, this type of screen is better able to handle coarse solids and upstream protective devices may not be required. Hooks protruding from the belt elements are provided to capture large solids such as cans, sticks, and rags. The screen has no submerged sprocket. Newer designs of internally fed, continuous-belt, rotary screens using wedge wire instead of screen fabric are structurally more rugged.

Design of Coarse Screen Installations. Considerations in the design of screening installations include (1) location; (2) approach velocity; (3) clear openings between bars or mesh size; (4) headloss through the screens; (5) screenings handling, processing, and disposal; and (6) controls. Because the purpose of coarse screens is to remove large objects that may damage or clog downstream equipment, in nearly all cases, they should be installed ahead of the grit chambers. If grit chambers are placed before screens, rags and other stringy material could foul the grit chamber collector mechanisms, wrap around air piping, and settle with the grit. If grit is pumped, further fouling or clogging of the pumps will likely occur.

In manually cleaned installations, it is essential that the velocity of approach be limited to approximately 0.45 m/s (1.5 ft/s) at average flow to provide adequate screen area for accumulation of screenings between raking operations. Additional area to limit the velocity may be obtained by widening the channel at the screen and by placing the screen at a flatter angle to increase the submerged area. As screenings accumulate, partially plugging the screen, the upstream head will increase, submerging new areas for the flow to pass through. The structural design of the screen should be adequate to prevent collapse if it becomes plugged completely.

For most mechanically cleaned coarse screen installations, two or more units should be installed so that one unit may be taken out of service for maintenance. Slide gates or recesses in the channel walls for the insertion of stop logs should be provided ahead of, and behind, each screen so that the unit can be dewatered for screen maintenance and repair. If only one unit is installed, it is essential that a bypass channel with a manually cleaned bar screen be provided for emergency use. If the mechanically cleaned screen should become inoperative, especially during unattended hours, an overflow should be provided to direct flow to the manually cleaned bar screen. Flow through the bypass channel should be prevented by a closed slide or sluice gate. The screen channel should be designed to prevent the settling and accumulation of grit and other heavy materials. An approach velocity of at least 0.4 m/s (1.25 ft/s) is recommended to minimize solids deposition in the channel. To prevent the pass-through of debris at peak flowrates, the velocity through the bar screen should not exceed 0.9 m/s (3 ft/s).

The velocity through the bar screen can be controlled by installation of a down stream head control device such as a Parshall flume, or, for screens located upstream of a pumping

station, by controlling the wetwell operating levels. If the channel velocities are controlled by wetwell levels, lower velocities can be tolerated provided flushing velocities occur during normal operating conditions.

Headloss through mechanically cleaned coarse screens is typically limited to about 150 mm (6 in.) by operational controls. The raking mechanisms are operated normally based on differential headloss through the screen or by a time clock. For time clock operation, a cycle length of approximately 15 min is recommended; however, either a high-water or high-differential contact should be provided that will place the screen in continuous operation when needed. Some mechanically cleaned screens are designed to operate at a higher speed at higher flowrates or higher differential flow conditions.

Hydraulic losses through bar screens are a function of approach velocity and the velocity through the bars. The headloss through coarse screens can be estimated using the following equation:

$$h_L = \frac{1}{C} \left(\frac{v_s^2 - v^2}{2g} \right) \quad (5-1)$$

where h_L = headloss, m (ft)

C = an empirical discharge coefficient to account for turbulence and eddy losses, typically 0.7 for a clean screen and 0.6 for a clogged screen

v_s = velocity of flow through the openings of the bar screen, m/s (ft/s)

v = approach velocity in upstream channel, m/s (ft/s)

g = acceleration due to gravity, 9.81 m/s² (32.2 ft/s²)

The headloss calculated using Eq. (5-1) applies only when the bars are clean. Headloss increases with the degree of clogging. The buildup of headloss can be estimated by assuming that a part of the open space in the upper portion of the bars in the flow path is clogged. The use of Eq. (5-1) is illustrated in Example 5-1.

EXAMPLE 5-1

Headloss Buildup in Coarse Screens Determine the buildup of headloss through a bar screen when fifty percent of the flow area is blocked off due to the accumulation of coarse solids. Assume the following conditions apply:

Approach velocity = 0.6 m/s

Velocity through clean bar screen = 0.9 m/s

Open area for flow through clean bar screen = 0.19 m²

Headloss coefficient for a clean bar screen = 0.7

Solution

- Compute the clean water headloss through bar screen using Eq. (5-1).

$$h_L = \frac{1}{C} \left(\frac{v_s^2 - v^2}{2g} \right)$$

$$h_L = \frac{1}{0.7} \left[\frac{(0.9 \text{ m/s})^2 - (0.6 \text{ m/s})^2}{2(9.81 \text{ m/s}^2)} \right] = 0.033 \text{ m}$$

- Estimate the headloss through the clogged bar screen (reducing the screen area by 50 percent results in a doubling of the velocity).

The velocity through the clogged bar screen is

$$v_s = 0.9 \text{ m/s} \times 2 = 1.8 \text{ m/s}$$

Assuming the flow coefficient for the clogged bar screen is approximately 0.6, the estimated headloss is

$$h_L = \frac{1}{0.6} \left[\frac{(1.8 \text{ m/s})^2 - (0.6 \text{ m/s})^2}{2(9.81 \text{ m/s}^2)} \right] = 0.24 \text{ m}$$

Comment Where mechanically cleaned coarse screens are used, the cleaning mechanism typically is actuated by the buildup of headloss. Headloss is determined by measuring the water level before and after the screen. In some cases, the screen is cleaned at predetermined time intervals, as well as at a maximum head differential.

Although most screens use rectangular bars, optional shapes, i.e., “teardrop” and trapezoidal, are available. For the optional shapes, the wider width dimension is located on the upstream side of the bar rack to make it easier to dislodge materials trapped between the bars. The alternative shapes also reduce headloss through the rack.

Fine Screens

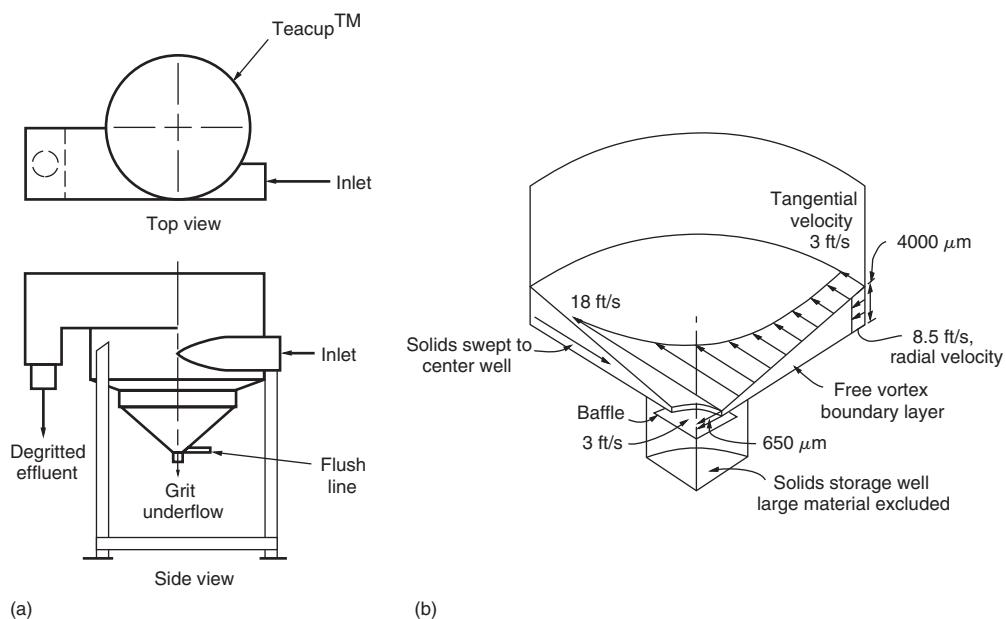
The applications for fine screens range over a broad spectrum and include preliminary treatment (following coarse bar screens), primary treatment (as a substitute for primary clarifiers), and treatment of combined sewer overflows. Fine screens can also be used to remove solids from primary effluent that could cause clogging problems in trickling filters or the membranes of membrane bioreactors. They may also be used for removal of solids from sludge flow streams prior to further treatment of the sludge (biosolids).

Screens for Preliminary and Primary Treatment. Fine screens used for preliminary treatment are (1) static wedgewire (fixed), (2) rotary drum (3) travelling belt, and (4) step type screens. Typically, the openings vary from 0.2 to 6 mm (0.01 to 0.25 in.). Examples of fine screens are illustrated on Fig. 5–4, descriptive information is provided in Table 5–6, and additional information is given below.

Fine screens may be used to replace primary treatment at small wastewater treatment plants, up to 0.13 to m^3/s (3 Mgal/d) in design capacity. Typical removal rates of BOD and TSS are reported in Table 5–7. Stainless-steel mesh or special wedge-shaped bars are used as the screening medium. Provision is made for the continuous removal of the collected solids, supplemented by water sprays to keep the screening medium clean. Headloss through the screens may range from about 0.8 to 1.4 m (2.5 to 4.5 ft).

Static Wedge Wire Screens. Static wedge wire screens [see Fig. 5–4(a)] customarily have 0.2 to 1.2 mm (0.01 to 0.06 in.) clear openings and are designed for flowrates of about 400 to 1200 $\text{L/m}^2 \cdot \text{min}$ (10 to 30 $\text{gal/ft}^2 \cdot \text{min}$) of screen area. Headloss ranges from 1.2 to 2 m (4 to 7 ft). The wedge wire medium consists of small, stainless steel, wedge-shaped bars with the flat part of the wedge facing the flow. Appreciable floor area is required for installation, and the screens must be cleaned once or twice daily with high-pressure hot water, steam, or degreaser to remove grease buildup. Static wedge wire screens are generally applicable to smaller plants or for industrial installations.

Drum Wedge Wire Screens. For the drum-type wedge wire screen [see Fig. 5–4(b) and (c)], the screening or straining medium is mounted on a cylinder that rotates in a flow channel. The construction varies, principally with regard to the direction of flow through the screening medium. In the most common arrangements, wastewater flows either into

**Figure 5-29**

Accelerated gravity separator: (a) outline sketch and (b) definition sketch. (From Hydro International.)

the entrance to the discharge port is 4.5 m/s (15 ft/s). The centrifugal force experienced by a particle within this flow field is equal to the square of the velocity divided by the radius. Because the centrifugal force is also proportional to the inverse of the radius, a fivefold decrease in the radius results in a 125-fold increase in the centrifugal force.

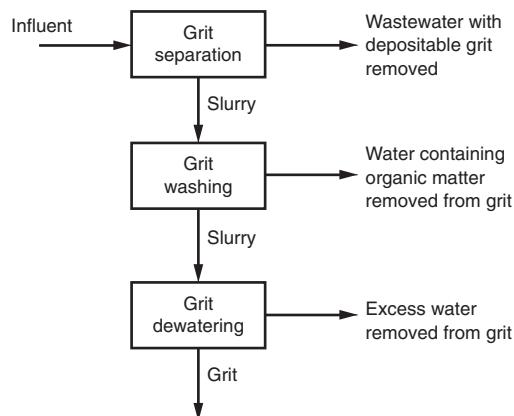
Because of the high centrifugal forces near the discharge port, some of the particles, depending on their size, density, and drag, are retained within the body of the free vortex near the center of the separator, while other particles are swept out of the unit. Grit and sand particles will be retained while organic particles are discharged from the unit. Organic particles having the same settling velocity as sand will typically be from 4 to 8 times as large. The corresponding drag forces for these organic particles will be from 16 to 64 times as great. As a result, the organic particles tend to move with the fluid and are transported out of the separator. The particles held in the free vortex ultimately settle to the bottom of the unit under the force of gravity. Organic particles that sometimes settle usually consist of oil and grease attached to grit or sand particles.

5-5 GRIT REMOVAL

Grit in wastewater consists of sand, gravel, cinders, or other heavy solid materials that have subsiding velocities or specific gravities substantially greater than those of the organic putrescible solids in wastewater. Grit removal is necessary to (1) reduce formation of heavy deposits in aeration tanks, aerobic digesters, pipelines, channels, and conduits; (2) reduce the frequency of digester cleaning caused by excessive accumulations of grit; and (3) protect moving mechanical equipment from abrasion and accompanying abnormal wear. The removal of grit is essential for equipment with closely machined metal surfaces, such as comminutors, fine screens, centrifuges, heat exchangers, and high-pressure diaphragm pumps.

Figure 5–30

A complete grit removal system, including grit separation, washing, and dewatering.
(Adapted from Wilson et al., 2007.)



The overall objectives of the grit removal system are to remove all depositable grit during normal and peak wet-weather flow and to produce an end product suitable for landfill disposal. As illustrated on Fig. 5–30, a complete grit removal system consists of three distinct unit processes each with a specific objective: (1) grit separation, (2) grit washing, and (3) grit dewatering. Before discussing these three distinct unit processes, it is appropriate to consider the characteristics of wastewater grit, as they will affect significantly the selection and design of the grit removal system. Grit separators for combined collection systems and stormwater are also reviewed along with degritting of primary sludge where grit removal is not used before primary sedimentation.

Wastewater Grit Characteristics

Traditionally, grit removal systems have been designed based on the assumption that grit consists of inorganic settleable solids ranging in size from 0.050 to 1.0 mm with settling characteristics similar to clean, spherical silica sand with a specific gravity of 2.65 and a particle size predominantly larger than 0.210 mm. As a consequence of these assumptions, many existing grit removal systems fall short of performance expectations resulting in excessive maintenance and operating costs.

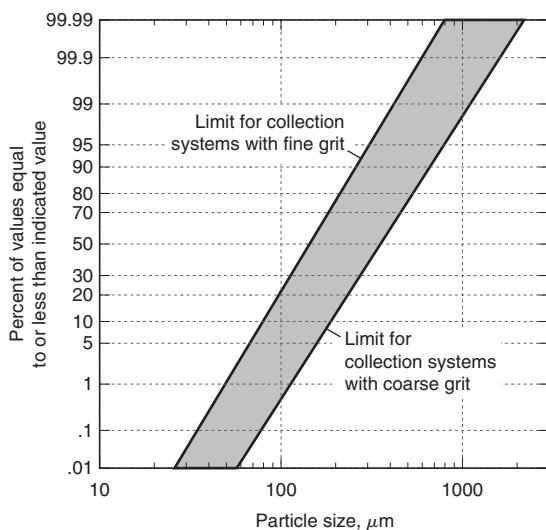
Grit Composition. Grit consists of sand, gravel, cinders, and other heavy materials. It also includes organic matter such as eggshells, bone chips, seeds, and coffee grounds. As grit travels through the collection system, grit particles come in contact with organic matter and surface active agents (SAAs) that can adhere to the grit particles. The presence of these constituents can alter the size and settling characteristics of the grit particles.

Generally, what is removed from the wastewater flow as grit is predominantly inert and relatively dry material. However, the composition can be highly variable, with moisture content ranging from 13 to 65 percent, and volatile content from 1 to 56 percent with a wide range of specific gravity. Often, enough organics are present in the grit so that it quickly putrefies if not properly handled after removal from the wastewater.

Grit Particle Size. Based on the limited available information, the actual size distribution of collected grit exhibits wide variation due to differences in collection system characteristics, as well as variations in grit-removal efficiency (see Fig. 5–31). Generally, most collected grit particles are retained on a 0.15 mm (100-mesh) sieve, reaching nearly 100 percent retention in some instances; however, particle size can vary significantly. In urban, high-density environments, wastewater grit tends to be coarser. In coastal, lower density environments, grit tends to be finer. In the southeastern United States, where fine

Figure 5-31

Typical grit particle size distribution range. Shaded region corresponds to the range of grit particle sizes measured at various wastewater treatment plants. (Adapted from Wilson et al., 2007.)



sand known as “sugar sand” constitutes a portion of the grit, less than 60 percent of the grit was retained on a 0.15 mm (100-mesh) screen in some cases.

Grit Quantities. The quantities of grit will vary greatly from one location to another, depending on the type of sewer system, the characteristics of the drainage area, the condition of the collection system, the frequency of street sanding to counteract icing conditions, the types of industrial wastes, the number of households with food waste grinders, and amount of infiltration in areas with sandy soils. Sand drift from beach areas in coastal cities is also a major contributor. Another factor in combined sewer systems is the occurrence of a first flush associated with rainfall events. The higher flowrates associated with wet weather events resuspend grit that settled in the collection system during dry weather. Thus, heavier grit loads are experienced in wastewater treatment plants soon after the start of a heavy rainfall.

Grit removal data are difficult to interpret because grit is poorly characterized and almost no data exists on relative removal efficiencies. The information on grit characteristics derives from what has been collected rather than the grit in the influent wastewater. Sieve analyses are not normally performed on grit chamber influents and effluents. For these reasons, the efficiencies of grit removal systems cannot be compared. A comparison of the reported quantities of grit removed from separate and combined sewer systems is presented in Table 5-15.

Grit Settling Characteristics. The settling characteristics of grit vary considerably from the point of entry in the collection system and its location in the treatment plant. Grit moves through gravity sewers within the collection system in three more-or-less distinct layers (see Fig. 5-32). The deposit limit for clean, inorganic grit particles for most collection systems is 0.225 mm. Consequently, particles above this size cannot be transported hydraulically under normal flow conditions and will deposit within the collection system. As SAAs in the wastewater pass over the deposited particles, some adhere to the deposited grit particles. When a sufficient amount of SAAs have accumulated, the coated grit particle buoyancy increases and deposited grit rises into the bed load. Here, the coated grit moves slowly to the treatment plant along the base of the collection system at less than the average wastewater velocity. Above the bed load is the suspended load consisting of the

Table 5–15
Comparison of quantities of grit removed from wastewater from separate and combined collection systems in aerated grit chambers

Type of collection system	Ratio of maximum day to average day	Average grit quantity	
		ft ³ /Mgal	m ³ /1000 m ³
Separate	1.5 to 3:1	0.5–5	0.004–0.037
Combined	3 to 15:1	0.5–27	0.004–0.20

lighter grit particles. This suspended grit load has a settling velocity of less than that of clean inorganic grit particles that are less than 0.225 mm in diameter and is the primary grit load that reaches the treatment plant under normal (day-to-day) conditions. Given that, traditionally, grit removal systems have been designed to remove clean inorganic grit particles that are greater than 0.210 mm, most grit passes through the grit removal flows under normal conditions. During periods of high flow, particularly in combined sewers, the heavier, deposited grit is re-suspended and the quantity of grit reaching the treatment plant increases substantially. It is, therefore, important that the grit removal system not only operate efficiently during normal flow conditions but also under sustained peak flows when the greatest volume of grit reaches the plant.

Because of SAAs, grit particles reaching the treatment plant under normal conditions are often lighter than anticipated during design. The impact of SAAs on particles size is demonstrated graphically on Fig. 5–33. Assuming that the specific gravity of clean sand is constant, the settling velocity of clean sand increases as particle size increases [see Fig. 5–33(a)]. However, the settling velocity of wastewater grit is independent of particle size because of the buoyant effect of SAAs [see Fig. 5–33(b)]. Consequently, wastewater grit passes through traditionally designed grit removal processes to the primary settling tanks where they settle out with the primary sludge or are passed to the aeration tanks. When exposed to biological activity, the SAAs decompose and the remaining high density grit particles settle rapidly. As a result, there is often excessive grit deposition in biological reactors, such as aeration tanks, aerobic digesters, and anaerobic digesters, that leads to frequent and costly cleaning to maintain process efficiency. Typically, two-thirds of grit that enters a WWTP is larger than the deposit limit size of clean sand.

Sand Equivalent Size. Many grit removal systems have been designed based on grit particles having a specific gravity of 2.65 (similar to silica sand) and a particle size predominantly larger than 0.210 mm and settling characteristics similar to clean sand. Yet, grit particles that are 0.210 mm and larger are often cited as the cause of most downstream problems. The sand equivalent size (SES) relates the settling velocity of wastewater grit to that of clean sand as shown on Fig. 5–33(c). As shown, a wastewater grit particle (coated

Figure 5–32

Distribution of suspended, bed load, and deposited grit particles in a collection system. (Adapted from Wilson et al., 2007.)

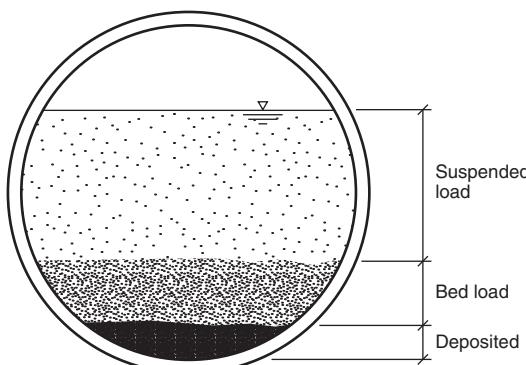
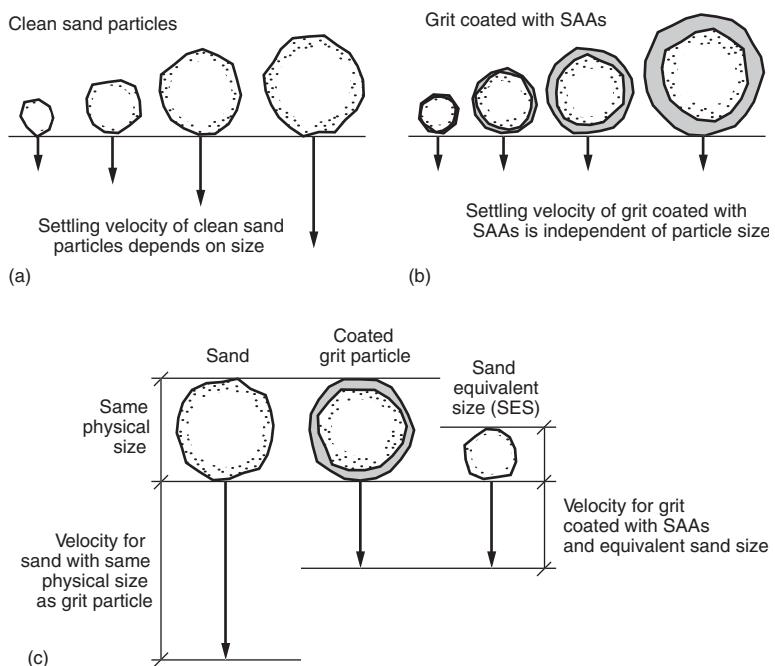


Figure 5-33

Settling velocities of grit particles reaching the treatment plant:
 (a) clean sand, (b) frothy grit coated with surface active agents, (c) equivalent size of coated grit particles as compared to clean sand particles. (Adapted from Wilson et al., 2007.)



with SAAs) with the same physical size as a clean particle of sand will have a lower settling velocity than the clean sand particle. The SES of a wastewater grit particle is the equivalent size to that of a clean sand particle size that has the same settling velocity.

Settling Velocity of Wastewater Grit. The level of grit that must be removed should be decided on a case-by-case basis. It is dependent on the characteristics of the wastewater grit entering the treatment plant and the level of removal that is required to protect the downstream treatment processes and equipment. While previous practice targeted clean particles of 0.225 mm, modern removal systems will typically target wastewater grit with a much lower SES. High, efficiency grit systems typically target grit with an SES ranging from 0.075 to 0.150 mm.

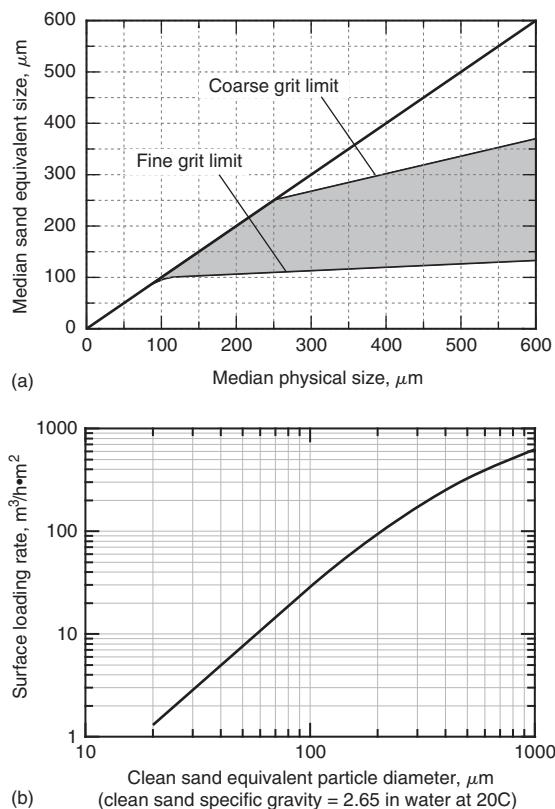
When possible, a grit study should be conducted to determine the SES of the incoming wastewater grit and the design SES to achieve the desired level of grit removal. If a grit study is not viable, regional data can be used as a substitute. When data are not available, design can be based on a qualitative evaluation of the collection system. The physical size of wastewater grit particles with their associated SES due to the impact of SAAs as measured at various locations are compared on Fig. 5-34(a). The physical size and the SES of wastewater grit begins to diverge at 0.106 mm with the disparity increasing as the physical size increases. For example, a physical particle size of 0.210 mm, the SES could vary from 0.106 to 0.210 mm. Consequently, to remove particles with a physical size of 0.210 mm, an SES as low as 0.106 mm may have to be used as a basis of design.

A design based on an SES of 0.106 mm will result in 90 percent grit removal for most collection systems. However, for those collection systems where fine silt is expected, the grit removal efficiency might be significantly lower (50 to 65 percent). If the goal is to achieve 90 percent grit removal, a design SES as low as 0.075 mm may be required.

Once the design SES has been identified, the surface loading rate (settling velocity) of the targeted grit particle can be established using the settling characteristics of clean sand [see Fig. 5-34(b)]. For an SES of 0.106 mm, the surface loading rate is 0.49 m/min

Figure 5–34

Process design information for vortex type grit removal units:
 (a) comparison of median physical size to SES at various US wastewater treatment plants (adapted from Hydro International) and (b) surface loading rate for sizing vortex grit units based on Eq. (5–18).



(12 gal/ft²·min). For an SES of 0.075 mm, the required surface loading rate is 0.24 m/min (6 gal/ft²·min), which doubles the size of the required grit removal system.

Grit Separators for Wastewater

The separation of grit from wastewater is usually accomplished in separate grit chambers designed to physically separate heavy grit particles from lighter organic solids. Grit chambers are most often located after the bar screens and before the primary sedimentation tanks to prevent screening debris from impacting the operation and maintenance of the grit removal equipment. For those treatment plants using comminution equipment, the grit chambers should be located upstream to reduce the wear on cutting blades. There are three general types of grit separation devices: horizontal-flow grit chambers, of either a rectangular or a square configuration, aerated grit chambers, or vortex grit chambers. Each type is considered separately below along with the associated washing and drying equipment that is typically provided. Grit washing, drying, and disposal are considered after the discussion of the grit separation processes.

Horizontal-Flow Grit Chambers. Rectangular and square horizontal-flow grit chambers have been used for many years. Their use, however, in new installations has been limited in favor of aerated and vortex-type grit chambers.

Rectangular Horizontal-Flow Grit Chambers. The oldest type of grit chamber used is the rectangular horizontal-flow, velocity-controlled type. Representative design data for

Table 5-16**Typical design information for horizontal-flow grit chambers**

	U.S. customary units			SI units		
	Unit	Range	Typical	Unit	Range	Typical
Detention time	s	45–90	60	s	45–90	60
Horizontal velocity	ft/s	0.8–1.3	1.0	m/s	0.25–0.4	0.3
Settling velocity for removal of:						
0.21 mm (70-mesh) material	ft/min ^a	3.2–4.2	3.8	m/min ^a	1.0–1.3	1.15
0.149 mm (100-mesh) material	ft/min ^a	2.0–3.0	2.5	m/min ^a	0.6–0.9	0.75
Headloss in a control section as percent of depth in channel	%	30–40	36 ^b	%	30–40	36 ^b
Added length allowance for inlet and outlet turbulence	%	25–50	30	%	25–50	30

^aIf the specific gravity of the grit is significantly less than 2.65, lower velocities should be used.

^bFor Parshall flume control.

rectangular horizontal-flow grit chambers are presented in Table 5-16. These units are designed to maintain a velocity as close to 0.3 m/s (1.0 ft/s) as practical and to provide sufficient time for grit particles to settle to the bottom of the chamber. The design velocity will carry most organic particles through the chamber and will tend to re-suspend any organic particles that settle but will permit the heavier grit to settle out.

The basis of design of rectangular horizontal-flow grit chambers is that, under the most adverse conditions, the lightest particle of depositable grit will reach the bed of the channel prior to its outlet end. Typically, grit chambers were designed to remove all grit particles that will be retained on a 0.21-mm-diameter (70-mesh) screen, although many chambers have been designed to remove grit particles retained by a 100-mesh screen (i.e., 0.15-mm-diameter). The settling velocity to be used should be based on the SES of the wastewater grit using the settling rates from Fig. 5-34(b). The length of channel is governed by the depth required by the settling velocity and the control section, and the cross-sectional area will be governed by the rate of flow and by the number of channels. Additional channel length should be added to allow for inlet and outlet turbulence.

Grit removal from horizontal-flow grit chambers is accomplished usually by a conveyor with scrapers, buckets, or plows. Screw conveyors or bucket elevators are used to elevate the removed grit for washing or disposal. In small plants, grit chambers are sometimes cleaned manually.

Square Horizontal-Flow Grit Chambers. Square horizontal-flow grit chambers, such as those shown on Fig. 5-35, have also been in use for over 60 y. Influent to the units is distributed over the cross section of the tank by a series of vanes or gates, and the

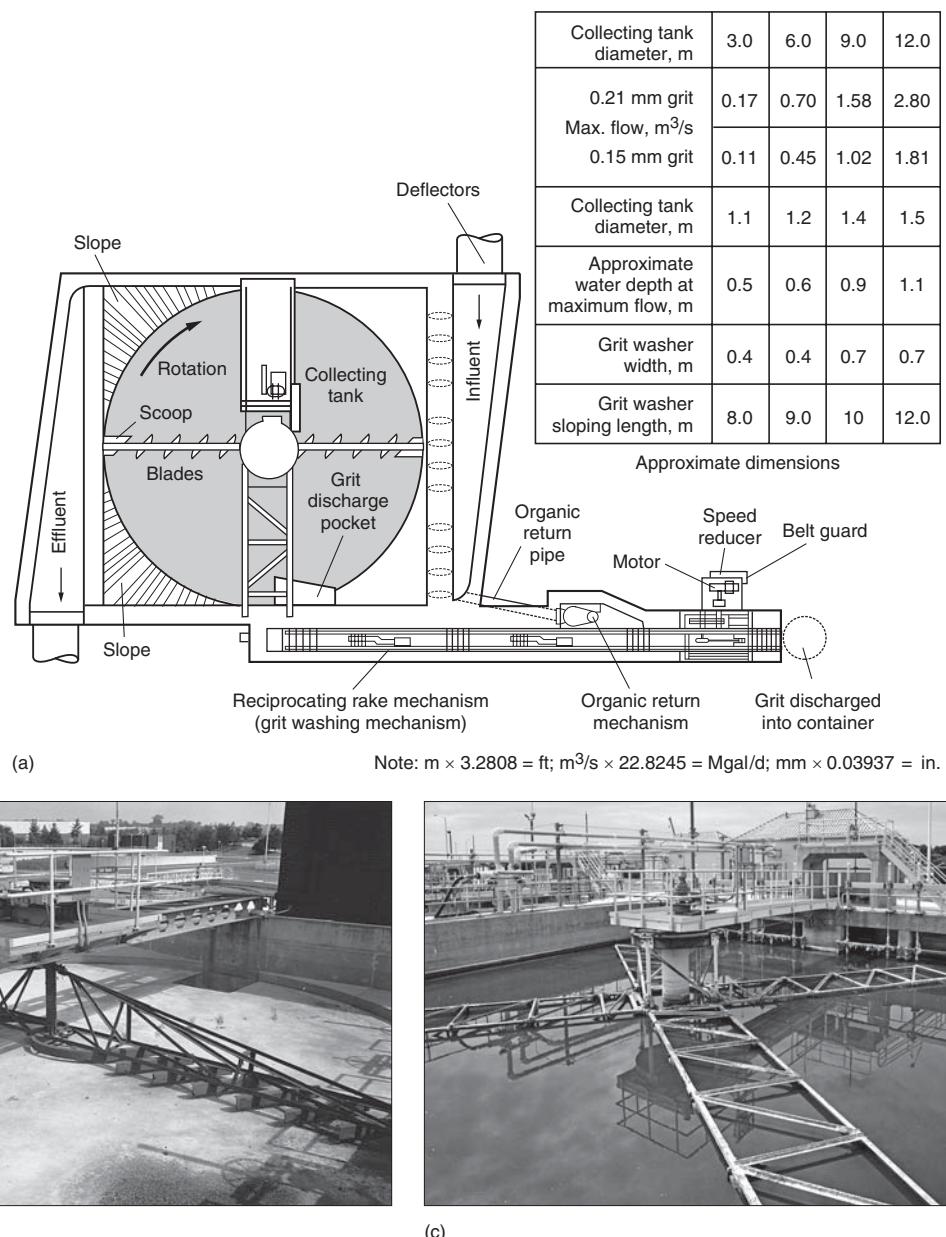


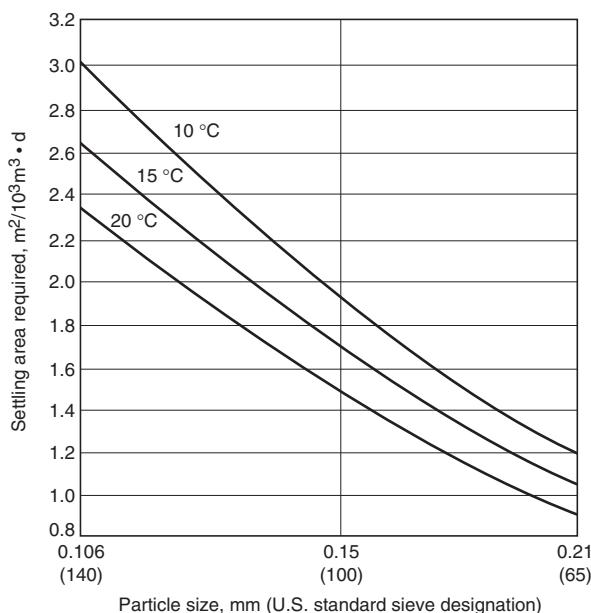
Figure 5-35

Typical square horizontal-flow grit chambers: (a) schematic with design information based on clean grit with a specific gravity of 2.65, (b) view of empty basin. The two rakes are used to move settled grit to the periphery for removal and (c) view of square grit chamber.

distributed wastewater flows in straight lines across the tank and overflows a weir in a free discharge. Where square grit chambers are used, it is generally advisable to use at least two units. These types of grit chambers are designed on the basis of overflow rates that are dependent on particle size and the temperature of the wastewater. Typically, these units have been designed to remove 95 percent of the 0.15-mm-diameter particles at peak flow.

Figure 5-36

Effect of wastewater temperature on the area required for settling grit particles with a specific gravity of 2.65 in a horizontal-flow grit chamber.

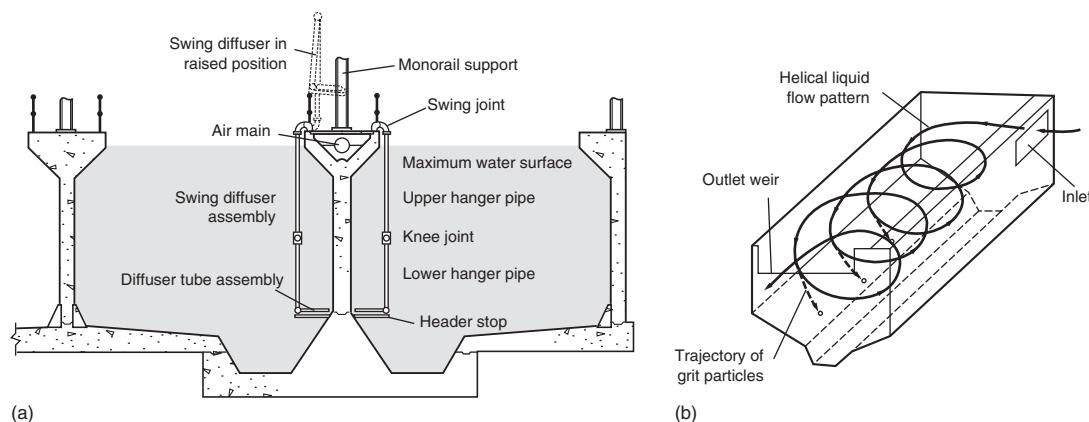


A typical set of design curves is shown on Fig. 5-36. The SES should be used for determining the settling area required.

In square grit chambers, the solids are removed by a rotating raking mechanism to a sump at the side of the tank (see Fig. 5-35). Settled grit may be moved up an incline by a reciprocating rake mechanism or an inclined screw conveyor. Grit may also be pumped from the tank through a cyclone degritter to separate the remaining organic material and concentrate the grit. The concentrated grit may then be washed again in a classifier using a submerged reciprocating rake or an inclined-screw conveyor. By either method, organic solids are separated from the grit and flow back into the basin, resulting in a cleaner, dryer grit.

Aerated Grit Chambers. In aerated grit chambers, air is introduced along one side of a rectangular tank to create a spiral flow pattern perpendicular to the flow through the tank (see Fig. 5-37). Wastewater, as shown on Fig. 5-37(b), will move through the tank in a spiral path and will make two to three passes across the bottom of the tank at maximum flow and more passes at average flow. The heavier grit particles settle to the bottom of the tank. Lighter, principally organic particles remain in suspension and pass through the tank. The velocity of roll or agitation governs the size of particles of a given specific gravity that will be removed. If the velocity is too high, grit will be carried out of the chamber; if it is too low, organic material will be removed with the grit. The quantity of air is easily adjusted. With proper adjustment, almost 100 percent of depositable grit will be removed and the grit will be well washed.

Design Considerations. Aerated grit chambers have been designed to remove 0.21-mm-diameter particles or larger, with 2- to 5-min detention periods at the peak hourly rate of flow. The cross section of the tank is similar to that provided for spiral circulation in activated-sludge aeration tanks, except that a grit hopper about 0.9 m (3 ft) deep with steeply sloping sides is located along one side of the tank under the air diffusers (see Fig. 5-37). The air diffusers are located about 0.45 to 0.6 m (1.5 to 2 ft) above the normal plane of the bottom. Influent and effluent baffles are used frequently for hydraulic control and improved grit-removal effectiveness. To determine the required headloss through the

**Figure 5-37**

Typical aerated grit chamber: (a) cross-section through grit chamber and (b) schematic of helical flow pattern through an aerated grit chamber.

chamber, the expansion in volume caused by the air must be considered. Basic design data for aerated grit chambers are presented in Table 5–17. The design of aerated grit chambers is illustrated in Example 5–8.

Grit Removal Facilities. The removal of grit from aerated grit chambers can be accomplished using grab buckets traveling on monorails and centered over the grit collection and storage trough. Other installations have been equipped with chain-and-bucket conveyors, running the full length of the storage troughs, which move the grit to one end of the trough and elevate it above the wastewater level in a continuous operation. Screw conveyors, tubular conveyors, jet pumps, and airlifts have also been used to remove the collected grit. Grit-removal equipment for aerated grit chambers is subject to the same wear as experienced in the horizontal-flow units.

Table 5-17
Typical design information for aerated grit chambers

Item	U.S. customary units			SI units		
	Unit	Range	Typical	Unit	Range	Typical
Detention time at peak flowrate	min	2–5	3	min	2–5	3
Dimensions:						
Depth	ft	7–16		m	2–5	
Length	ft	25–65		m	7.5–20	
Width	ft	8–23		m	2.5–7	
Width-depth ratio	Ratio	1:1 to 5:1	1.5:1	Ratio	1:1 to 5:1	1.5:1
Length-width ratio	Ratio	3:1 to 5:1	4:1	Ratio	3:1 to 5:1	4:1
Air supply per unit of length	ft ³ /ft·min	3–8		m ³ /m·min	0.2–0.5	
Grit quantities ^a	ft ³ /Mgal	0.5–27	2	m ³ /10 ³ m ³	0.004–0.20	0.015

^a From combined collection system.

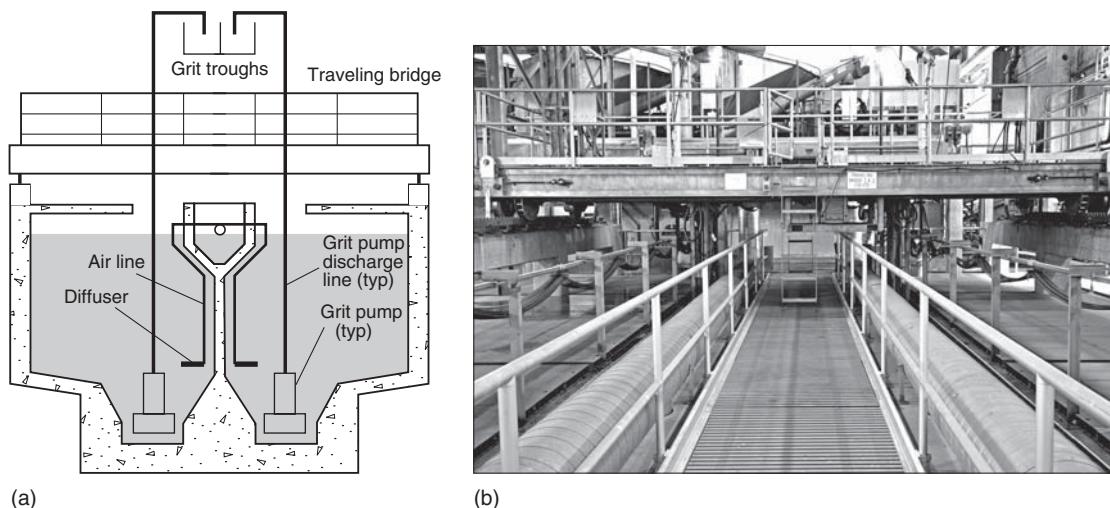


Figure 5-38

Aerated grit chamber with traveling bridge type grit removal system: (a) pumps mounted on the traveling bridge for removal of grit from the grit hopper [diffusers create the helical flow pattern as shown in Fig. 5-37(b)] and (b) view of covered aerated grit chamber equipped with traveling bridge.

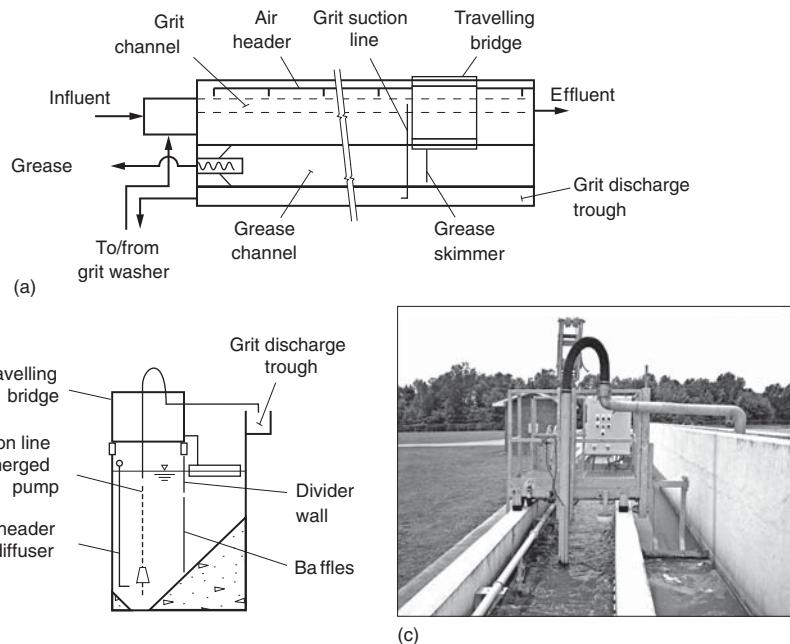
For large installations, traveling-bridge grit collectors, as shown on Fig. 5-38, are being used. Grit pumps are immersed in the grit chambers and travel the entire length, pumping grit into a stationary grit collection trough. The pumps can operate continuously or they can be programmed to run on cycles based on time or flow. A variation of this system uses a grit air lift pump installed on the traveling bridge for each grit tank rather than grit pumps. In another variation, a scraper blade attached to a traveling bridge is used in combination with grit air lifts to push heavier accumulations of grit to a pit at the inlet end of the grit chamber where it is removed using a grit pump. The scraper blade is installed on a pivoting arm and is raised during reverse travel of the bridge. Troughs receiving the pumped grit slurry must be designed for abrasion resistance.

Scum Removal. Aerated grit chambers may also be designed or existing grit chambers modified to provide FOG/scum removal (see Fig. 5-39). In this application, a baffle wall is provided that passes longitudinally through the aerated grit chamber. The baffle wall extends below the water surface and separates the tank into two parallel channels; a grit channel and a grease channel. The rising air buoys up entrained grease within the wastewater and brings it to the surface where the spiral roll moves the grease across the grit channel towards the longitudinal baffle. The lighter grease particles move through openings in the baffle to the grease channel. The heavier grit particles settle and move downward, where they collect in the channel hopper at the bottom, similar to a conventional aerated grit chamber. The grease collected in the grease channel is continuously moved by air/water skimming jets to the discharge end of the channel, where the collected grease is removed by a grease removal screw.

Emissions from Aerated Grit Chambers. Influent wastewater may contain constituents that when subjected to air agitation may be stripped from the wastewater. Where wastewater has been retained in the collection system for a long time, especially in warm

Figure 5–39

Aerated grit chamber with scum and FOG removal for small and intermediate size treatment plant: (a) plan view, (b) typical cross section, and (c) view of grit chamber (adapted from Schreiber).



climates, hydrogen sulfide and other odorous gases may be released from aerated grit chambers. In areas where industrial wastewater is discharged to the collection system, the release of volatile organic compounds (VOCs) may need to be considered as the release of significant amounts of VOCs can be a health risk to the treatment plant operators. Where emissions are an important consideration, covers may be required or non-aerated-type grit chambers used.

EXAMPLE 5–8

Design of Aerated Grit Chamber Design an aerated grit chamber for the treatment of municipal wastewater. The average flowrate is $0.5 \text{ m}^3/\text{s}$ (11.4 Mg/d), and the peaking factor curve given on Fig. 3–13 is applicable.

Solution

- Establish the peak flowrate for design. Assume that the aerated grit chamber will be designed for the 1-d sustained peak flowrate. From Fig. 3–11, the peaking factor is found to be 2.75, and the peak design flowrate is

$$\text{Peak flowrate} = 0.5 \text{ m}^3/\text{s} \times 2.75 = 1.38 \text{ m}^3/\text{s}$$

- Determine the grit chamber volume. Because it will be necessary to drain the chamber periodically for routine maintenance, use two chambers. Assume the average detention time at the peak flowrate is 3 min.

$$\text{Grit chamber volume, m}^3 \text{ (each)} = (1/2)(1.38 \text{ m}^3/\text{s})(3 \text{ min})(60 \text{ s/min}) = 124.2 \text{ m}^3$$

- Determine the dimensions of each grit chamber. Use a width-to-depth ratio of 1.2:1 and assume that the depth is 3 m.
 - Width = $1.2(3 \text{ m}) = 3.6 \text{ m}$

$$\text{b. Length} = \frac{\text{volume}}{\text{width} \times \text{depth}} = \frac{124.2 \text{ m}^3}{3 \text{ m} \times 3.6 \text{ m}} = 11.5 \text{ m}$$

4. Determine the average detention time in each grit chamber at average flow.

$$\text{Detention time} = \frac{124.2 \text{ m}^3}{(0.25 \text{ m}^3/\text{s})} = 496.8 \text{ s} \left(\frac{1 \text{ min}}{60 \text{ s}} \right) = 8.28 \text{ min}$$

5. Determine the air supply requirement. Assume that $0.3 \text{ m}^3/\text{min} \cdot \text{m}$ of length will be adequate.

$$\begin{aligned}\text{Air required (length basis)} &= (11.5 \text{ m})(0.3 \text{ m}^3/\text{min} \cdot \text{m}) \\ &= 3.45 \text{ m}^3/\text{min} \text{ for each grit chamber}\end{aligned}$$

$$\text{Total air supply required} = 3.45 \times 2 = 6.9 \text{ m}^3/\text{min} (244 \text{ ft}^3/\text{min})$$

6. Estimate the daily quantity of grit. Assume a value of $0.05 \text{ m}^3/10^3 \text{ m}^3$.

$$\begin{aligned}\text{Volume grit} &= [(0.5 \text{ m}^3/\text{s})(86,400 \text{ s/d})(0.05 \text{ m}^3/10^3 \text{ m}^3)] \\ &= 2.16 \text{ m}^3/\text{d} (76 \text{ ft}^3/\text{d})\end{aligned}$$

Comment In designing aerated grit chambers, methods of regulating the air flowrate should be provided to control grit removal and improve the cleanliness of the grit.

Vortex-Type Grit Chambers. Grit is also removed in devices that use a vortex flow pattern. Three types of devices are shown on Fig. 5-40 and discussed below.

Mechanically Induced Vortex. In the mechanically induced vortex unit illustrated on Fig. 5-40(a), wastewater is directed to the vortex unit by a long, straight inlet channel that is designed to guide the wastewater flow into the vortex unit while the grit is directed downward. The toroidal motion caused by the tangential entry is enhanced by a rotating turbine impeller within the unit that causes the grit to move along the flat floor towards the center where it passes under a separating plate into a grit slurry hopper while the lighter organic materials are maintained in suspension and are guided to the surface where they leave the unit through the tangential exit. The grit hopper contents are kept fluidized by the axial flow propeller or by supplemental water jets. Grit may be removed by a grit slurry or air-lift pump. Typical design data are presented in Table 5-18. If more than two units are installed, special arrangements for flow splitting are required to prevent settling of grit upstream of grit chambers.

Hydraulically Induced Vortex. In the hydraulically induced vortex unit, the vortex, illustrated on Fig. 5-40(c), the vortex is generated by the flow entering the unit without any mechanical rotating devices. Wastewater is directed to the unit by a long, straight inlet channel and enters tangentially into the cylindrical unit, causing the contents to rotate slowly about the vertical axis. Flow spirals down the perimeter, allowing the grit and sand particles to settle. The internal components direct the main flow away from the perimeter and back to the middle where the degritted effluent is discharged near the center of the unit into an effluent channel. The heavy grit moves down a spiral path to the center where it passes under a center cone to the grit slurry hopper.

Headloss in the unit is a function of the size particle to be removed and increases significantly for very fine particles. Vortex grit-removal units are sized to handle peak flowrates up to $0.3 \text{ m}^3/\text{s}$ (7 Mgal/d) per unit. Grit is removed from the unit by a cleated belt conveyor. Because of its overall height, this type of grit system requires a deep basement, or a lift station if it is installed above grade.

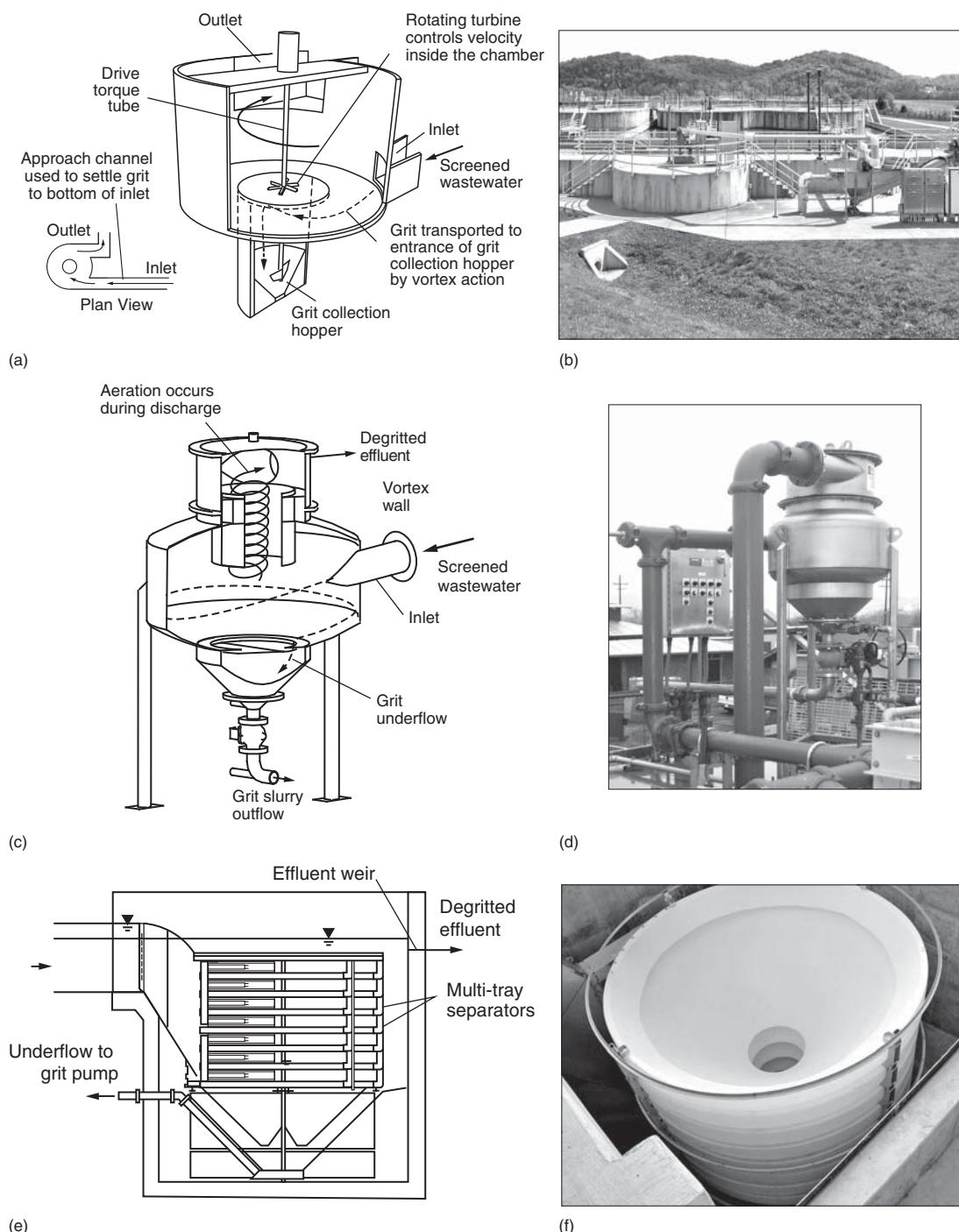


Figure 5–40

Vortex-type grit chambers: (a) schematic Pista® Grit Separator (adapted from Smith & Loveless), (b) view of typical installation (courtesy of Smith & Loveless) (c) schematic of Eutek TeaCup® separator (adapted from Hydro International), (d) view of Eutek TeaCup® separator (courtesy of Hydro International), (e) section through seven-tray Eutek HeadCell® grit separator, and (f) view of Eutek HeadCell of tray grit separator (courtesy of Hydro International).

Table 5-18**Typical design information for vortex-type grit chambers**

Item	U.S. customary units			SI units		
	Unit	Range	Typical	Unit	Range	Typical
Detention time at average flowrate	s	20–30	30	s	20–30	30
Diameter						
Upper chamber	ft	4.0–24.0		m	1.2–7.2	
Lower chamber	ft	3.0–6.0		m	0.9–1.8	
Height	ft	9.0–16.0		m	2.7–4.8	
Removal rates ^a						
0.30 mm (50 mesh)	%	92–98	95+	%	92–98	95+
0.21 mm (70 mesh)	%	80–90	85+	%	80–90	85+
0.149 mm (100 mesh)	%	60–70	65+	%	60–70	65+

^aBased on grit with a specific gravity of 2.5 to 2.65.

Multi-tray Vortex Grit Separator. The proprietary multi-tray vortex grit separator consists of multiple stacked trays which maximizes surface area and minimizing settling distances on Fig. 5-40(e). This allows for a very compact installation with low headloss. Flow is directed into the multiple tray unit by an influent distributor header which feeds a manifold which evenly distributes influent tangentially into the multiple-tray system. The tangential feed establishes a hydraulically forced vortex flow pattern where grit settles by gravity along the sloped surface of each tray and through a center opening which allows collected grit from each tray to settle into a single sump located below the center of the unit. Degritted effluent flows out of the space between the trays and into the surrounding basin where it then exits over a weir.

Grit Separators for Combined Wastewater and Stormwater

Solids-separation devices such as swirl concentrators and vortex separators have been used in Europe and, to a lesser extent, in the United States for the treatment of combined sewer overflows (CSOs) and stormwater. These devices are compact solids separation units with no moving parts.

Vortex-Type Separators. A typical vortex-type CSO solids-separation unit is illustrated on Fig. 5-41. Operation of vortex separators is based on the movement of particles within the unit. Water velocity moves the particles in a swirling action around the separator, additional flow currents move the particles toward the vortex, gravity pulls particles down, and a sweeping action moves heavier particles across the sloping floor toward the central drain.

During wet weather, the outflow from the unit is throttled, causing the unit to fill and to self-induce a swirling vortex-like flow regime. In the device shown on Fig. 5-41, secondary flow currents rapidly separate settleable grit and floatable matter. Concentrated foul matter is intercepted for treatment while the cleaner, treated flow is discharged to receiving waters. The device is intended to operate under extremely high flow regimes.

Grit Drying

The clean grit must then be dewatered to remove all free water prior to disposal. Typically, grit is disposed in sanitary landfills and depending on local regulations may have to pass a paint filter liquid test (U.S. EPA, 2004). The objective of the drying process is to achieve a clean, dry grit with a total solids concentration greater than 60 percent while retaining at least 95 percent of the depositable grit.

Disposal of Grit

The most common method of grit disposal is transport to a landfill. In some large plants, grit is incinerated with solids. As with screenings, some states require grit to be lime stabilized before disposal in a landfill. Disposal in all cases should be done in conformance with the appropriate environmental regulations. In larger plants where trucks are used to transfer grit, elevated grit storage facilities may be provided with bottom-loading gates. Difficulties experienced in getting the grit to flow freely from the storage hoppers have been minimized by using steep slopes on the storage hoppers, by applying air beneath the grit, and by the use of hopper vibrators. Drainage facilities for collection and disposal of drippings from the bottom-loading gates are desirable. Grab buckets operating on a monorail system may also be used to load trucks directly from the grit chambers. Pneumatic conveyors are sometimes used to convey grit short distances. Advantages of pneumatic conveying include (1) no elevated storage hoppers are required and (2) attendant odor problems associated with storage are eliminated. The principal disadvantage is the considerable wear on piping, especially at bends.

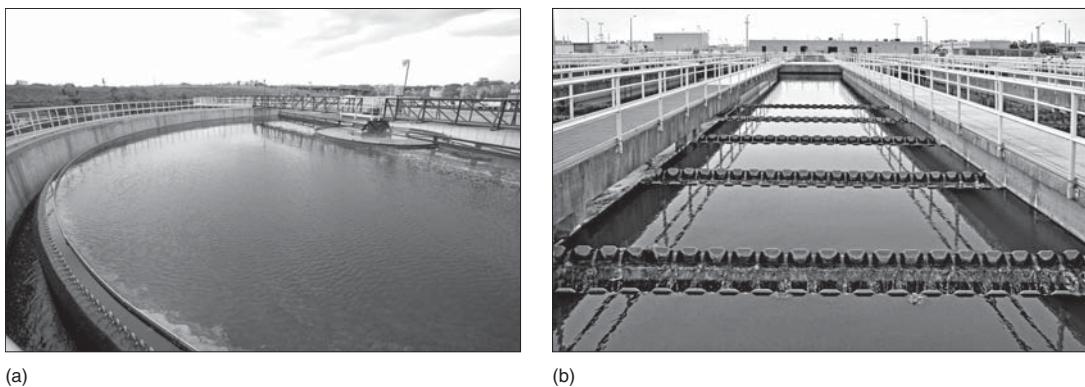
Solids (Sludge) Degritting

Where grit chambers are not used and the grit is allowed to settle in the primary settling tanks, grit removal is accomplished by pumping dilute quantities of primary sludge to a cyclone degritter. The cyclone degritter acts as a centrifugal separator in which the heavy particles of grit and solids are separated by the action of a vortex and discharged separately from the lighter particles and the bulk of the liquid. The principal advantage of cyclone degritting is the elimination of the cost of constructing, operating, and maintaining grit chambers. The disadvantages are (1) pumping of dilute quantities of solids usually requires solids thickeners and (2) pumping of grit with the liquid primary solids increases the cost of operating and maintaining solids collectors and the primary sludge pumps.

5–6 PRIMARY SEDIMENTATION

The objective of treatment by sedimentation is to remove readily settleable solids and floating material found in wastewater and thus reduce the suspended solids content. Primary sedimentation is typically the first step in the further processing of the wastewater following the removal of coarse solids and grit. Efficiently designed and operated, from 50 to 70 percent of the suspended solids and from 25 to 40 percent of the BOD can be removed by primary sedimentation tanks.

Sedimentation tanks have also been used as stormwater retention tanks to provide a moderate detention period (10 to 30 min) for overflows from either combined sewers or storm sewers. The purpose of sedimentation is to remove a substantial portion of the organic solids that otherwise would be discharged directly to the receiving waters. Sedimentation tanks have also been sized to provide detention periods sufficient for effective disinfection of such overflows. The purpose of this section is (1) to describe the various



(a)

(b)

Figure 5-44

Typical sedimentation basins used at wastewater treatment plant: (a) circular with peripheral V-notch effluent weirs and (b) rectangular with inboard V-notch effluent weirs.

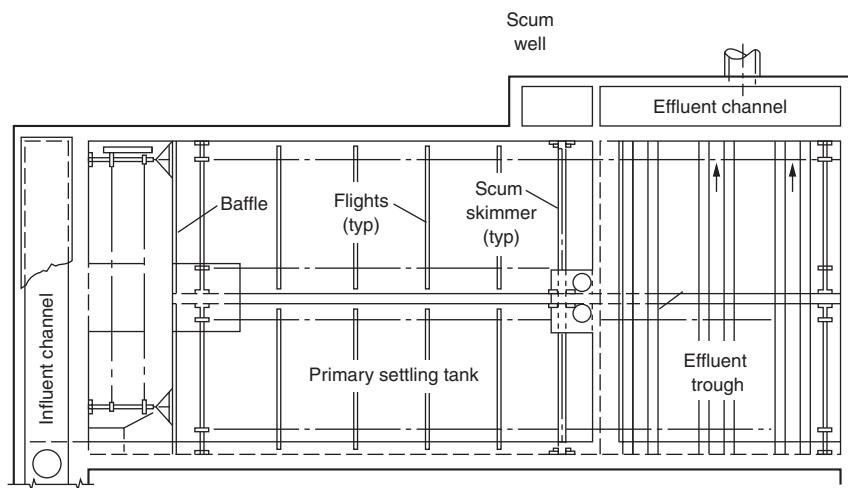
types of sedimentation facilities, (2) to consider their performance, and (3) to review important design considerations. Sedimentation tanks used for secondary treatment are considered in Chap. 8.

Description

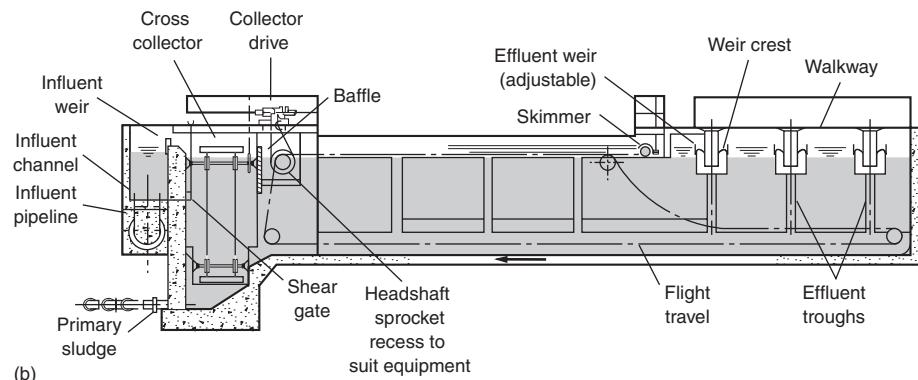
Almost all treatment plants that have primary sedimentation use mechanically cleaned sedimentation tanks of standardized circular or rectangular design (see Fig. 5-44). The selection of the type of sedimentation unit for a given application is governed by the size of the installation, by rules and regulations of local control authorities, by local site conditions, and by the experience and judgment of the engineer. Two or more tanks should be provided so that the process may remain in operation while one tank is out of service for maintenance and repair work. At large plants, the number of tanks is determined largely by size limitations.

Rectangular Tanks. Rectangular sedimentation tanks may use either chain-and-flight solids collectors or traveling-bridge-type collectors. A rectangular tank that uses a chain-and-flight-type collector is shown on Fig. 5-45. Multiple rectangular tanks require less land area than multiple circular tanks and find application where site space is at a premium. Rectangular tanks also lend themselves to nesting with preaeration tanks and aeration tanks in activated-sludge plants, thus permitting common wall construction and reducing construction costs. They are also used generally where tank roofs or covers are required. While construction costs may be less, the chain-and-flight collector systems used in rectangular tanks generally require more maintenance than the rotating sludge collector mechanism used with circular settling tanks. Important issues with rectangular tanks include (1) sludge removal, (2) flow distribution, and (3) scum removal.

Sludge Removal. Equipment for settled solids removal generally consists of a pair of endless conveyor chains, manufactured of alloy steel, cast iron, or thermoplastic. Attached to the chains at approximately 3 m (10 ft) intervals are scraper flights made of wood or fiberglass, extending the full width of the tank or bay [see Fig. 5-45(d)]. The solids settling in the tank are scraped to solids hoppers in small tanks and to transverse troughs in large tanks. The transverse troughs are equipped with collecting mechanisms (cross collectors),



(a)



(b)



(c)



(d)

Figure 5–45

Typical rectangular primary sedimentation tank: (a) plan, (b) section, (c) view of large rectangular sedimentation tank with weirs similar to those shown on (b), and (d) view of empty tank with sludge removal mechanism.

usually either chain-and-flight or screw-type collectors, which convey solids to one or more collection hoppers. In very long units (over 50 m), two collection mechanisms can be used to scrape solids to collection points near the middle of the tank length.

Where possible, it is desirable to locate solids pumping facilities close to the collection hoppers. Where cross collectors are not provided, multiple solids hoppers must be installed. Solids hoppers have operating difficulties, notably solids accumulation on the slopes and in the corners and arching over the solids drawoff piping. Wastewater may also be drawn through the solids hopper, bypassing some of the accumulated solids and resulting in a "rathole" effect. A cross collector is more advisable, except possibly in small plants, because a more uniform and concentrated solids can be withdrawn and many of the problems associated with solids hoppers can be eliminated. Rectangular tanks may also be cleaned by a bridge-type mechanism that travels up and down the tank on rubber wheels or on rails supported on the sidewalls. One or more scraper blades are suspended from the bridge. Some of the bridge mechanisms are designed so that the scraper blades can be lifted clear of the solids blanket on the return travel.

Flow Distribution. Because flow distribution in rectangular tanks is critical, one of the following inlet designs is used: (1) full-width inlet channels with inlet weirs, (2) inlet channels with submerged ports or orifices, (3) or inlet channels with wide gates and slotted baffles. Inlet weirs, while effective in spreading flow across the tank width, introduce a vertical velocity component into the solids hopper that may re-suspend the solids particles. The weirs also introduce additional headloss in the plant hydraulic profile. Inlet ports can provide good distribution across the tank width if the velocities are maintained in the 3 to 9 m/min (10 to 30 ft/min) range. Inlet baffles are effective in reducing the high initial velocities and distribute flow over the widest possible cross-sectional area. Where full-width baffles are used, they should extend from 150 mm (6 in.) below the surface to 300 mm (12 in.) below the entrance opening.

For installations of multiple rectangular tanks, below-grade pipe and equipment galleries can be constructed integrally with the tank structure and along the influent end. The galleries are used to house the sludge pumps and sludge drawoff piping. The galleries also provide access to the equipment for operation and maintenance. Galleries can also be connected to service tunnels for access to other plant units.

Scum Removal. Scum is usually collected at the effluent end of rectangular tanks with the flights returning at the liquid surface. The scum is moved by the flights to a point where it is trapped by baffles before removal. Water sprays can also move the scum. The scum can be scraped manually up an inclined apron, or it can be removed hydraulically or mechanically, and for scum removal a number of means have been developed. For small installations, the most common scum drawoff facility consists of a horizontal, slotted pipe that can be rotated by a lever or a screw. Except when drawing scum, the open slot is above the normal tank water level. When drawing scum, the pipe is rotated so that the open slot is submerged just below the water level, permitting the scum accumulation to flow into the pipe. Use of this equipment results in a relatively large volume of scum liquor.

Another method for removing scum is by a transverse rotating helical wiper attached to a shaft. Scum is removed from the water surface and moved over a short inclined apron for discharge to a cross-collecting scum trough. The scum may then be flushed to a scum ejector or hopper ahead of a scum pump. Another method of scum removal consists of a chain-and-flight type of collector that collects the scum at one side of the tank and scrapes it up a short incline for deposit in scum hoppers, whence it can be pumped to disposal units. Scum is also collected by special scum rakes in rectangular tanks that are equipped with the carriage or bridge type of sedimentation tank equipment. In installations where appreciable

amounts of scum are collected, the scum hoppers are usually equipped with mixers to provide a homogeneous mixture prior to pumping. Scum is usually disposed of with the solids and biosolids produced at the plant; however, separate scum disposal is used at many plants.

Circular Tanks. Circular sedimentation tanks are customarily arranged in groups of two or four. The flow is divided among the tanks by a flow-split structure, commonly located between the tanks. Solids are usually withdrawn by sludge pumps for discharge to the solids processing and disposal units. Important considerations include (1) method of introducing flow, (2) energy dissipation, and (3) sludge removal.

Flow Pattern. In circular tanks the flow pattern is radial (as opposed to horizontal in rectangular tanks). To achieve a radial flow pattern, the wastewater to be settled can be introduced in the center or around the periphery of the tank, as shown on Fig. 5–46. Both flow configurations have proved to be satisfactory generally, although the center-feed type is more commonly used, especially for primary treatment. In the center-feed design [see Fig. 5–46(a)], the wastewater is transported to the center of the tank in a pipe suspended from the bridge, or encased in concrete beneath the tank floor. At the center of the tank, the wastewater enters a circular well designed to distribute the flow equally in all directions [see Fig. 5–46(a)]. The center well has a diameter typically between 15 and

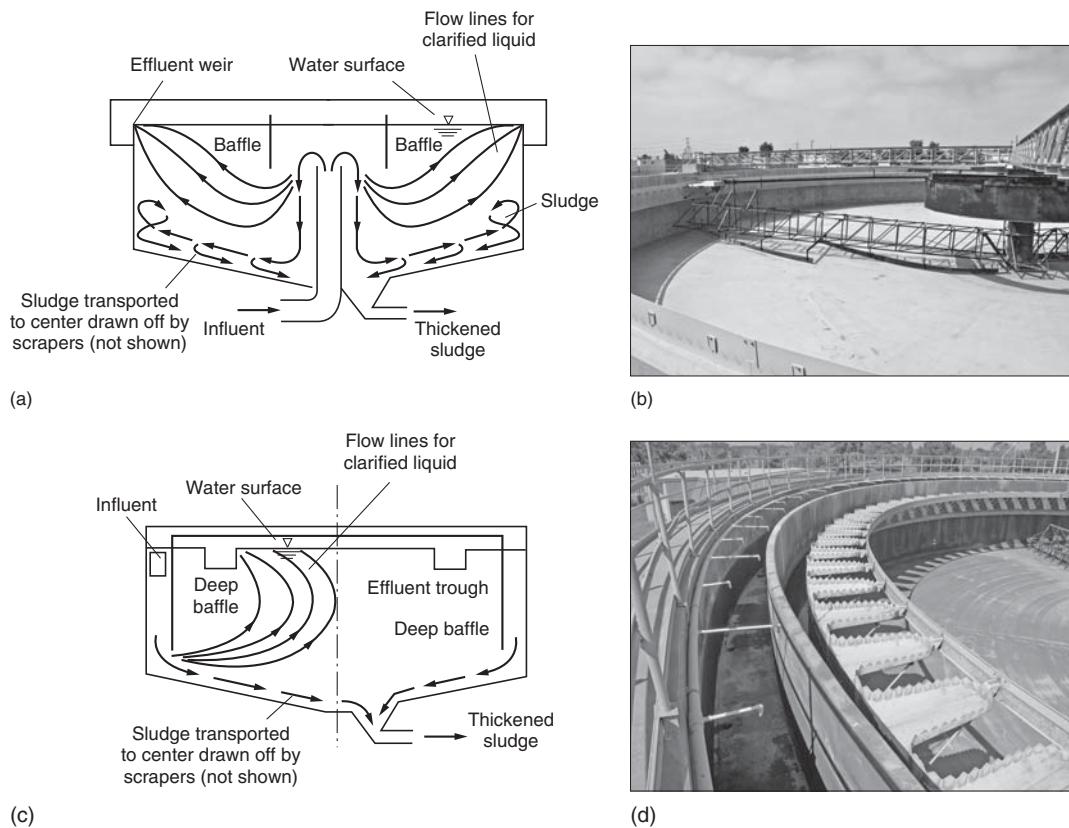
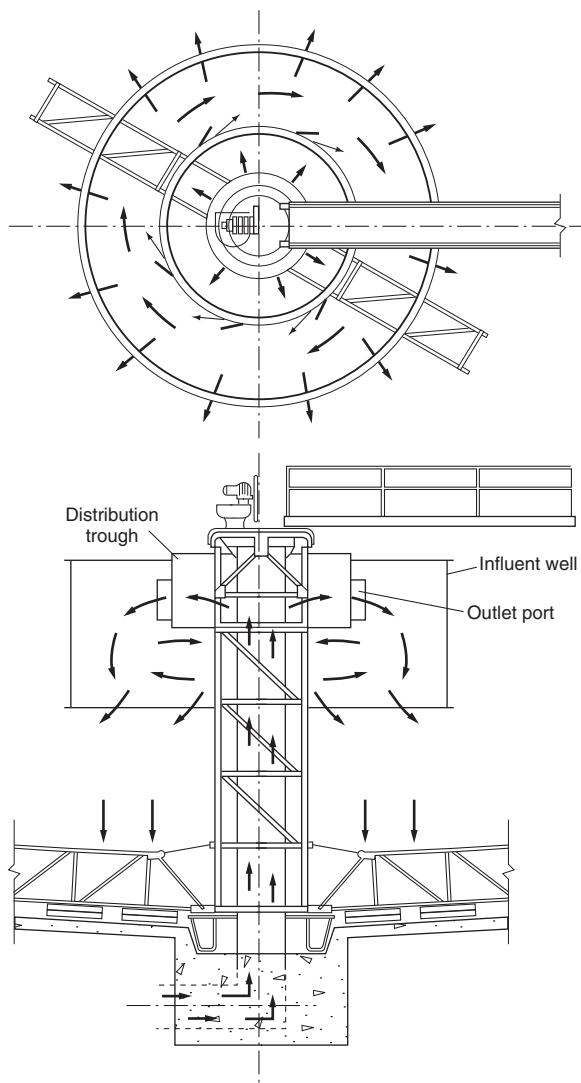


Figure 5–46

Typical circular sedimentation tanks: (a) schematic of center feed, (b) view of center feed unit, (c) schematic of peripheral feed, and (d) view of a peripheral feed unit.

Figure 5-47

Typical energy dissipating and flow distribution inlet for a center-feed sedimentation tank. The inner ring is used to create a tangential flow pattern (Randle, et al., 1992).



25 percent of the total tank diameter and ranges from 1 to 2.5 m (3 to 8 ft) in depth and should have a tangential energy-dissipating inlet within the feedwell.

Energy Dissipation. The energy-dissipating device (see Fig. 5-47) functions to collect influent from the center column and discharge it tangentially into the upper 0.5 to 0.7 m of the feedwell. The discharge ports are sized to produce a velocity of 0.75 m/s at maximum flow and 0.30 to 0.45 m/s at average flow. The feedwell should be sized so that the maximum downward velocity does not exceed 0.75 m/s. The depth of the feedwell should extend about 1 meter below the energy-dissipating inlet ports (Randall et al., 1992). An alternative approach to energy dissipation is to taper the inlet riser pipe so that the pipe increases in size from bottom to top. A center column formed in concrete can provide this configuration. The outlet of the riser pipe should terminate below the water surface. Vertical supports extending upward from the center column are used to support the center drive for the sludge collector mechanism. The tapered inlet with submerged outlet has the effect of reducing the inlet velocity of flow to the feedwell, thereby dissipating inlet energy.

In the peripheral-feed design [see Fig. 5–46(b)], a suspended circular baffle forms an annular space into which the inlet wastewater is discharged in a tangential direction. The wastewater flows spirally around the tank and underneath the baffle, and the clarified liquid is skimmed off over weirs on both sides of a centrally located weir trough. Grease and scum are confined to the surface of the annular space. Peripheral feed tanks are used generally for secondary clarification.

Sludge Removal. Circular tanks 3.6 to 9 m (12 to 30 ft) in diameter have the solids-removal equipment supported on beams spanning the tank. Tanks 10.5 m (35 ft) in diameter and larger have a central pier that supports the mechanism and is reached by a walkway or bridge. The bottom of the tank is sloped at about 1 in 12 (vertical:horizontal) to form an inverted cone, and the solids are scraped to a relatively small hopper located near the center of the tank. Airlift pumps can also be used to withdraw sludge minimizing the costs associated with operating and maintaining sludge pumps.

Combination Flocculator-Clarifier. Combination flocculator-clarifiers are often used in water treatment and sometimes used for wastewater treatment, especially in cases where enhanced settling, such as for industrial wastewater treatment or for biosolids concentration, is required. Inorganic chemicals or polymers can be added to improve flocculation. Circular clarifiers are ideally suited for incorporation of an inner, cylindrical flocculation compartment (see Fig. 5–48). Wastewater enters through a center shaft or well and flows into the flocculation compartment, which is generally equipped with a paddle-type or low-speed mixer. The gentle stirring causes flocculent particles to form. From the flocculation compartment, flow then enters the clarification zone by passing down and radially outward. Settled solids and scum are collected in the same way as in a conventional clarifier.

Stacked (Multilevel) Clarifiers. Stacked clarifiers originated in Japan in the 1960s where limited land area is available for the construction of wastewater treatment facilities. Since that time, stacked clarifiers have been used in the United States, the most notable installation of which is at the Deer Island Wastewater Treatment Plant constructed in Boston Harbor. Design of these types of clarifiers recognizes the importance of settling

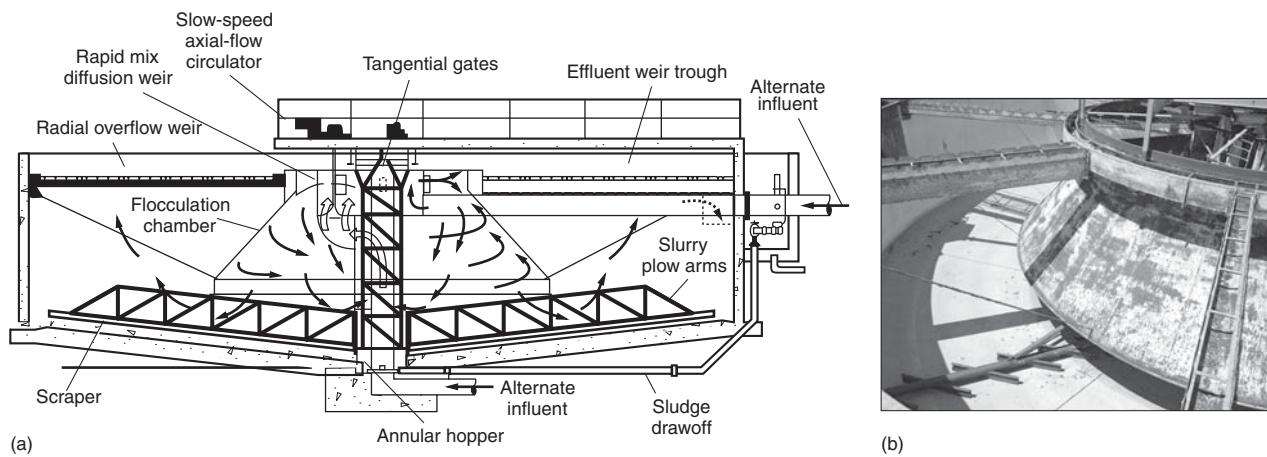
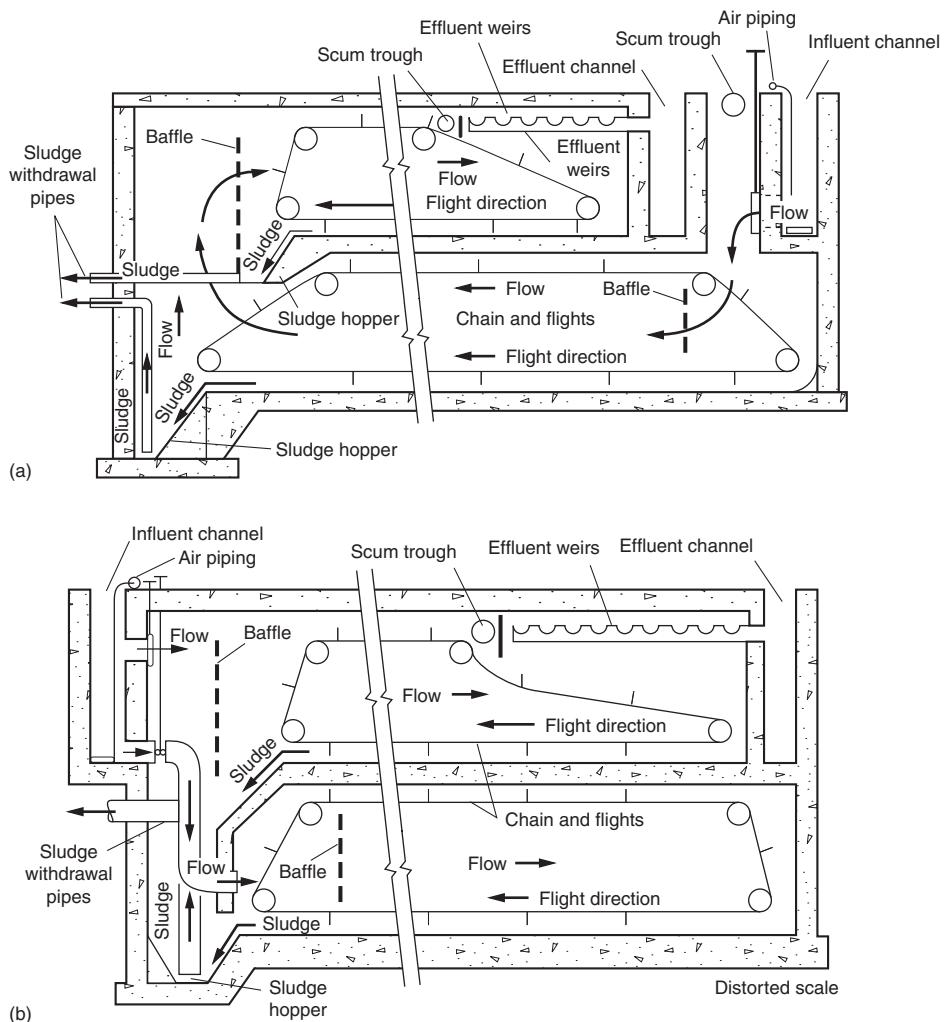


Figure 5–48

Typical flocculator-clarifier: (a) schematic and (b) view of empty tank. In some designs, turbine or propeller mixers are included in the flocculation chamber.

Figure 5-49

Typical section through a stacked clarifier (a) series flow and (b) parallel flow type used at the Deer Island Wastewater Treatment Plant, Boston, MA. Note: In the parallel flow type, the upper effluent weirs serve both the upper and lower clarifiers. Channels for the discharge of effluent from the lower to the upper clarifier are located on either side of the sludge collection mechanism in the upper clarifier.



area to settling efficiency. Operation of stacked rectangular clarifiers is similar to conventional rectangular clarifiers in terms of influent and effluent flow patterns and solids collection and removal. The stacked clarifiers are actually two (or more) tanks, one located above the other, operating on a common water surface (see Fig. 5-49). Each clarifier is fed independently, resulting in parallel flow through the lower and upper tanks. Settled solids are collected from each tank with chain and flight solids collectors, discharging to a common hopper. In addition to saving space, advantages claimed for stacked clarifiers include less piping and pumping requirements. Because the facilities are more compact and have less exposed surface area, better control of odors and volatile organic compound emissions is possible. Disadvantages include higher construction cost than conventional clarifiers and more complex structural design. Design criteria for stacked clarifiers, as regards overflow and weir rates, are similar to conventional primary and secondary clarifiers.

Sedimentation Tank Weirs. Settled effluent from sedimentation tanks is discharged over weirs into an effluent launder which, in turn, discharges into a conduit or open channel leading to the biological treatment process. Small rectangular settling tanks are often fitted with a single transverse weir on the discharge end wall. However, larger

rectangular settling tanks require additional weir length to minimize headloss and avoid short circuiting. Headloss and short circuiting can be minimized by adding a series of inboard launders with double weirs that are aligned either longitudinally (parallel to the length of the settling tank) or transversely (perpendicular to the length of the settling tank) [see Fig. 5–45(c)] or both longitudinally and transversely [see Fig. 5–50(a)].



(a)



(b)



(c)



(d)



(e)



(f)

Figure 5–50

Typical sedimentation tank appurtenances used for sedimentation tanks: (a) internal longitudinal and transverse weirs in rectangular sedimentation tanks, (b) peripheral V-notch effluent weirs launders in peripheral feed circular basin, (c) and (d) covered peripheral weirs to limit algal growth, and (e) and (f) covers for primary clarifiers to eliminate the release of odors.

Because most circular settling tanks are designed with center feed, treated effluent is discharged over a series of V-notch weirs located peripherally at the tank wall [see Figs. 5-46(a) and (b)]. A similar arrangement is used for peripheral feed circular clarifier [see Fig. 5-50(b)]. For large circular settling tanks, inboard launders with double weirs may be provided to increase the number of V-notches and reduce the headloss at high flow [see Fig. 5-46(d)]. In these cases the weir plates must be designed to balance the flow from both weirs by proportioning the number of V-notches on each launder by the proportion of tank surface area that it serves.

The V-notches are spaced equally along the tank circumference and provide a simple means to detect short circuiting when the weir is not level. The V-notches are normally 90 degree Vs and have a total depth of 75 mm (3 in.). The number of V-notches used is based on limiting the depth from about 13 to 63 mm (0.5 to 2.5 in.) from low to high flow. Surface baffles are placed in front of the peripheral weir to retain surface scum [see Figs. 5-46(b) and Fig. 5-50(b)]. Density currents tend to form along the floor and wall of the settling tanks that can re-suspend solids. Consequently, the weir launder box is constructed on the inside of the perimeter wall or horizontal baffles are provided on the tank wall to redirect the density currents. To minimize maintenance cost and to limit algal growth, peripheral weirs are often covered [see Figs. 5-50(c) and (d)].

Covers for Sedimentation Tanks. In many locations where residential development has surrounded a pre-existing wastewater treatment plant, primary sedimentation facilities are often covered to limit odorous emissions. A variety of different types of covers have been used (see Fig. 5-50(e) and (f)). The use of covers and the processing of the gases released from the sedimentation tanks is considered in greater detail in Chap. 16.

Sedimentation Tank Performance

The efficiency of sedimentation basins with respect to the removal of BOD and TSS is reduced by (1) eddy currents formed by the inertia of the incoming fluid, (2) wind-induced circulation cells formed in uncovered tanks, (3) thermal convection currents, (4) cold or warm water causing the formation of density currents that move along the bottom of the basin and warm water rising and flowing across the top of the tank, and (5) thermal stratification in hot, arid climates (Fair and Geyer, 1954). Factors that affect performance are considered in the following discussion.

BOD and TSS Removal. Typical performance data for the removal of BOD and TSS in primary sedimentation tanks, as a function of the detention time and constituent concentration, are presented on Fig. 5-51. The curves shown on Fig. 5-51 are derived from observations of the performance of actual sedimentation tanks. The curvilinear relationships in the figure can be modeled as rectangular hyperbolas using the following relationship (Crites and Tchobanoglous, 1998):

$$R = \frac{t}{a + bt} \quad (5-45)$$

where R = expected removal efficiency

t = nominal detention time, T

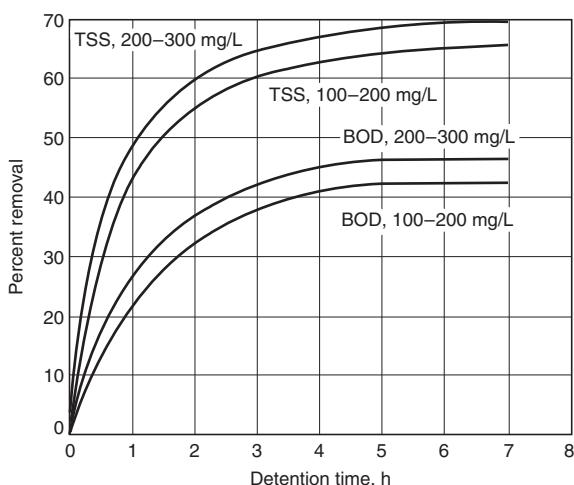
a, b = empirical constants

Typical values for the empirical constants in Eq. (5-45) at 20°C are as follows:

Item	b	a
BOD	0.020	0.018
TSS	0.014	0.0075

Figure 5–51

Typical BOD and TSS removal in primary sedimentation tanks (Greeley, 1938).

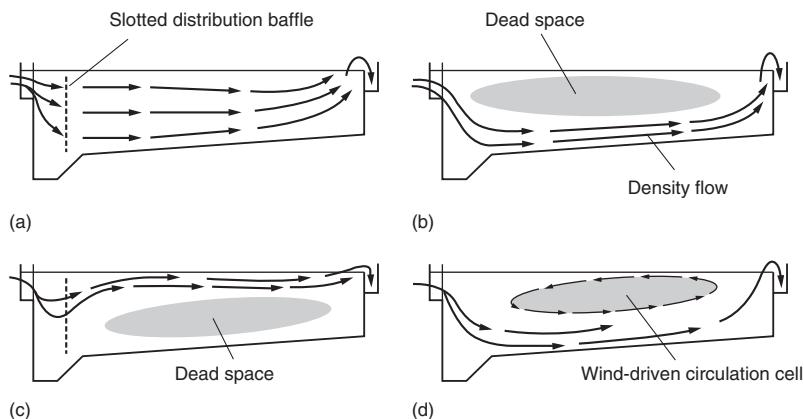


A fact that is often overlooked in sedimentation tank performance is the change in the wastewater characteristics that occurs through the sedimentation process. Larger, more slowly biodegradable suspended solids settle first, leaving a more volatile fraction in suspension that remains in the primary tank effluent. The strict use of removal curves, such as those given on Fig. 5–51, does not account for the transformation in wastewater characteristics that actually occurs. Where possible for domestic wastewater, primary tank influent and effluent should be characterized to determine concentration and composition of the constituents. Such characterization is important when determining the organic loading rate required to be treated by the succeeding biological treatment units. Further discussion on the effects of wastewater characterization on biological processes is contained in Chaps. 7 and 8.

Short Circuiting and Hydraulic Stability. In an ideal sedimentation basin, a given block of entering water should remain in the basin for the full detention time (see Fig. 5–52(a)). Short circuiting, which can lead to reduced treatment performance, can result from temperature differences [see Figs. 5–52(b) and (c)], wind driven circulation patterns [see Fig. 5–52(d)], and the presence of dead zones resulting from poor design, inadequate mixing and dispersion (see Fig. H-1 in Appendix H).

Figure 5–52

Typical flow patterns observed in rectangular sedimentation tanks:
(a) ideal flow, (b) effect of density flow or thermal stratification (water in tank is warmer than influent), (c) effect of density stratification (influent wastewater is warmer than water in tank) and (d) formation of wind-driven circulation cell (Crites and Tchobanoglous, 1998).



Temperature Effects. Temperature effects can be significant in sedimentation basins. It has been shown that a 1°C temperature differential between the incoming wastewater and the wastewater in the sedimentation tank will cause a density current to form. The impact of the temperature effects on performance will depend on the material being removed and its characteristics. Temperature effects can be more pronounced in secondary settling tanks where a less dense sludge is processed.

Wind Effects. Wind blowing across the top of open sedimentation basins can cause circulation cells to form. When circulation cells form, the effective volumetric capacity of the basin is reduced. As with temperature effects, the impact of the reduced volume on performance will depend on the material being removed and its characteristics.

Modeling Basin Performance. To determine if short circuiting exists and to what extent, tracer studies should be performed, as discussed in Appendix H. Time-concentration curves should be developed for analysis. If in the repeated tests the time-concentration curves are similar, then the basin is stable. If the time-concentration curves (also known as residence time distribution (RTD) curves (see Appendix H) are not repeatable, the basin is unstable and the performance of the basin will be erratic (Fair and Geyer, 1954). The method of influent flow distribution, as discussed above, will also affect short circuiting. The effects of short circuiting on the performance of sedimentation tanks and other reactors was examined extensively by Morrill (1932), who based on his work developed the Morrill Dispersion Index (MDI) for the evaluation of reactor performance (see Appendix H). The application of the MDI method of analysis is illustrated in Example 12-8 in Chap. 12.

Design Considerations

If all solids in wastewater were discrete particles of uniform size, uniform density, uniform specific gravity, and uniform shape, the removal efficiency of these solids would be dependent on the surface area of the tank and time of detention. The depth of the tank would have little influence, provided that horizontal velocities would be maintained below the scouring velocity. However, the solids in most wastewaters are not of such regular character but are heterogeneous in nature, and the conditions under which they are present range from total dispersion to complete flocculation. Design parameters for sedimentation are considered below. Typical design information and dimensions for rectangular and circular sedimentation tanks used for primary treatment are presented in Tables 5-19 and 5-20, respectively. Additional details on the analysis and design of sedimentation tanks may be found in WPCF, 1985. A design procedure is illustrated in Example 5-9.

Detention Time. The bulk of the finely divided solids reaching primary sedimentation tanks is incompletely flocculated but is susceptible to flocculation. Flocculation is aided by eddying motion of the fluid within the tanks and proceeds through the coalescence of fine particles, at a rate that is a function of their concentration and of the natural ability of the particles to coalesce upon collision. As a general rule, coalescence of a suspension of solids becomes more complete as time elapses; thus, detention time is a consideration in the design of sedimentation tanks. The mechanics of flocculation are such, however, that as the time of sedimentation increases, less and less coalescence of remaining particles occurs.

Normally, primary sedimentation tanks are designed to provide 1.5 to 2.5 h of detention based on the average rate of wastewater flow. Tanks that provide shorter detention periods (0.5 to 1 h), with less removal of suspended solids, are sometimes used for preliminary

Table 5–19**Typical design information for primary sedimentation tanks^a**

Item	U.S. customary units			SI units		
	Unit	Range	Typical	Unit	Range	Typical
Primary sedimentation tanks followed by secondary treatment						
Detention time	h	1.5–2.5	2.0	h	1.5–2.5	2.0
Overflow rate						
Average flowrate	gal/ft ² ·d	800–1200	1000	m ³ /m ² ·d	30–50	40
Peak hourly flowrate	gal/ft ² ·d	2000–3000	2500	m ³ /m ² ·d	80–120	100
Weir loading rate	gal/ft·d	10,000–40,000	20,000	m ³ /m·d	125–500	250
Primary settling with waste activated sludge return						
Detention time	h	1.5–2.5	2.0	h	1.5–2.5	2.0
Overflow rate						
Average flowrate	gal/ft ² ·d	600–800	700	m ³ /m ² ·d	24–32	28
Peak hourly flowrate	gal/ft ² ·d	1200–1700	1500	m ³ /m ² ·d	48–70	60
Weir loading rate	gal/ft·d	10,000–40,000	20,000	m ³ /m·d	125–500	250

^aComparable data for secondary clarifiers are presented in Chap. 8.

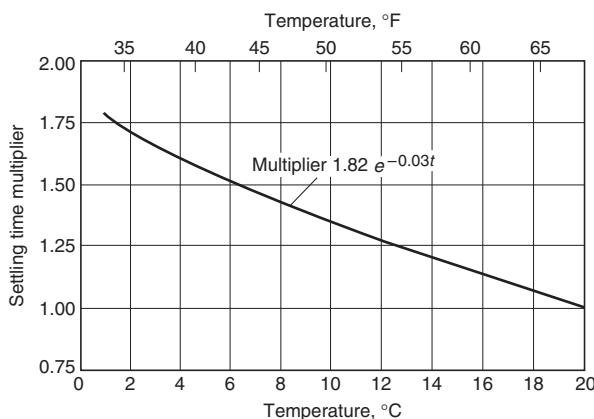
Table 5–20**Typical dimensional data for rectangular and circular sedimentation tanks used for primary treatment of wastewater**

Item	U.S. customary units			SI units		
	Unit	Range	Typical	Unit	Range	Typical
Rectangular:						
Depth	ft	10–16	14	m	3–4.9	4.3
Length	ft	50–300	80–130	m	15–90	24–40
Width ^a	ft	10–80	16–32	m	3–24	4.9–9.8
Flight speed	ft/min	2–4	3	m/min	0.6–1.2	0.9
Circular:						
Depth	ft	10–16	14	m	3–4.9	4.3
Diameter	ft	10–200	40–150	m	3–60	12–45
Bottom slope	in./ft	3/4–2	1.0	mm/mm	1/16–1/6	1/12
Flight speed	rev/min	0.02–0.05	0.03	rev/min	0.02–0.05	0.03

^aIf widths of rectangular mechanically cleaned tanks are greater than 6 m (20 ft), multiple bays with individual cleaning equipment may be used, thus permitting tank widths up to 24 m (80 ft) or more.

Figure 5-53

Curve of the increase in detention time required at cooler temperatures to achieve the same sedimentation performance as achieved at 20°C.



treatment ahead of biological treatment units. In cold climates, increases in water viscosity at lower temperatures retard particle settling in clarifiers and reduce performance at wastewater temperatures below 20°C (68°F). A curve showing the increase in detention time necessary to equal the detention time at 20°C is presented on Fig. 5-53 (WPCF, 1985). For wastewater having a temperature of 10°C, for example, the detention period is 1.38 times that required at 20°C to achieve the same efficiency. Thus, in cold climates, safety factors should be considered in clarifier design to ensure adequate performance.

Surface Loading Rates. Sedimentation tanks are normally designed on the basis of a surface loading rate (commonly termed *overflow rate*) expressed as cubic meters per square meter of surface area per day, $\text{m}^3/\text{m}^2 \cdot \text{d}$ (gallons per square foot of surface area per day, $\text{gal}/\text{ft}^2 \cdot \text{d}$). The selection of a suitable loading rate depends on the type of suspension to be separated. Typical values for various suspensions, without and with return activated sludge, are reported in Table 5-19. Designs for municipal plants must also meet the approval of state regulatory agencies, many of which have adopted standards for surface loading rates that must be followed. When the area of the tank has been established, the detention period in the tank is governed by water depth. Overflow rates in current use result in nominal detention periods of 2.0 to 2.5 h, based on average design flow.

The effect of the surface loading rate and detention time on suspended solids removal varies widely depending on the character of the wastewater, proportion of settleable solids, concentration of solids, and other factors. It should be emphasized that overflow rates must be set low enough to ensure satisfactory performance at peak rates of flow, which may vary from over 3 times the average flow in small plants to 2 times the average flow in large plants (see discussion of peak flowrates in Chap. 3).

Weir Loading Rates. In general, weir loading rates have little effect on the efficiency of primary sedimentation tanks and should not be considered when reviewing the appropriateness of clarifier design. For general information purposes only, typical weir loading rates are given in Table 5-19. The placement of weirs and baffles in secondary sedimentation applications is discussed in Sec. 8-8 of Chap. 8. Baffles should be placed ahead of effluent weirs in primary settling tanks to reduce hydraulic short circuiting. The baffles also prevent the discharge of scum into the effluent.

Scour Velocity. To avoid the resuspension (scouring) of settled particles, horizontal velocities through the tank should be kept sufficiently low. Using the results from studies

by Shields (1936) and Camp (1946) developed the following equation for the critical velocity.

$$v_H = \left[\frac{8k(s - 1)gd}{f} \right]^{1/2} \quad (5-46)$$

where v_H = horizontal velocity that will just produce scour, LT^{-1} (m/s)

k = constant that depends on type of material being scoured (unitless)

s = specific gravity of particles

g = acceleration due to gravity, LT^{-2} (9.81 m/s^2)

d = diameter of particles, L

f = Darcy-Weisbach friction factor (unitless)

Typical values of k are 0.04 for unigranular sand and 0.06 for more sticky, interlocking matter. The term f (the Darcy-Weisbach friction factor) depends on the characteristics of the surface over which flow is taking place and the Reynolds number. Typical values of f are 0.02 to 0.03. Either SI or U.S. customary units may be used in Eq. (5-46), so long as they are consistent, because k and f are dimensionless.

Computational fluid dynamic (CFD) modeling is sometimes used to optimize the design of settling tanks, particularly large diameter circular settling tanks. CFD modeling can be used to optimize feedwell diameter, feedwell depth, center column height and tank depth.

EXAMPLE 5-9

Design of a Primary Sedimentation Basin

The average flowrate at a small municipal wastewater treatment plant is $20,000 \text{ m}^3/\text{d}$. The highest observed peak daily flowrate is $50,000 \text{ m}^3/\text{d}$. Design rectangular primary clarifiers with a channel width of 6 m (20 ft). Use a minimum of two clarifiers. Calculate the scour velocity, to determine if settled material will become resuspended. Estimate the BOD and TSS removal at average and peak flow. Use an overflow rate of $40 \text{ m}^3/\text{m}^2 \cdot \text{d}$ at average flow (see Table 5-19) and a side water depth of 4 m (13.1 ft).

Solution

- Calculate the required surface area. For average flow conditions, the required area is

$$A = \frac{Q}{OR} = \frac{(20,000 \text{ m}^3/\text{d})}{(40 \text{ m}^3/\text{m}^2 \cdot \text{d})} = 500 \text{ m}^2$$

- Determine the tank length.

$$L = \frac{A}{W} = \frac{500 \text{ m}}{2 \times 6 \text{ m}} = 41.7 \text{ m}$$

However, for the sake of convenience, the surface dimensions will be rounded to 6 m by 42 m.

- Compute the detention time and overflow rate at average flow.

Using the assumed sidewater depth of 4 m,

$$\text{Tank volume} = 4 \text{ m} \times 2(42 \text{ m} \times 6 \text{ m}) = 2016 \text{ m}^3$$

$$\text{Overflow rate} = \frac{Q}{A} = \frac{(20,000 \text{ m}^3/\text{d})}{2(6 \text{ m} \times 42 \text{ m})} = 39.7 \text{ m}^3/\text{m}^2 \cdot \text{d}$$

$$\text{Detention time} = \frac{V}{Q} = \frac{(2016 \text{ m}^3)(24 \text{ h/d})}{(20,000 \text{ m}^3/\text{d})} = 2.42 \text{ h}$$

4. Determine the detention time and overflow rate at peak flow.

$$\text{Overflow rate} = \frac{Q}{A} = \frac{(50,000 \text{ m}^3/\text{d})}{2(6 \text{ m} \times 42 \text{ m})} = 99.2 \text{ m}^3/\text{m}^2 \cdot \text{d}$$

$$\text{Detention time} = \frac{V}{Q} = \frac{(2016 \text{ m}^3)(24 \text{ h/d})}{(50,000 \text{ m}^3/\text{d})} = 0.97 \text{ h}$$

5. Calculate the scour velocity (Eq. 5-46), using the following values:

Cohesion constant: $k = 0.05$

Specific gravity: $s = 1.25$

Acceleration due to gravity: $g = 9.81 \text{ m/s}^2$

Diameter of particles: $d = 100 \mu\text{m} = 100 \times 10^{-6} \text{ m}$

Darcy-Weisbach friction factor: $f = 0.025$

$$v_{H(p)} = \left[\frac{8k(s - 1)gd}{f} \right]^{1/2} = \left[\frac{(8)(0.05)(0.25)(9.81)(100 \times 10^{-6})}{0.025} \right]^{1/2} = 0.063 \text{ m/s}$$

6. Compare the scour velocity calculated in the previous step to the peak flow horizontal velocity (the peak flow divided by the cross-sectional area through which the flow passes).

The peak flow horizontal velocity through the settling tank is

$$v_{H(p)} = \frac{Q}{A_x} = \left[\frac{(50,000 \text{ m}^3/\text{d})}{2(6 \text{ m} \times 4 \text{ m})} \right] \left[\frac{1}{(24 \text{ h/d})(3600 \text{ s/h})} \right] = 0.012 \text{ m/s}$$

The horizontal velocity value, even at peak flow, is substantially less than the scour velocity. Therefore, settled matter should not be resuspended.

7. Use Eq. (5-45) and the accompanying coefficients to estimate the removal rates for BOD and TSS at average and peak flow.

- a. At average flow:

$$\text{BOD removal} = \frac{t}{a + bt} = \frac{2.42}{0.018 + (0.020)(2.42)} = 36\%$$

$$\text{TSS removal} = \frac{t}{a + bt} = \frac{2.42}{0.0075 + (0.014)(2.42)} = 58\%$$

- b. At peak flow:

$$\text{BOD removal} = \frac{t}{a + bt} = \frac{0.97}{0.018 + (0.020)(0.97)} = 26\%$$

$$\text{TSS removal} = \frac{t}{a + bt} = \frac{0.97}{0.0075 + (0.014)(0.97)} = 46\%$$

Characteristics and Quantities of Solids (Sludge) and Scum

Typical values of specific gravity and solids concentration of solids (sludge) and scum removed from primary sedimentation tanks are presented in Table 5-21. Scum consists of a variety of floatable materials, and solids concentrations vary widely. In primary sedimentation tanks used in activated-sludge plants, provision may be required for handling the excess activated sludge that may be discharged into the influent of the primary tanks for

Table 5–21

Typical values of specific gravity and solids concentration of solids and scum removed from primary sedimentation tanks

Type of solids (sludge)	Specific gravity	Solids concentration, % ^a	
		Range	Typical
Primary only:			
Medium strength wastewater	1.03	4–12	6
From combined sewer system	1.05	4–12	6.5
Primary and waste activated sludge	1.03	2–6	3
Primary and trickling filter humus sludge	1.03	4–10	5
Scum	0.95	b	—

^a Percent dry solids.

^b Range is highly variable.

settlement and consolidation with the primary sludge. For treatment plants where waste-activated sludge is returned to the primary sedimentation tanks, the primary sedimentation tanks should include provisions for light flocculent solids of 98 to 99.5 percent moisture and for concentrations ranging from 1500 to 10,000 mg/L in the influent mixed liquor.

The volume of solids produced in primary settling tanks must be known or estimated so that these tanks and subsequent solids pumping, processing, and disposal facilities can be properly designed. The solids volume will depend on (1) the characteristics of the untreated wastewater, including strength and freshness; (2) the period of sedimentation and the degree of purification to be effected in the tanks; (3) the condition of the deposited solids, including specific gravity, water content, and changes in volume under the influence of tank depth or mechanical solids-removal devices; and (4) the period between solids-removal operations. Additional information on the characteristics and quantities of solids produced during primary sedimentation and other treatment operations and processes is provided in Chap. 13.

5–7 HIGH-RATE CLARIFICATION

High-rate clarification employs physical/chemical treatment and utilizes special flocculation and sedimentation systems to achieve rapid settling. The essential elements of high-rate clarification are enhanced particle settling and the use of inclined plate or tube settlers. Advantages of high-rate clarification are (1) units are compact and thus reduce space requirements, (2) start-up times are rapid (usually less than 30 min) to achieve peak efficiency, and (3) a highly clarified effluent is produced. Enhanced particle flocculation and high-rate clarification applications are discussed in this section. Inclined plate and tube settlers were discussed previously in Sec. 5–4.

Enhanced Particle Flocculation

Enhanced particle flocculation has been used in Europe for more than 15 y but has only been introduced relatively recently in the United States. In its most basic form, enhanced particle flocculation involves the addition of an inert ballasting agent (usually silica sand or recycled chemically conditioned sludge) and a polymer to a coagulated and partially flocculated suspension. The polymer appears to coat the ballasting particles and forms the “glue” that binds the chemical floc to the ballasted particles (see Fig. 5–54). After contact with the ballasting agent, the mixture is stirred gently in a maturation tank that allows the floc particles to grow. The particles grow as the larger, faster-settling particles overtake and collide with slower-settling particles [see Fig. 5–11(b)]. The velocity gradient G for flocculation is important as a high gradient will cause a breakdown in the

7

Fundamentals of Biological Treatment

7-1	OVERVIEW OF BIOLOGICAL WASTEWATER TREATMENT	555
	Objectives of Biological Treatment	555
	Role of Microorganisms in Wastewater Treatment	555
	Types of Biological Processes for Wastewater Treatment	556
7-2	COMPOSITION AND CLASSIFICATION OF MICROORGANISMS	561
	Cell Components	562
	Cell Composition	564
	Environmental Factors	564
	Microorganism Identification and Classification	565
	Use of Molecular Tools	568
7-3	INTRODUCTION TO MICROBIAL METABOLISM	571
	Carbon and Energy Sources for Microbial Growth	571
	Nutrient and Growth Factor Requirements	573
7-4	BACTERIAL GROWTH, ENERGETICS, AND DECAY	573
	Bacterial Reproduction	574
	Bacterial Growth Patterns in a Batch Reactor	574
	Bacterial Growth and Biomass Yield	575
	Measuring Biomass Growth	575
	Estimating Biomass Yield and Oxygen Requirements from Stoichiometry	576
	Estimating Biomass Yield from Bioenergetics	579
	Stoichiometry of Biological Reactions	586
	Biomass Synthesis Yields for Different Growth Conditions	587
	Biomass Decay	587
	Observed versus Synthesis Yield	588
7-5	MICROBIAL GROWTH KINETICS	588
	Microbial Growth Kinetics Terminology	589
	Rate of Utilization of Soluble Substrates	589
	Other Rate Expressions for Soluble Substrate Utilization	591
	Rate of Soluble Substrate Production from Biodegradable Particulate Organic Matter	591
	Net Biomass Growth Rate	592
	Kinetic Coefficients for Substrate Utilization and Biomass Growth	593
	Rate of Oxygen Uptake	593
	Effects of Temperature	594
	Total Volatile Suspended Solids and Active Biomass	594
	Net Biomass Yield and Observed Yield	595
7-6	MODELING SUSPENDED GROWTH TREATMENT PROCESSES	597
	Description of Suspended Growth Treatment Processes	597
	Solids Retention Time	597
	Biomass Mass Balance	598
	Substrate Mass Balance	600

	Mixed Liquor Solids Concentration and Solids Production	600
	The Observed Yield	603
	Oxygen Requirements	603
	Design and Operating Parameters	606
	Process Performance and Stability	607
	Modeling Plug-Flow Reactors	609
7-7	SUBSTRATE REMOVAL IN ATTACHED GROWTH TREATMENT PROCESS	610
	Biofilm Characteristics	611
	Biomass Characterization	611
	Mechanistic Models	612
	Substrate Flux in Biofilms	612
	Substrate Mass Balance for Biofilm	613
	Substrate Flux Limitations	613
7-8	AEROBIC OXIDATION	615
	Process Description	615
	Microbiology	615
	Process Operation Issues	616
	Stoichiometry of Aerobic Biological Oxidation	617
	Growth Kinetics	617
	Environmental Factors	618
7-9	BIOLOGICAL OXIDATION OF INORGANIC NITROGEN	618
	Process Description	619
	Microbiology	619
	Stoichiometry of Biological Nitrification	622
	Nitification Kinetics	624
	AOB Kinetics	626
	NOB Kinetics	627
	Environmental Factors	628
7-10	DENITRIFICATION	631
	Process Description	632
	Microbiology	633
	Stoichiometry of Biological Denitrification and Denitritation	634
	Organic Substrate Requirements for Denitrification and Denitritation	635
	Denitrification Kinetics	637
	Environmental Factors	640
7-11	ANAEROBIC AMMONIUM OXIDATION	640
	Process Description	640
	Microbiology	641
	Anammox Stoichiometry	641
	Growth Kinetics	644
	Environmental Factors	645
7-12	GREENHOUSE GAS FROM BIOLOGICAL NITROGEN TRANSFORMATIONS	645
	Source of Nitrous Oxide Emissions	645
	Nitrous Oxide Production Pathways	646
7-13	ENHANCED BIOLOGICAL PHOSPHORUS REMOVAL	648
	Process Description	648
	Processes Occurring in the Anaerobic Zone	650

	Processes Occurring in a Downstream Aerobic or Anoxic Zone	650
	Microbiology	651
	Other Process Considerations for EBPR	652
	Stoichiometry of Enhanced Biological Phosphorus Removal	653
	Growth Kinetics	655
	Environmental Factors	655
7-14	ANAEROBIC FERMENTATION AND OXIDATION	655
	Process Description	656
	Microbiology	657
	Stoichiometry of Anaerobic Fermentation and Oxidation	659
	Process Kinetics	660
	Environmental Factors	663
7-15	BIOLOGICAL REMOVAL OF TOXIC AND RECALCITRANT ORGANIC COMPOUNDS	663
	Development of Biological Treatment Methods	664
	Aerobic Biodegradation	665
	Abiotic Losses	666
	Modeling Biotic and Abiotic Losses	669
7-16	BIOLOGICAL REMOVAL OF TRACE ORGANIC COMPOUNDS	671
	Removal of Trace Organic Compounds	672
	Steady-State Fate Model	672
7-17	BIOLOGICAL REMOVAL OF HEAVY METALS	674
	PROBLEMS AND DISCUSSION TOPICS	674
	REFERENCES	684

WORKING TERMINOLOGY

Term	Definition
Acetogenesis	Biological conversion of volatile acids to acetic acid and hydrogen.
Activated sludge process	Biological treatment process that involves the conversion of organic matter and/or other constituents in the wastewater to gases and cell tissue by a large mass of aerobic microorganisms maintained in suspension by mixing and aeration. The microorganisms form flocculent particles that are separated from the process effluent in a sedimentation tank (clarifier) or by membranes and are returned subsequently to the aeration process or wasted.
Aerobic (oxic) process	Biological treatment process that occur in the presence of free dissolved oxygen; oxygen is consumed by aerobic microorganisms to drive metabolic reactions.
Anaerobic fermentation	See fermentation.
Anaerobic process	Biological treatment process that occur in the absence of free dissolved oxygen and oxidized compounds.
Anammox process	An anaerobic biological treatment process in which nitrite is used by specialized planctomycete bacteria as the electron acceptor to convert ammonia and nitrite to nitrogen gas primarily.
Anoxic process	Biological treatment process that occurs in the absence of free dissolved oxygen where oxidized compounds such as nitrate and nitrite are used to drive metabolic reactions; denitrification is an example of an anoxic process.
Attached-growth process (also known as fixed-film processes)	Biological treatment process in which the microorganisms responsible for the conversion of organic matter or other constituents in the wastewater to gases and cell tissue are attached to some inert medium such as rocks, slag or specially designed ceramic or plastic materials.

Term	Definition
Bioenergetics	The study of energy transformations that take place in living organisms.
Biological nutrient removal (BNR)	The term applied to the removal of nitrogen and phosphorus in biological treatment processes.
Biofilm	An accumulation of biological growth on the surface of an object such as attached-growth media.
Biomass	The mass of microorganisms in a biological treatment process.
Carbonaceous BOD removal	Biological conversion of the carbonaceous organic matter in wastewater to cell tissue and various gaseous end products. In the conversion, it is assumed that the nitrogen present in the various compounds is converted to ammonia.
Combined aerobic/anoxic/anaerobic processes	Various combination of aerobic, anoxic and anaerobic biological treatment processes grouped together to achieve biological nitrogen and phosphorus removal.
Denitritation	Biological reduction of nitrite to nitrogen gas and other gaseous end products containing nitrogen.
Denitrification	Biological reduction of nitrate to nitrogen gas and other gaseous end products containing nitrogen.
Enhanced biological phosphorus removal	The term applied to phosphorus removal by certain bacteria with phosphorus storage capabilities greater than normally found in biological wastewater treatment.
Energetics	The scientific study of energy flows and transformations.
Facultative processes	Biological conversion processes which can occur in the presence or absence of molecular oxygen.
Fermentation (also referred to as acidogenesis)	The conversion of organic matter to carbon dioxide and other low molecular weight compounds.
Greenhouse gas	Gaseous compounds that contribute to global warming. Methane and nitrous oxide are potent greenhouse gases that can be produced in biological treatment processes.
Hybrid process	Term used to describe combined suspended and attached growth biological process.
Methanogenesis	Biological conversion of acetic acid or hydrogen and carbon dioxide to methane.
Nitratation	Biological oxidation of nitrite to nitrate.
Nitritation	Biological oxidation of ammonia to nitrite.
Nitrification	The two-step biological oxidation of nitrogen (mostly in the form of ammonia) to nitrite and then to nitrate.
Substrate	Wastewater or solids constituents used to promote biological growth.
Suspended growth process	Biological treatment process in which the microorganisms responsible for the conversion of the organic matter or other constituents in the wastewater to gases and cell tissues are maintained in suspension within the liquid.
Yield	The amount of biological solids produced relative to the amount of substrate removed.

With proper analysis and environmental control, almost all wastewaters containing biodegradable constituents can be treated biologically. Therefore, it is essential that the environmental engineer understand the characteristics of each biological process to ensure that the proper environment is produced and controlled effectively. The principal purposes of this chapter are (1) to provide fundamental background information on the microorganisms used to treat wastewater and (2) to consider the application of biological process fundamentals for the biological treatment of wastewater. The information presented in this

chapter provides the necessary background material needed for the design of biological treatment processes discussed in Chaps. 8 through 10. For ease of computation, constituent concentrations in this chapter and in Chaps. 8, 9, and 10 are expressed in g/m^3 instead of mg/L because flowrate is given in units of m^3/s or m^3/d .

The fundamentals of biological treatment introduced in the first seven sections of this chapter include (1) an overview of biological wastewater treatment, (2) the composition and classification of the microorganisms used for wastewater treatment, (3) an introduction to important aspects of microbial metabolism, (4) bacterial growth and energetics, (5) microbial growth kinetics, (6) modeling suspended growth treatment processes, and (7) modeling attached-growth treatment processes. Following the presentation of fundamentals, the remaining eight sections deal with an introduction to the general classes of biological processes used for the treatment of wastewater. The material in these sections will also serve as an introduction to the material presented in Chaps. 8, 9 and 10. Because the focus of this book is on quantifiable treatment processes, the role of algae in wastewater treatment, principally in lagoon systems, is not considered. Information on algae is available in publications from the U.S. EPA, WEF, and several textbooks. Similarly, constructed wetlands are not considered because of space constraints and the fact that there are numerous publications including several books devoted solely to the analysis, design, and implementation of these systems.

7-1 OVERVIEW OF BIOLOGICAL WASTEWATER TREATMENT

The objectives of biological treatment, some useful definitions, the role of microorganisms in the biological treatment of wastewater, and biological processes used for wastewater treatment are introduced in this section to provide a perspective for the material to be presented in this chapter.

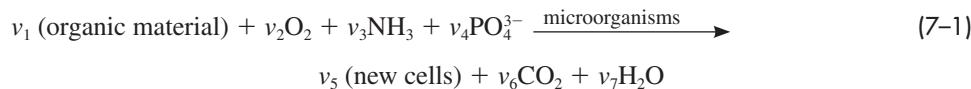
Objectives of Biological Treatment

The overall objectives of the biological treatment of domestic wastewater are to (1) transform (i.e., oxidize) dissolved and particulate biodegradable constituents into acceptable end products, (2) capture and incorporate suspended and nonsettleable colloidal solids into a biological floc or biofilm, (3) transform or remove nutrients, such as nitrogen and phosphorus, and (4) in some cases, remove specific trace organic constituents and compounds. For industrial wastewater, the objective is to remove or reduce the concentration of organic and inorganic compounds. Because some of the constituents and compounds found in industrial wastewater are toxic to microorganisms, pretreatment may be required before the industrial wastewater can be discharged to a municipal collection system. For agricultural irrigation return (drainage) water containing nutrients, the objective is to remove the nutrients, specifically nitrogen and phosphorus, that are capable of stimulating the growth of aquatic plants.

Role of Microorganisms in Wastewater Treatment

The removal of dissolved and particulate carbonaceous BOD and the stabilization of organic matter found in wastewater is accomplished biologically using a variety of microorganisms, principally bacteria. Microorganisms are used to oxidize (i.e., convert) the dissolved and particulate carbonaceous organic matter into simple end products and

additional biomass, as represented by the following equation for the aerobic biological oxidation of organic matter.



where v_i = the stoichiometric coefficient, as defined previously in Sec. 1–9 in Chap. 1.

In Eq. (7-1), oxygen (O_2), ammonia (NH_3), and phosphate (PO_4^{3-}) are used to represent oxygen and the nutrients needed for the conversion of the organic matter to simple end products [i.e., carbon dioxide (CO_2) and water]. The term shown over the directional arrow is used to denote the fact that microorganisms are needed to carry out the oxidation process. The term *new cells* is used to represent the biomass produced as a result of the oxidation of the organic matter. Microorganisms are also used to remove nitrogen and phosphorus in wastewater treatment processes. Specific bacteria are capable of oxidizing ammonia (nitrification) to nitrite and nitrate, while other bacteria can reduce the oxidized nitrogen to gaseous nitrogen. For phosphorus removal, biological processes are configured to encourage the growth of bacteria with the ability to take up and store large amounts of inorganic phosphorus.

Because the biomass has a specific gravity slightly greater than that of water, the biomass can be removed from the treated liquid by gravity settling. It is important to note that unless the biomass produced from the organic matter is removed on a periodic basis, complete treatment has not been accomplished because the biomass, which itself is organic, will be measured as BOD in the effluent. Without the removal of biomass from the treated liquid, the only treatment achieved is that associated with the bacterial oxidation of a portion of the organic matter originally present.

Types of Biological Processes for Wastewater Treatment

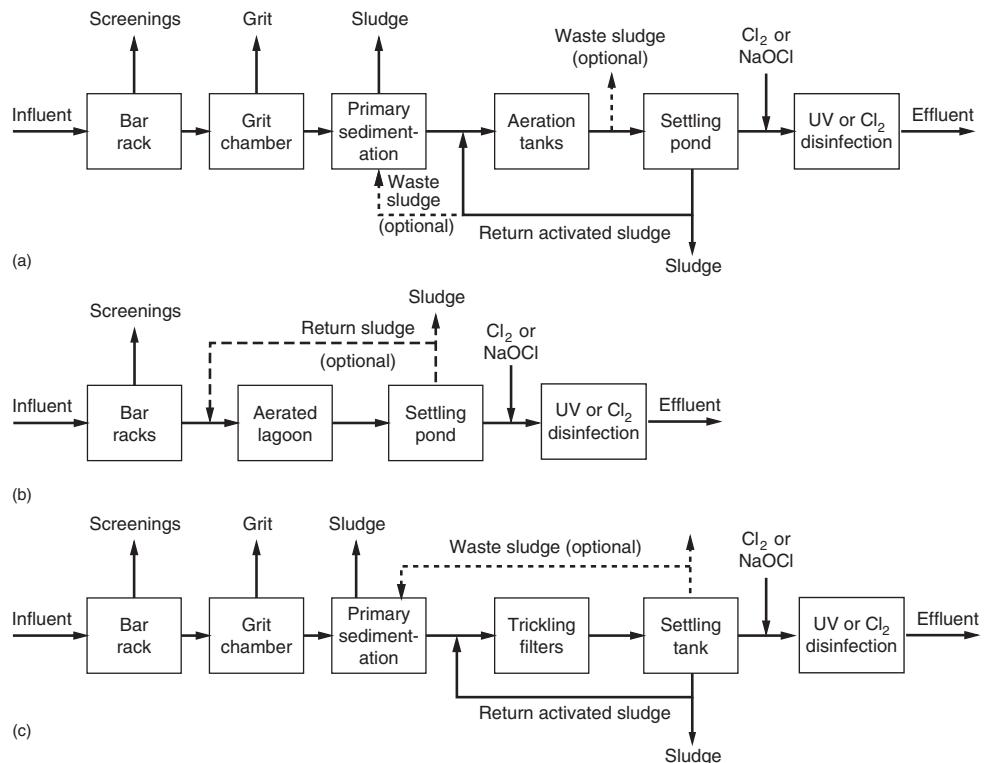
The principal biological processes used for wastewater treatment can be divided into two main categories: *suspended growth* and *attached growth* (or *biofilm*) processes. Examples of commonly used suspended and attached growth biological treatment processes are shown in Fig. 7-1. Typical process applications for suspended and attached growth biological treatment processes are given in Table 7-1, along with other treatment processes. The successful design and operation of the processes listed in Table 7-1 requires an understanding of the types of microorganisms involved, the specific reactions that they perform, the environmental factors that affect their performance, their nutritional needs, and their reaction kinetics. These subjects are considered in the sections that follow.

Suspended Growth Processes. In suspended growth processes, the microorganisms responsible for treatment are maintained in liquid suspension by appropriate mixing methods. Many suspended growth processes used in municipal and industrial wastewater treatment for biodegradation of organic substances are operated with dissolved oxygen (aerobic) or nitrate/nitrite (anoxic) utilization, but applications exist where suspended growth anaerobic (no oxygen present) reactors are used, such as for high organic concentration industrial wastewaters and organic sludges. The most common suspended growth process used for municipal wastewater treatment is the activated sludge process shown on Fig. 7-2 and discussed below.

Early Developments. The activated sludge process was developed around 1913 at the Lawrence Experiment Station in Massachusetts by Clark and Gage (Metcalf and Eddy, 1930)

Figure 7-1

Typical (simplified) flow diagrams for biological processes used for wastewater treatment:
 (a) activated sludge process,
 (b) aerated lagoons, and
 (c) trickling filters.



and by Ardern and Lockett (1914) at the Davyhulme Sewage Works, in Manchester, England. The activated sludge process was so named because it involved the production of an activated mass of microorganisms capable of stabilizing a waste under aerobic conditions. In the aeration tank, contact time is provided for mixing and aerating influent wastewater with the microbial suspension, generally referred to as the mixed liquor suspended solids (MLSS) or mixed liquor volatile suspended solids (MLVSS). Mechanical equipment is used to provide the mixing and transfer of oxygen into the process (see Sec. 5–11, Chap. 5). The mixed liquor then flows to a clarifier where the microbial suspension is settled and thickened. The settled biomass, described as *activated sludge* because of the presence of active microorganisms, is returned to the aeration tank to continue biodegradation of the influent organic material. A portion of the thickened solids is removed daily or periodically as the process produces excess biomass that would accumulate with nonbiodegradable solids contained in the influent wastewater. If the accumulated solids are not removed, the solids level in the clarifier would increase until solids eventually exit via the system effluent flow.

An important feature of the activated sludge process is the formation of floc particles, ranging in size from 50 to 200 mm, which can be removed by gravity settling, leaving a relatively clear liquid as the treated effluent. Typically, greater than 99 percent of the suspended solids can be removed in the clarification step. As will be discussed in Chap. 8, the characteristics and thickening properties of the flocculent particles will affect the clarifier design and performance.

Treatment Objective and Process Developments. The treatment objectives and process configurations for the activated sludge process have changed considerably since its

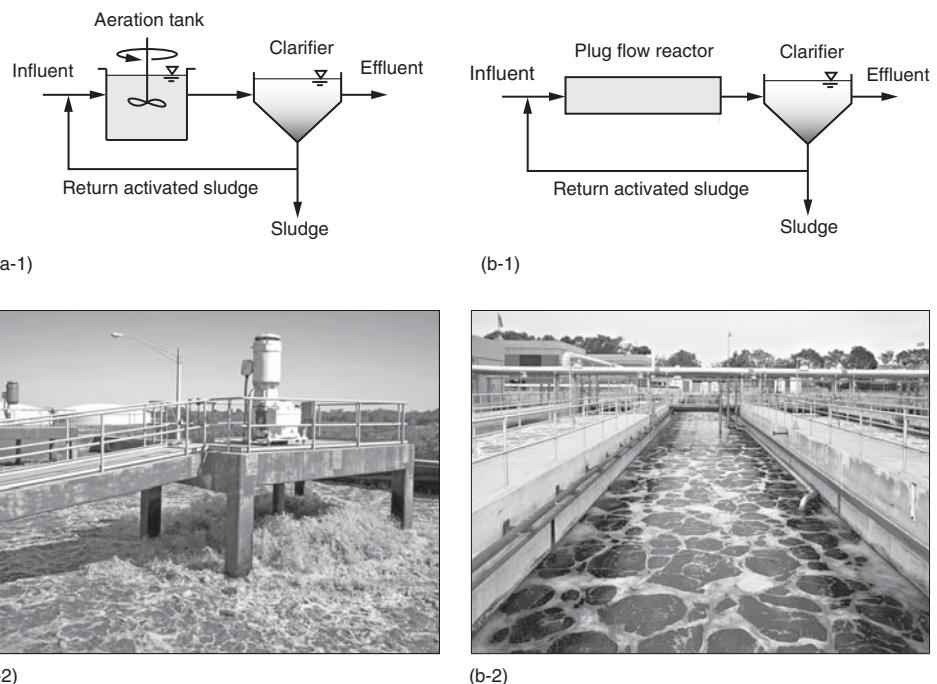
Table 7-1**Major biological treatment processes used for wastewater treatment**

Type	Common name	Use ^a
Aerobic processes:		
Suspended growth	Activated sludge process(es)	Carbonaceous BOD removal, nitrification
	Aerated lagoons	Carbonaceous BOD removal, nitrification
	Aerobic digestion	Stabilization, carbonaceous BOD removal
	Membrane bioreactor	Carbonaceous BOD removal, nitrification
	Nitritation process	Nitritation
Attached growth	Biological aerated filters	Carbonaceous BOD removal, nitrification
	Moving bed bioreactor	Carbonaceous BOD removal, nitrification
	Packed-bed reactors	Carbonaceous BOD removal, nitrification
	Rotating biological contactors	Carbonaceous BOD removal, nitrification
	Trickling filters	Carbonaceous BOD removal, nitrification
Hybrid processes	Trickling filter/activated sludge	Carbonaceous BOD removal, nitrification
	Integrated fixed film activated sludge (IFAS)	Carbonaceous BOD removal, nitrification
Anoxic processes:		
Suspended growth	Suspended-growth denitrification	Denitrification
Attached growth	Attached growth denitrification filter	Denitrification
Anaerobic processes:		
Suspended growth	Anaerobic contact processes	Carbonaceous BOD removal
	Anaerobic digestion	Stabilization, solids destruction, pathogen kill
	Anammox process	Denitrification, ammonia removal
Attached growth	Anaerobic packed and fluidized bed	Carbonaceous BOD removal, waste stabilization, denitrification
Sludge blanket	Upflow anaerobic sludge blanket	Carbonaceous BOD removal, especially high strength wastes
Hybrid	Upflow sludge blanket/attached growth	Carbonaceous BOD removal
Combined aerobic, anoxic, and anaerobic processes:		
Suspended growth	Single- or multi-stage processes, Various proprietary processes	Carbonaceous BOD removal, nitrification, denitrification, and phosphorus removal
Hybrid	Single- or multi-stage suspended growth processes with fixed film media	Carbonaceous BOD removal, nitrification, denitrification, and phosphorus removal
Lagoon processes:		
Aerobic lagoons	Aerobic lagoons	Carbonaceous BOD removal, nitrification
Maturation (tertiary) lagoons	Maturation (tertiary) lagoons	Carbonaceous BOD removal, nitrification
Facultative lagoons	Facultative lagoons	Carbonaceous BOD removal
Anaerobic-lagoons	Anaerobic-lagoons	Carbonaceous BOD removal (waste stabilization)

^a Adapted from Tchobanoglous and Schroeder (1985).

Figure 7-2

Suspended growth biological treatment process: (a-1) schematic and (a-2) view of complete-mix activated sludge process and (b-1) schematic and (b-2) view of plug-flow activated sludge process.

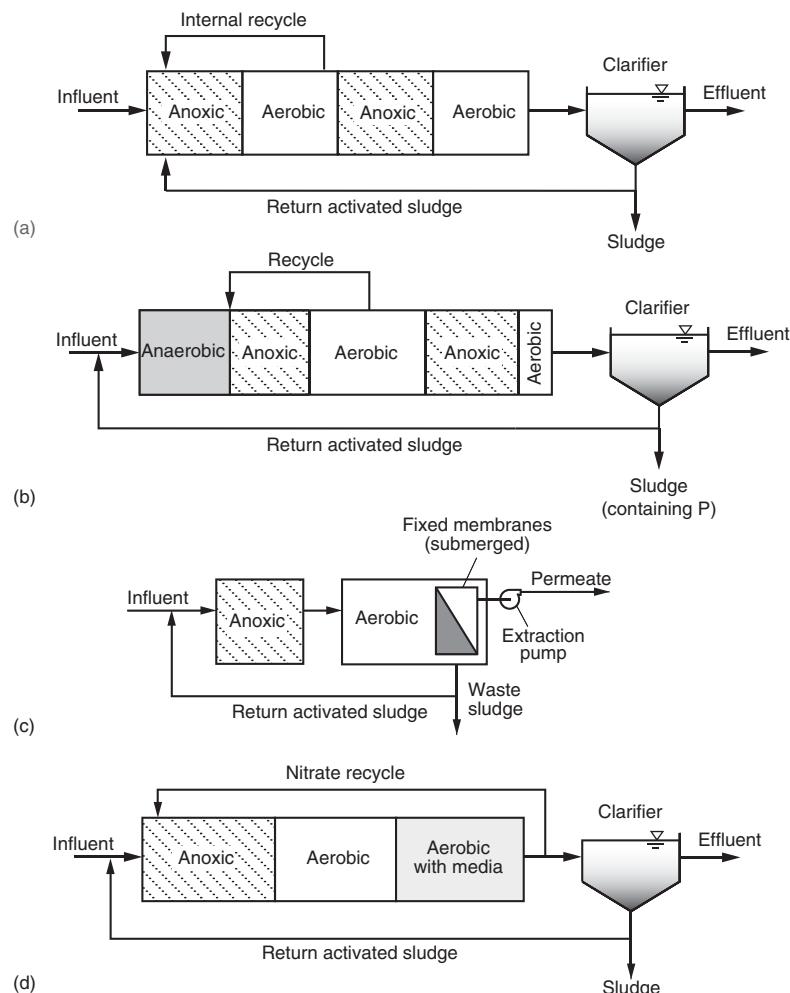


development and application in the early 1900s. Significant changes have been brought about by enhanced understanding of the process microbiology, improved characterization of the individual microorganisms involved, improvements in aeration technology, and new equipment innovations. As a result of these discoveries and innovations, the process capabilities have been expanded and treatment objectives have become more rigorous. In the late 1950s, Pasveer developed a race track configuration for activated sludge treatment (Hao et al., 1997) that led to the popular oxidation ditch system with biological nitrogen removal that is employed at hundreds of wastewater treatment plants around the world today. A common hindrance to operation and performance over the first 60 y of using activated sludge was settling problems associated with the growth of filamentous bacteria. Work by Chudoba et al. (1973) led to the concept of biological selectors and staged reactor configurations to prevent filamentous growth. Staged activated sludge reactor configurations proposed by Barnard (1974) led to activated sludge process applications for biological removal of nitrogen and phosphorus in addition to carbonaceous BOD removal.

Processes with a sequence of anoxic-aerobic-anoxic-aerobic zones for biological nitrogen removal and the addition of an anaerobic contact zone to promote enhanced biological phosphorus removal are shown on Figs. 7-3(a) and (b). Another major change in activated sludge process design occurred in the late 1990s when membrane materials and manufacturing technology had advanced sufficiently so that membranes could be used in activated sludge for effluent liquid-solids separation instead of clarifiers. A membrane bioreactor (MBR) system is depicted in Fig. 7-3(c). The membrane separation unit is immersed in an aerated activated sludge compartment and a sufficient head is provided by gravity or pumping so that the effluent or permeate can be discharged essentially free of suspended solids. As will be shown in Chap. 8, the MBR process can also be incorporated with biological nutrient removal designs.

Figure 7–3

Progression of activated sludge processes: (a) anoxic-aerobic activated sludge for nitrogen removal, (b) anaerobic-anoxic-aerobic-anoxic-aerobic process for nitrogen and phosphorus removal, (c) anoxic-aerobic treatment in membrane bioreactor process with nitrogen removal, and (d) integrated fixed film activated sludge process with nitrogen removal.



Attached Growth Processes. In attached growth processes, the microorganisms responsible for the conversion of organic material or nutrients are attached to an inert packing material. The organic material and nutrients are removed from the wastewater flowing past the attached growth, also known as a biofilm. Packing materials used in attached growth processes include rock, gravel, slag, sand, redwood, and a wide range of plastic and other synthetic materials. Attached growth processes can also be operated as aerobic or anaerobic processes. The packing can be submerged completely in liquid or partially submerged, with air or gas space above the biofilm liquid layer.

The most common aerobic attached growth process used is the trickling filter in which wastewater is distributed over the top area of a vessel containing nonsubmerged packing material (see Fig. 7–4). Historically, rock was used most commonly as the packing material for trickling filters [see Fig. 7–4(a)], with typical depths ranging from 1.25 to 2 m (4 to 6 ft). Most modern trickling filters vary in height from 5 to 10 m (16 to 33 ft) and are filled with a plastic packing material for biofilm attachment [see Fig. 7–4(b)]. The plastic packing material is designed such that about 90 to 95 percent of the volume in the tower consists of void space. Air circulation in the void space, by either natural draft or blowers,

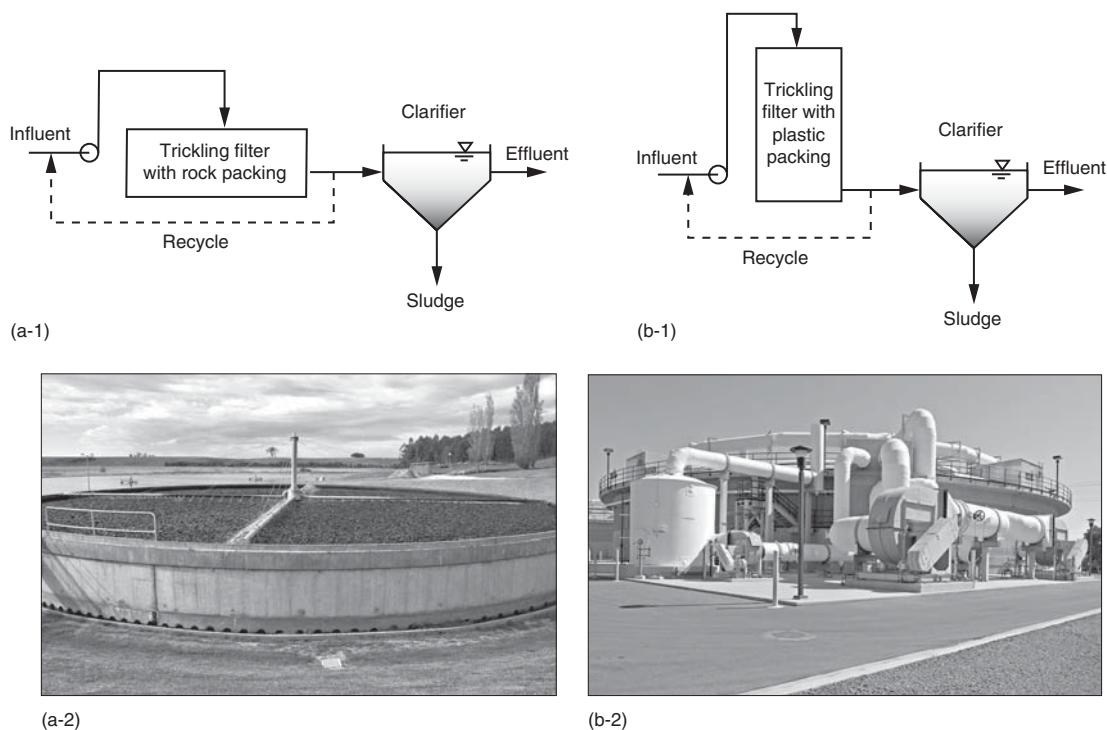


Figure 7-4

Attached growth biological treatment process: (a-1) schematic and (a-2) view of trickling filter with rock packing; and (b-1) schematic and (b-2) view of covered tower trickling filter with plastic packing. The air injection and odor control facilities are shown on the foreground. The tower filter is 10 m high and 50 m in diameter.

provides oxygen for the microorganisms growing as an attached biofilm. Influent wastewater is distributed over the packing and flows as a nonuniform liquid film over the attached biofilm. Excess biomass sloughs from the attached growth periodically and some form of liquid/solids separation is needed to provide an effluent with an acceptable suspended solids concentration. The solids are collected at the bottom of the clarifier and removed for waste-sludge processing.

The popularity of trickling filters had declined with the increasing applications for biological nutrient removal. Advances in the 1990s involved the combination of fixed film and activated sludge treatment, termed an integrated fixed film activated sludge process (IFAS) [see Fig. 7-3(d)]. Other fixed film processes that save space and/or can be used with biological nutrient removal have received more attention and are presented in Chap. 9.

7-2 COMPOSITION AND CLASSIFICATION OF MICROORGANISMS

Biological processes for wastewater treatment consist of mixed microbial communities which may include bacteria, archaea, protozoa, fungi, rotifers, and possibly algae. The basic characteristics and important roles of these organisms have been described in Chap. 2. In some cases, biological treatment goals can only be accomplished by the presence of a specific

within the cell during metabolism (endogenous), or it may be obtained from outside the cell (i.e., dissolved oxygen) (exogenous). Organisms that generate energy by enzyme-mediated electron transport to an external electron acceptor are said to have a *respiratory metabolism*. The use of an internal electron acceptor is termed *fermentative metabolism* and is a less efficient energy-yielding process than respiration. Heterotrophic organisms that are strictly fermentative are characterized by lower growth rates and cell yields than respiratory heterotrophs.

Utilization of Oxygen. When oxygen is used for the electron acceptor the reaction is termed *aerobic*, and reactions involving other electron acceptors are considered anaerobic. The term *anoxic* is used to distinguish the use of nitrite or nitrate for electron acceptors from the others under anaerobic conditions. Under anoxic conditions nitrite or nitrate reduction to gaseous nitrogen occurs, and this reaction is also referred to as biological denitrification. Organisms that can only meet their energy needs with oxygen are called *obligate aerobic* microorganisms. Some bacteria can use oxygen or nitrate/nitrite as electron acceptors when oxygen is not available. These bacteria are called *facultative aerobic* bacteria.

Energy for Fermentation. Organisms that generate energy by fermentation and that can exist only in an environment that is devoid of oxygen are *obligate anaerobes*. *Facultative anaerobes* have the ability to grow in either the presence or absence of molecular oxygen and fall into two subgroups, based on their metabolic abilities. True facultative anaerobes can shift from fermentative to aerobic respiratory metabolism, depending upon the presence or absence of molecular oxygen. *Aerotolerant anaerobes* have a strictly fermentative metabolism but are relatively insensitive to the presence of molecular oxygen.

Nutrient and Growth Factor Requirements

Nutrients, rather than carbon or energy sources, may at times be the limiting material for microbial cell synthesis and growth. The principal inorganic nutrients needed by microorganisms are N, S, P, K, Mg, Ca, Fe, Na, and Cl. Minor nutrients of importance include Zn, Mn, Mo, Se, Co, Cu, and Ni (Madigan et al., 2012). Required organic nutrients, known as growth factors, are compounds needed by an organism as precursors or constituents of organic cell material, which cannot be synthesized from other carbon sources. Although growth factor requirements differ from one organism to another, the major growth factors fall into the following three classes: (1) amino acids, (2) nitrogen bases (i.e., purines and pyrimidines), and (3) vitamins.

For municipal wastewater treatment sufficient nutrients are generally present, but for industrial wastewaters nutrients may need to be added to the biological treatment processes. The lack of sufficient nitrogen and phosphorus is common especially in the treatment of food-processing wastewaters or wastewaters high in organic content. Using the formula $C_{12}H_{87}O_{23}N_{12}P$ (given previously) for the composition of cell biomass, about 12.2 g of nitrogen and 2.3 g of phosphorus are needed per 100 g of cell biomass.

7-4 BACTERIAL GROWTH, ENERGETICS, AND DECAY

In the description of microbial metabolism it was noted that as microorganisms consume substrate and carry out oxidation-reduction reactions, growth occurs by the production of additional cells. Thus, in wastewater treatment applications biomass is produced continuously as the substrate in the wastewater is consumed and biodegraded. Topics considered in this section include (1) bacterial reproduction, (2) bacterial growth patterns in a batch

reactor, (3) bacterial growth and biomass yield, (4) methods used to measure biomass growth, (5) estimating cell yield and oxygen requirements from stoichiometry, (6) estimating cell yield from bioenergetics, and (7) observed versus synthesis yield. The material presented in this section will serve as a basis for the sections that follow and the material presented in Chaps. 8, 9, and 10 on the various treatment processes.

Bacterial Reproduction

Bacteria can reproduce, as noted in Chap. 2, by binary fission, by asexual mode, or by budding. Generally, bacteria reproduce by binary fission, in which the original cell becomes two new organisms. The time required for each division, which is termed the generation time, can vary from days to less than 20 min. For example, if the generation time is 30 min, one bacterium would yield 16,777,216 (i.e., 2^{24}) bacteria after a period of 12 h. Assuming spherical-shaped bacteria with a $1\text{ }\mu\text{m}$ diameter and specific gravity of 1.0, the weight of 1 cell is approximately $5.0 \times 10^{-13}\text{ g}$. In 12 h the bacteria mass would be about $8.4 \times 10^{-6}\text{ g}$ or $8.4\text{ }\mu\text{g}$; thus the number of cells is quite large compared to the mass. This rapid change in biomass with time is a hypothetical example. In biological treatment systems bacteria would not continue to divide indefinitely because of environmental limitations, such as substrate and nutrient availability.

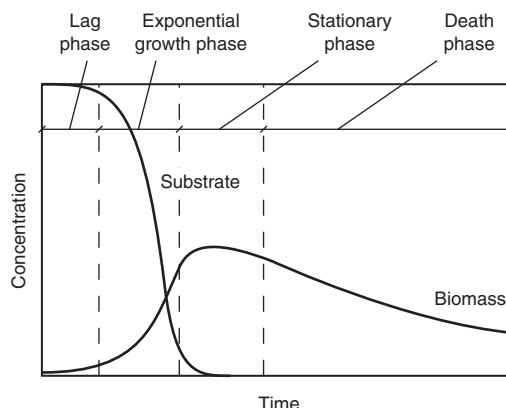
Bacterial Growth Patterns in a Batch Reactor

Bacterial growth in a batch reactor (see Sec. 1–7) is characterized by identifiable phases as illustrated on Fig. 7–10. The curves shown on Fig. 7–10 represent what occurs in a batch reactor in which, at time zero, substrate and nutrients are present in excess and only a very small population of biomass exists. As substrate is consumed, four distinct growth phases develop sequentially.

1. The lag phase. Upon addition of the biomass, the lag phase represents the time required for the organisms to acclimate to their new environment before significant cell division and biomass production occur. During the lag phase enzyme induction may be occurring and/or the cells may be acclimating to changes in salinity, pH, or temperature. The apparent extent of the lag phase may also be affected by the ability to measure the low biomass concentration during the initial batch phase.
2. The exponential-growth phase. During the exponential-growth phase, bacterial cells are multiplying at their maximum rate, as there is no limitation due to substrate

Figure 7–10

Batch process biomass growth phases with changes in substrate and biomass versus time.



or nutrients. The biomass growth curve increases exponentially during this period. With unlimited substrate and nutrients the only factor that affects the rate of exponential growth is temperature.

3. The stationary phase. During this phase, the biomass concentration remains relatively constant with time. In this phase, bacterial growth is no longer exponential, and the amount of growth is offset by the death of cells.
4. The death phase. In the death phase, the substrate has been depleted so that no growth is occurring, and the change in biomass concentration is due to cell death. An exponential decline in the biomass concentration is often observed as an approximate constant fraction of the biomass remaining that is lost each day.

Bacterial Growth and Biomass Yield

In biological treatment processes, cell growth occurs concurrent with the oxidation of organic or inorganic compounds, as described above. The ratio of the amount of biomass produced to the amount of substrate consumed (g biomass/g substrate) is defined as the *biomass synthesis yield* and typically is defined relative to the electron donor used.

$$\text{Biomass synthesis yield, } Y = \frac{\text{g biomass produced}}{\text{g substrate consumed}} \quad (7-2)$$

For example, for aerobic heterotrophic reactions with organic substrates, the yield is expressed as g biomass/g organic substrate; for nitrification the yield is expressed as g biomass/g NH_4^+ -N oxidized; and for the anaerobic degradation of volatile fatty acids (VFAs) to produce methane, the yield is expressed as g biomass/g VFAs used. Where specific compounds are measured and known, such as ammonia, the yield is quantified relative to the amount of compound used. For aerobic or anaerobic treatment of municipal and industrial wastewater containing a large number of organic compounds, the yield is based on a measurable parameter reflecting the overall organic compound consumption, such as COD or BOD. Thus, the yield would be g biomass/g COD removed or g biomass/g BOD removed.

Measuring Biomass Growth

Because biomass is mostly organic material, an increase in biomass can be measured by volatile suspended solids (VSS) or particulate COD (total COD minus soluble COD). Other more direct parameters that are used to indicate biomass growth are protein content, DNA, and adenosine triphosphate (ATP), a cellular nucleotide compound involved in energy transfer. Of these growth measurement parameters, VSS is the parameter used most commonly to follow biomass growth in full-scale biological wastewater treatment systems because its measurement is simple, and minimal time is required for analysis. It should be noted that the VSS measured includes other particulate organic matter in addition to biomass. Most wastewaters contain some amount of nonbiodegradable VSS and possibly influent VSS that may be degraded slowly in the biological reactor. These solids are included with biomass in the VSS measurement. Nevertheless the VSS measurement is used as an apparent indicator of biomass production and also provides a useful measurement of reactor solids in general.

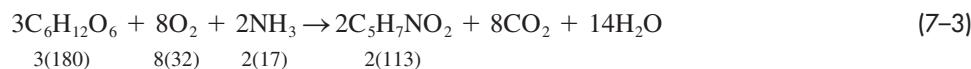
For laboratory research on biological treatment processes, growth parameters that can be related to true microbial mass are often used. Of these, protein is the most popular growth parameter due to the relative ease of measurement and the fact that about 50 percent of biomass dry weight is protein. Both ATP and DNA have also been used, especially

where the reactor solids contain proteins and other solids that are not associated with biomass. Where very low biomass concentrations are involved, turbidity measurements may be used to provide a rapid and simple means of observing cell growth. Bacterial cell counts have also been used to enumerate the biomass population. A portion of a diluted sample is applied to an agar growth plate, and after incubation, the number of colonies formed are counted and used to determine the number of bacterial cells in the culture. It should be noted, however, that not all bacteria are culturable.

Estimating Biomass Yield and Oxygen Requirements from Stoichiometry

As given by Eq. (7–1), a definite stoichiometric relationship exists between the substrate removed, the amount of oxygen consumed during aerobic heterotrophic biodegradation, and the observed biomass yield. The most common approach used to define the fate of the substrate is to prepare a COD mass balance. The COD is used because the substrate concentration in the wastewater can be defined in terms of its oxygen equivalence, which can be accounted for by being conserved in the biomass or oxidized.

Biomass Yield. In general, the exact stoichiometry involved in the biological oxidation of a mixture of wastewater compounds is never known. However, for the purpose of illustration, it is assumed that organic matter can be represented as $C_6H_{12}O_6$ (glucose) and new cells can be represented as $C_5H_7NO_2$ (Hoover and Porges, 1952). Thus, neglecting nutrients other than nitrogen, Eq. (7–1) can be written as



As given by the above equation, the substrate used (glucose in this case) is divided between that found in new cells and that oxidized to produce CO_2 and H_2O . The yield based on the glucose consumed can be obtained as follows:

$$Y = \frac{\Delta(C_5H_7NO_2)}{\Delta(C_6H_{12}O_6)} = \frac{2(113 \text{ g/mole})}{3(180 \text{ g/mole})}$$

$$= 0.42 \text{ g cells/g glucose used}$$

In practice, COD and VSS are used to represent the organic matter and the new cells, respectively. To express the yield on a COD basis, the COD of glucose must be determined. The COD of glucose can be determined by writing a balanced stoichiometric reaction for the oxidation of glucose to carbon dioxide as follows:



The COD of glucose is

$$COD = \frac{\Delta(O_2)}{\Delta(C_6H_{12}O_6)} = \frac{6(32 \text{ g/mole})}{(180 \text{ g/mole})} = 1.07 \text{ g O}_2/\text{g glucose}$$

The theoretical yield expressed in terms of COD, accounting for the portion of the substrate converted to new cells, is

$$Y = \frac{\Delta(C_5H_7NO_2)}{\Delta(C_6H_{12}O_6 \text{ as COD})} = \frac{2(113 \text{ g/mole})}{3(180 \text{ g/mole})(1.07 \text{ g COD/g glucose})}$$

$$= 0.39 \text{ g cells/g COD used or } 0.39 \text{ g VSS/g COD used}$$

It should be noted that the actual observed yield in a biological treatment process will be less than the value given above, because a portion of the substrate incorporated into the cell mass will be later oxidized with time by the bacteria to obtain energy for cell maintenance.

Oxygen Requirements. The quantity of oxygen utilized can be accounted for by considering (1) the oxygen used for substrate oxidation to CO_2 and H_2O , (2) the COD of the biomass, and (3) the COD of any substrate not degraded. For the formula $\text{C}_5\text{H}_7\text{NO}_2$, the oxygen equivalent of the biomass (typically measured as VSS) is approximately 1.42 g COD/g biomass VSS, as given below.



The COD of cell tissue is

$$\frac{\Delta(\text{O}_2)}{\Delta(\text{C}_5\text{H}_7\text{NO}_2)} = \frac{5(32 \text{ g/mole})}{(113 \text{ g/mole})} = 1.42 \text{ g O}_2/\text{g cells} = 1.42 \text{ g COD/g VSS}$$

Based on the above relationships, the oxygen consumed per unit of COD utilized for the reaction given by Eq. (7-3) can be determined from a mass balance on COD.

The COD removed is oxidized or accounted for in cell growth.

$$\text{COD}_r = \text{COD}_{\text{cells}} + \text{COD}_{\text{ox}} \quad (7-6)$$

where COD_r = COD utilized, g COD/d

$\text{COD}_{\text{cells}}$ = COD incorporated into cell synthesis, g COD/d

COD_{ox} = COD oxidized, g COD/d

The COD of oxidized substrate is equal to the oxygen consumed; thus

$$\text{Oxygen consumed} = \text{COD utilized} - \text{COD cells} \quad (7-7)$$

$$\begin{aligned} &= \left(\frac{1.07 \text{ g O}_2}{\text{g glucose}} \right) \left(3 \text{ mole} \times \frac{180 \text{ g glucose}}{\text{mole}} \right) \\ &\quad - \left(\frac{1.42 \text{ g O}_2}{\text{g cells}} \right) \left(2 \text{ mole} \times \frac{113 \text{ g cells}}{\text{mole}} \right) \\ &= 577.8 \text{ g O}_2 - 320.9 \text{ g O}_2 = 256.9 \text{ g O}_2 \end{aligned}$$

Thus, the oxygen consumed per unit of COD used is

$$\begin{aligned} \frac{\text{Oxygen consumed}}{\text{Glucose as COD}} &= \frac{256.9 \text{ g O}_2}{3 \text{ mole}(1.07 \text{ g COD/g glucose})(180 \text{ g glucose/mole})} \\ &= 0.44 \text{ g O}_2/\text{g COD used} \end{aligned}$$

The amount of oxygen required based on the COD balance, as given above, is in agreement with the oxygen use based on the stoichiometry as defined by Eq. (7-3) in which 8 moles of oxygen are required for 3 moles of glucose.

$$\begin{aligned} \frac{\text{Oxygen used}}{\text{Glucose as COD}} &= \frac{8(32 \text{ g O}_2/\text{mole})}{3(180 \text{ g/mole})(1.07 \text{ g COD/g glucose})} \\ &= 0.44 \text{ g O}_2/\text{g COD used} \end{aligned}$$

Note that the cell yield as 0.39 g VSS/g COD used is equal to 0.56 g cells as COD/g COD used, and the sum of the g oxygen used for energy yielding oxidation/g COD used (0.44) plus the g cells as COD/g COD used = 1.0 g COD/g COD used. The COD is accounted for by the oxygen used and by the COD of biomass produced.

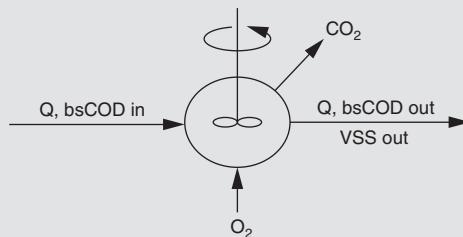
The relationship of the observed biomass yield to the oxygen consumed for substrate oxidation by aerobic heterotrophic biomass based on typical measurements made at wastewater-treatment plants is illustrated in Example 7–1.

EXAMPLE 7–1

Observed Biomass Yield and Oxygen Consumption

An aerobic complete-mix biological treatment process without recycle, as shown below, receives wastewater with a biodegradable soluble COD (bsCOD) concentration of 500 g/m³. The flowrate is 1000 m³/d and the reactor effluent bsCOD and VSS concentrations are 10 and 200 g/m³, respectively. Based on these data, determine the:

1. Observed yield in g VSS/g COD removed?
2. Amount of oxygen used in g O₂/g COD removed and in g/d?



Solution

1. Determine the observed yield. Assume the following general reaction is applicable.



- a. The g VSS/d produced is:

$$\text{g VSS/d} = 200 \text{ g/m}^3 (1000 \text{ m}^3/\text{d}) = 200,000 \text{ g VSS/d}$$

- b. The g bsCOD/d removed is:

$$\begin{aligned} \text{g COD/d} &= (500 - 10) \text{ g COD/m}^3 (1000 \text{ m}^3/\text{d}) \\ &= 490,000 \text{ g COD/d} \end{aligned}$$

- c. The observed yield is:

$$Y_{\text{obs}} = \frac{(200,000 \text{ g VSS/d})}{(490,000 \text{ g COD/d})} = 0.41 \text{ g VSS/g COD removed}$$

2. Determine the amount of oxygen used per g bsCOD removed.

- a. Prepare a steady-state COD mass balance around the reactor

accumulation = inflow – outflow + conversion

$$0 = \text{COD}_{\text{in}} - \text{COD}_{\text{out}} - \text{oxygen used (expressed as COD)}$$

$$\text{Oxygen used} = \text{COD}_{\text{in}} - \text{COD}_{\text{out}}$$

$$\text{COD}_{\text{in}} = 500 \text{ g COD/m}^3 (1000 \text{ m}^3/\text{d}) = 500,000 \text{ g COD/d}$$

$$\text{COD}_{\text{out}} = \text{bsCOD}_{\text{out}} + \text{biomass COD}_{\text{out}}$$

$$\text{bsCOD}_{\text{out}} = 10 \text{ g/m}^3 (1000 \text{ m}^3/\text{d}) = 10,000 \text{ g COD/d}$$

$$\begin{aligned}\text{Biomass COD}_{\text{out}} &= 200,000 \text{ g VSS/d} (1.42 \text{ g COD/g VSS}) \\ &= 284,000 \text{ g COD/d}\end{aligned}$$

$$\text{Total COD}_{\text{out}} = 10,000 \text{ g/d} + 284,000 \text{ g/d} = 294,000 \text{ g COD/d}$$

b. The oxygen used is

$$\begin{aligned}\text{Oxygen used} &= 500,000 \text{ g COD/d} - 294,000 \text{ g COD/d} \\ &= 206,000 \text{ g COD/d} = 206,000 \text{ g O}_2/\text{d}\end{aligned}$$

c. Amount of oxygen used per unit COD removed is

$$\text{Oxygen/COD} = (206,000 \text{ g/d})/(490,000 \text{ g/d}) = 0.42 \text{ g O}_2/\text{g COD}$$

Comment The general COD balance that accounts for the cell yield fraction of COD oxidized is
 $\text{g COD cells} + \text{g COD oxidized} = \text{g COD removed}$
 $(0.41 \text{ g VSS/g COD})(1.42 \text{ g O}_2/\text{g VSS}) + 0.42 \text{ g O}_2/\text{g COD} = 1.0 \text{ g O}_2/\text{g COD}$

Estimating Biomass Yield from Bioenergetics

Most cell yield values are based on measurements from laboratory reactors, pilot plants, or full-scale systems. However, an approach that has been developed to estimate cell yield, based on bioenergetics, involves the application of thermodynamic principles to biological reactions. An introduction to bioenergetics and the application of bioenergetics to estimate the biomass yield for different types of biological reactions is provided in this section (McCarty 1971, 1975).

Gibbs Free Energy. Chemical reactions, which involve changes in energy, can be described thermodynamically by a change in the free energy G° , known as the *Gibbs free energy*. The change in energy due to the reaction is termed ΔG° . The superscript is used to indicate that the free energy values were obtained at standard conditions of pH = 7.0 and 25°C. The net Gibbs free energy, positive or negative, can be evaluated for reactants and products based on standard free energy values available for the half reactions. Half reactions describe the transfer of 1 mole of electron in oxidation-reduction and synthesis reactions. Free energy changes for various half reactions are listed in Table 7–6. Reactions that result in a negative change in the free energy are those that release energy and are called *exergonic* reactions. These reactions will proceed spontaneously in the direction shown. However, if the free energy change results in a positive value, the reaction is termed *endergonic*, and such a reaction will not occur spontaneously. Reactions with positive free energy values require energy to proceed in the direction indicated.

The basis of the analysis of free energy changes for reactions is that in oxidation-reduction reactions one compound loses electrons (electron donor) and the other compound gains electrons (electron acceptor) (see discussion in Sec. 6–6 in Chap. 6). The moles of compound used as an electron acceptor and electron donor per mole of electron (e^-) transferred from the half reactions along with the change in free energy are used to develop energy balances in bioenergetic analyses. Determination of the free energy change resulting from the oxidation of hydrogen by oxygen is illustrated in Example 7–2.

decrease are *decay*, *endogenous decay*, or *endogenous respiration*. A number of factors contribute to the biomass loss, including cell maintenance energy needs, cell lysis due to death or stress from environmental factors, and predation (Hao et al., 2010). Examples of maintenance energy needs are energy for resynthesis of essential cell components and for controlling osmotic pressure. Energy may come from oxidation of cellular organic material or substrates from the bulk liquid. If substrate is not available, the cell mass may decrease and cellular activity can approach a resting state. Cell death and lysis can be due to virus, environmental stress (pH, temperature and toxins), or to program cell death related to cell age (Rice and Bayle 2008).

Cell lysis results in the release of cellular substrates, which are mainly biodegradable proteins and polysaccharides. In addition, cell debris, which remains after cell lysis, has generally been assumed as inert volatile solids that accumulate in the bioreactor, but more recently cell debris has been found to have a very slow biodegradation rate (Ramdan et al., 2012). Protozoan and rotifers consume bacteria that are not well captured in activated sludge floc. Predation losses by protozoan and rotifers are greater for systems with free bacteria or more dispersed growth conditions. The parameter, specific endogenous decay, is commonly used to account for all of these biomass loss mechanisms, which may be occurring to a varying degree, depending on cell age and operating conditions. The specific endogenous decay coefficient is used to describe a specific rate of biomass loss, which can be in the range from 0.08 to 0.20 g biomass loss/g biomass present.

Observed versus Synthesis Yield

In the evaluation and modeling of biological treatment systems a distinction is made between the observed yield and the synthesis yield (or true yield). The observed biomass yield is based on the actual measurements of the net biomass production and substrate consumption and is actually less than the synthesis yield, because of cell loss by biomass decay (Sec. 7–6) concurrent with cell growth. In full-scale wastewater treatment processes the term solids production (or solids yield) is also used to describe the amount of VSS generated in the treatment process. The term is different from the synthesis biomass yield values because it contains other organic solids from the wastewater that are measured as VSS and have not been biologically degraded.

The synthesis yield is the amount of biomass produced immediately upon consumption of the growth substrate or oxidation of the electron donor in the case of autotrophic bacteria. The synthesis yield is seldom measured directly and is often interpreted from evaluating biomass production data for reactors operating under different conditions. Synthesis yield values for bacterial growth are affected by the energy that can be derived from the oxidation-reduction reaction, by the growth characteristics of the carbon source, by the nitrogen source, and by environmental factors such as temperature, pH, and osmotic pressure. As illustrated in this section, the synthesis yield can be estimated if the stoichiometry or the amount of energy produced in the oxidation-reduction reaction is known.

7–5 MICROBIAL GROWTH KINETICS

The performance of biological processes used for wastewater treatment depends on the dynamics of substrate utilization and microbial growth. Effective design and operation of such systems requires an understanding of the biological reactions occurring and an understanding of the basic principles governing the growth of microorganisms. Further, the need to understand all of the environmental conditions that affect the substrate utilization and microbial growth rate cannot be overemphasized, and it may be necessary to control such conditions as pH, dissolved oxygen (DO), and nutrients to provide effective treatment.

The purpose of this section is to present an introduction to microbial growth kinetics. The topics considered in this section include (1) microbial growth kinetics terminology, (2) rate of utilization of soluble substrate, (3) other rate expressions for the utilization of soluble substrate, (4) rate of soluble substrate production from biodegradable particulate organic matter, (5) the rate of biomass growth with soluble substrates, (6) kinetic coefficients for substrate utilization and biomass growth, (7) oxygen utilization rates, (8) effects of temperature, (9) total volatile suspended solids and active biomass, (10) rate of biomass decay and soluble substrate production, and (11) net biomass growth and observed yield.

Microbial Growth Kinetics Terminology

The kinetics of microbial growth govern the oxidation (i.e., utilization) of substrate and the production of biomass, which contributes to the total suspended solids concentration in a biological reactor. Common terms used to describe the transformations that occur in substrate oxidation and biomass growth as well as in biological treatment processes are defined in the Working Terminology. Because municipal and industrial wastewaters contain numerous substrates, the concentration of organic compounds is defined, most commonly, by the *biodegradable COD* (bCOD) or UBOD, both of which are comprised of soluble (dissolved), colloidal, and particulate biodegradable components. Both bCOD and UBOD represent measurable quantities that apply to all of the compounds. In the formulation of kinetic expressions in this chapter *biodegradable soluble COD* (bsCOD) will be used to quantify the fate of biodegradable organic compounds because it is related to the stoichiometry of substrate oxidized or used in cell growth [see Eq. (7-7)]. Some of the biodegradable COD in wastewater is in particulate or colloidal form, which must be hydrolyzed to bsCOD first before biological consumption. Kinetic expressions are also used to describe the rate of bsCOD production from particulate and colloidal material.

The biomass solids in a laboratory bioreactor fed only bsCOD are commonly measured as *total suspended solids* (TSS) and *volatile suspended solids* (VSS). These measurements are not equal to the active biomass, as the solids will contain cell debris material from bacteria decay.

In activated sludge systems used to treat municipal and industrial wastewater, the mixture of solids resulting from combining recycled sludge with influent wastewater in the bioreactor is termed *mixed liquor suspended solids* (MLSS) and *mixed liquor volatile suspended solids* (MLVSS). The solids are comprised of biomass, *nonbiodegradable volatile suspended solids* (nbVSS), and *inert inorganic total suspended solids* (iTSS). The nbVSS is derived from the influent wastewater and is also produced as cell debris from bacteria decay. The iTSS originates in the influent wastewater. Additional wastewater characterization terminology is considered in Sec. 8-2 in Chap. 8.

Rate of Utilization of Soluble Substrates

In the introduction to this chapter, it was noted that one of the principal concerns in wastewater treatment is the removal of substrate. Stated another way, the goal in biological wastewater treatment is, in most cases, to reduce the electron donor (i.e., organic compounds in aerobic oxidation) to minimal levels. For heterotrophic bacteria the electron donors are the organic substances being degraded; for autotrophic nitrifying bacteria it is ammonia or nitrite or other reduced inorganic compounds. The substrate utilization rate for soluble substrates in biological systems can be modeled with Eq. (7-12), in which the rate of substrate utilization increases as the reactor substrate concentration increases for a given biomass concentration.

$$r_{su} = \frac{kXS}{K_s + S} \quad (7-12)$$

where r_{su} = substrate utilization rate per unit of reactor volume, g/m³·d
 k = maximum specific substrate utilization rate, g substrate/g microorganisms·d
 X = biomass (microorganism) concentration, g/m³
 S = growth-limiting substrate concentration in solution, g/m³
 K_s = half-velocity constant, substrate concentration at one-half the maximum specific substrate utilization rate, g/m³

Equation (7-12) will be recognized as a saturation-type equation, as described previously in Chap. 1. Though Eq. (7-12) is an empirical model, based on using coefficients derived from biological reactor data, it is often referred to as the Michaelis-Menten equation (Bailey and Ollis, 1986), which was formulated from an enzymatic-substrate model. A plot of r_{su} versus the substrate concentration is shown on Fig. 7-11. As shown on Fig. 7-11, the maximum substrate utilization rate occurs at high substrate concentrations. Further, as the substrate concentration decreases below some critical value, the value of r_{su} also decreases almost linearly. In practice, biological treatment systems are designed to produce an effluent with extremely low substrate concentrations.

Equation (7-12) is also of the form proposed by Monod for the specific growth rate of bacteria in which the limiting substrate is available to the microorganisms in a dissolved form (Monod, 1942, 1949).

$$r_g = \frac{\mu_m X S}{(K_s + S)} \quad (7-13)$$

where r_g = bacteria growth rate from substrate utilization, g/m³·d

μ_m = maximum specific bacteria growth rate, g biomass/g biomass·d

As bacteria consume the substrate (electron donor), the energy produced from the substrate oxidation is used to process carbon and nutrients to produce new biomass. The new growth is directly proportional to the substrate used and is termed the true yield or synthesis yield coefficient as g biomass produced/g substrate used. The bacteria growth rate can thus be equated to the substrate utilization rate as follows:

$$r_g = Y r_{su} \quad (7-14)$$

$$\text{and } r_{su} = \frac{\mu_m X S}{Y(K_s + S)} \quad (7-15)$$

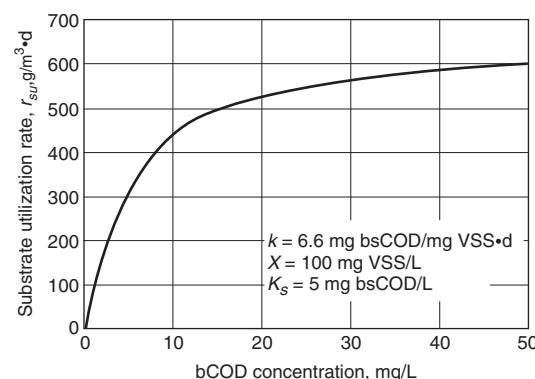
where Y = synthesis yield coefficient, g biomass/g substrate used

When the substrate is being used at its maximum rate, the bacteria are also growing at their maximum rate. The maximum specific growth rate of the bacteria is thus related to the maximum specific substrate utilization rate as follows.

$$\mu_m = k Y \quad (7-16)$$

Figure 7-11

Rate of change of substrate utilization versus biodegradable soluble COD concentration based on the saturation-type model [see Eq. (7-12)].



Other Rate Expressions for Soluble Substrate Utilization

In reviewing kinetic expressions used to describe substrate utilization and biomass growth rates, it is important to remember that the expressions used to model biological processes are all empirical, based on coefficient values determined experimentally. Besides the substrate limited relationship presented above, other expressions that have been used to describe soluble substrate utilization rates include the following:

$$r_{su} = kS \quad (7-17)$$

$$r_{su} = kXS \quad (7-18)$$

$$r_{su} = kX \frac{S}{S_o} \quad (7-19)$$

The particular rate expression used to define kinetics of substrate utilization depends mainly on the experimental data available to fit the kinetic equations and the application of the kinetic model. In many cases, the pseudo first-order model as given by Eq. (7-18) is satisfactory for describing substrate utilization rates when the biological treatment process is operated at relatively low substrate concentrations. Fundamental in the use of any rate expression is its application in a mass-balance analysis as discussed in the following section. Also, with regard to modeling biological treatment processes, kinetic models should not be applied outside of the range of the conditions used to develop model coefficients.

Rate of Soluble Substrate Production from Biodegradable Particulate Organic Matter

The rate expressions for substrate utilization and biomass growth presented thus far are based on the utilization of soluble substrates. In municipal wastewater treatment only about 20 to 50 percent of the degradable organic material enters as soluble compounds, and for some industrial wastewaters the soluble organic material may be a low to moderate fraction of the total degradable organic substrates. Bacteria cannot consume the particulate substrates directly and must employ extracellular enzymes to hydrolyze the particulate organics to soluble substrates. The particulate substrate conversion rate is also a rate-limiting process that is dependent on the particulate substrate and biomass concentrations. A rate expression for particulate substrate conversion is shown as follows (Gujer et al., 1999):

$$r_{X_s} = -\frac{k_h(X_s/X_H)X_H}{(K_X + X_s/X_H)} \quad (7-20)$$

where r_{X_s} = rate of hydrolysis of particulate substrate to conversion to soluble substrate, g/m³·d

k_h = maximum specific hydrolysis rate, g X_s /g X_H ·d

X_s = particulate substrate concentration, g/m³

X_H = heterotrophic biomass concentration, g/m³

K_X = half-velocity hydrolysis coefficient, g/g

The particulate degradation concentration is expressed relative to the biomass concentration, because the particulate hydrolysis is related to the relative contact area between the nonsoluble organic material and the biomass. The effect of particulate organic constituents is considered further in Chap. 8.

Net Biomass Growth Rate

The net biomass growth rate is the biomass growth rate less the endogenous respiration rate. In Sec. 7–4, the biomass growth rate was shown to be proportional to the substrate utilization rate by the synthesis yield coefficient, and biomass decay was shown to be proportional to the biomass present. Thus, the following relationship between the net biomass growth rate and the substrate utilization rate is applicable in both batch and continuous culture systems.

$$r_X = Yr_{su} - bX \quad (7-21)$$

$$r_X = Y \frac{kXS}{K_s + S} - bX \quad (7-22)$$

where r_X = net biomass growth rate per unit reactor volume, g VSS/m³·d

b = specific endogenous decay coefficient, g VSS/g VSS·d

Other terms are as defined above.

If both sides of Eq. (7–22) are divided by the biomass concentration X , the net specific growth rate is defined as follows:

$$\mu_{net} = \frac{r_X}{X} = Y \frac{kS}{K_s + S} - b \quad (7-23)$$

where μ_{net} = net specific biomass growth rate, g VSS/g VSS·d

As shown, the net specific growth rate corresponds to the change in biomass per day relative to the amount of biomass present and is a function of the reactor substrate concentration and the specific endogenous decay coefficient.

As noted in Sec. 7–4, the specific endogenous decay coefficient accounts for the loss in cell mass due to oxidation of internal storage products for energy for cell maintenance, cell death, and predation by organisms higher in the food chain. These factors are usually lumped together under endogenous decay, and it is assumed that the decrease in cell mass caused by them is proportional to the biomass concentration present. In Eq. (7–21), the coefficient b is the *specific endogenous decay* rate coefficient. An alternative approach used to describe the endogenous decay known as a *lysis-regrowth* model is described in Sec. 8–10 in Chap. 8. In biological treatment processes, both the substrate utilization and biomass growth rates are controlled by some limiting substrate, as given by Eqs. (7–12) and (7–22). The growth limiting substrate can be any of the essential requirements for cell growth (i.e., electron donor, electron acceptor, or nutrients), but often it is the electron donor that is limiting, as other requirements are usually available in excess. Thus, when the term substrate is used to describe growth kinetics, it generally refers to the electron donor.

For many substrate utilization modeling applications, it is convenient to use a general equation that includes factors other than the electron donor substrate and the substrate utilization rate. In some cases, low dissolved oxygen concentration or low nutrient concentrations can be of concern. For general model applications it is convenient to describe these effects as a multiple of rate expressions for each important factor affecting the soluble substrate utilization rate. For example, Eq. (7–24), used in the International Water Association ASM2d and ASM3 (Gujer et al., 1999) models, is used to account for possible effects of low dissolved oxygen and ammonia-nitrogen concentrations limiting the bacteria growth rate and thus the substrate utilization rate. At very low ammonia-nitrogen concentrations (less than 0.05 mg/L), the availability of nitrogen for cell synthesis may limit the biomass growth rate.

$$r_{su} = \left[\frac{\mu_{H,\max} S_S}{Y_H(K_s + S_S)} \right] \left(\frac{S_o}{K_o + S_o} \right) \left(\frac{S_{NH}}{K_{NH} + S_{NH}} \right) X_H \quad (7-24)$$

where r_{su} = substrate utilization rate, g/m³·d

$\mu_{H,\max}$ = maximum specific growth rate of heterotrophic bacteria, g VSS/g VSS·d

Y_H = heterotrophic bacteria synthesis yield coefficient, g VSS/g COD used

S_S = bsCOD concentration, g/m³

S_o = dissolved oxygen concentration, g/m³

S_{NH} = ammonia-nitrogen concentration, g/m³

K_S = bsCOD half-velocity constant, g/m³

K_o = dissolved oxygen half-velocity constant, g/m³

K_{NH} = ammonia-nitrogen half-velocity constant, g/m³

X_H = heterotrophic bacteria concentration, g VSS/m³

Kinetic Coefficients for Substrate Utilization and Biomass Growth

The values used for the coefficients k , K_s , Y , and b to predict the rate of substrate utilization and biomass growth can vary as a function of the wastewater source, microbial population, and temperature. Kinetic coefficient values are determined from bench-scale testing or by fitting a model to full-scale plant test results. For modeling substrate removal in municipal and industrial wastewater, the coefficient values represent the net effect of microbial kinetics on the simultaneous degradation of a variety of different wastewater constituents by a variety of microorganisms. Typical values for kinetic coefficients are reported in Table 7-8 for the aerobic oxidation of BOD in domestic wastewater. Additional kinetic coefficient values are given in Chaps. 8, 9, and 10.

Rate of Oxygen Uptake

The rate of oxygen uptake is related stoichiometrically to the organic utilization rate and growth rate (see Sec. 7-4). Thus, the oxygen uptake rate can be defined as

$$r_o = r_{su} - 1.42r_x \quad (7-25)$$

where r_o = oxygen uptake rate, g O₂/m³·d

r_{su} = substrate utilization rate per unit of reactor volume, g bsCOD/m³·d

1.42 = the COD of biomass, g COD/g VSS

r_x = rate of biomass growth, g VSS/m³·d

The factor 1.42 represents the COD of biomass as defined previously by Eq. (7-5).

Table 7-8

Typical kinetic coefficients for the activated sludge process for the removal of BOD from domestic wastewater

Coefficient	Unit	Value ^a	
		Range	Typical
k	g bsCOD/g VSS·d	4–12	6
K_s	mg/L BOD	20–60	30
	mg/L bsCOD	5–30	15
Y	mg VSS/mg BOD	0.4–0.8	0.6
	mg VSS/mg COD	0.4–0.6	0.45
b	g VSS/g VSS·d	0.06–0.15	0.10

^a Values reported are for 20°C.

Effects of Temperature

The temperature dependence of the biological reaction-rate constants is very important in assessing the overall efficiency of a biological treatment process. Temperature not only influences the metabolic activities of the microbial population but also has a profound effect on such factors as gas-transfer rates and the settling characteristics of the biological solids. The effect of temperature on the reaction rate of a biological process is expressed using the same type of relationship developed previously in Chap. 1 [see Eq. (1-44)] and repeated here for ease of reference.

$$k_T = k_{20}\theta^{(T-20)} \quad (1-44)$$

where k_T = reaction-rate coefficient at temperature T , °C

k_{20} = reaction-rate coefficient at 20°C

θ = temperature-activity coefficient

T = temperature, °C

Values for θ in biological systems can vary from 1.02 to 1.25. Temperature correction factors for various kinetic coefficients are given in Chap. 8.

Total Volatile Suspended Solids and Active Biomass

The kinetic expressions used to describe biological kinetics and growth are related to the active biomass concentration X in the treatment reactor. In reality the VSS in a reactor consists of more than active biomass, and the fraction of active biomass can vary depending on the wastewater characteristics and operating conditions. The other components that contribute to the VSS concentration are cell debris, resulting from endogenous decay, and non-biodegradable VSS (nbVSS) in the influent wastewater fed to the biological reactor.

During cell death, cell lysis occurs with the release of cellular materials into the liquid for consumption by other bacteria. A portion of the cell mass (cell wall) is not dissolved and remains as nonbiodegradable particulate matter in the system. The remaining nonbiodegradable material is referred to as cell debris and represents about 10 to 15 percent of the original cell weight. Cell debris is also measured as VSS and contributes to the total VSS concentration measured in the reactor mixed liquor. The rate of production of cell debris is directly proportional to the endogenous decay rate.

$$r_{x,i} = f_d(b)X \quad (7-26)$$

where $r_{x,i}$ = rate of cell debris production, g VSS/m³•d

f_d = fraction of biomass that remains as cell debris, 0.10 – 0.15 g VSS/g biomass
VSS depleted by decay

Other terms as defined previously.

The inert VSS concentration resulting from cell debris is typically a relatively small fraction of the VSS in a bioreactor used to treat municipal and some industrial wastewaters. As noted above, a variable amount of MLVSS that is not biomass originates from the nbVSS in the influent wastewater. For typical untreated municipal wastewaters the nbVSS concentration may be in the range from 60 to 100 mg/L, and following primary treatment may range from 10 to 40 mg/L.

Total Volatile Suspended Solids. The VSS production rate in the aeration tank can be defined as the sum of the biomass production rate given by Eq. (7-21), the nbVSS production given by Eq. (7-26), and the nbVSS in the influent wastewater:

$$r_{X_r,VSS} = Yr_{su} - bX + f_d(b)X + QX_{o,i}/V \quad (7-27)$$

net biomass nbVSS nbVSS
 VSS from from cells in influent
 soluble bCOD

where $r_{X_r,VSS}$ = total VSS production rate, g/m³·d

Q = influent flowrate, m³/d

$X_{o,i}$ = influent nbVSS concentration, g/m³

V = volume of reactor, m³

Other terms are as defined previously.

Active Biomass. From Eq. (7-27), the fraction of active biomass in the mixed liquor VSS (MLVSS) is the ratio of the net active biomass production rate, r_X in Eq. (7-21) divided by the total MLVSS production:

$$F_{X,act} = (Yr_{su} - bX)/r_{X_r,VSS} \quad (7-28)$$

where $F_{X,act}$ = active fraction of biomass in MLVSS, g VSS/g VSS

Net Biomass Yield and Observed Yield

The term true yield was defined in Sec. 7-4 as the amount of biomass produced during cell synthesis relative to the amount of substrate degraded. In the design and analysis of biological treatment processes, two other yield terms are important: (1) the net biomass yield and (2) the observed solids yield. The first is used as an estimate of the amount of active microorganisms in the system, and the second as the amount of sludge production.

Net Biomass Yield. The net biomass yield is the ratio of the net biomass growth rate in Eq. (7-21), and the substrate utilization rate:

$$Y_{bio} = r_x/r_{su} \quad (7-29)$$

where Y_{bio} = net biomass yield, g biomass/g substrate used

Observed Yield. The observed yield accounts for the actual solids production that would be measured for the system and is shown as follows:

$$Y_{obs} = r_{X_r,VSS}/r_{su} \quad (7-30)$$

where Y_{obs} = observed yield, g VSS produced/g substrate removed

A determination of the net biomass is considered in Example 7-5.

EXAMPLE 7-5 Determine Biomass and Solids Yields An aerobic complete-mix treatment process is used to treat an industrial wastewater. The amount of bsCOD in the influent wastewater is 300 g/m³ and the influent nbVSS concentration is 50 g/m³. The influent flowrate is 1000 m³/d, the aerobic tank biomass concentration is 2000 g/m³, the reactor bsCOD concentration is 2.4 g/m³, and the reactor volume is 335 m³. If the cell debris fraction f_d is 0.10, determine the net biomass yield, the observed solids yield, and the biomass fraction in the MLVSS. Use the kinetic coefficients given in Table 7-8.

Solution

1. Determine the net biomass yield using Eq. (7–29).

$$Y_{\text{bio}} = r_X/r_{su}$$

- a. Solve for r_{su} using Eq. (7–12) and the information given in Table 7–8

$$\begin{aligned} r_{su} &= \frac{kXS}{K_s + S} \\ &= -\frac{(6/d)(2000 \text{ g/m}^3)(2.4 \text{ g bsCOD/m}^3)}{(15 + 2.4) \text{ g/m}^3} \\ &= 1655.2 \text{ g bsCOD/m}^3 \cdot \text{d} \end{aligned}$$

- b. Determine the net biomass production rate r_X using Eq. (7–21)

$$\begin{aligned} r_X &= Yr_{su} - bX \\ &= (0.45 \text{ g VSS/g bsCOD})(1655.2 \text{ g bsCOD/m}^3 \cdot \text{d}) \\ &\quad - (0.10 \text{ g VSS/g VSS} \cdot \text{d})(2000 \text{ g VSS/m}^3) \\ &= 544.8 \text{ g VSS/m}^3 \cdot \text{d} \end{aligned}$$

- c. Calculate the net biomass yield

$$\begin{aligned} Y_{\text{bio}} &= r_X/r_{su} = (544.8 \text{ g VSS/m}^3 \cdot \text{d})/(1655.2 \text{ g bsCOD/m}^3 \cdot \text{d}) \\ &= 0.33 \text{ g VSS/g bsCOD} \end{aligned}$$

2. Determine VSS production rate using Eq. (7–26).

$$\begin{aligned} r_{X_r, \text{VSS}} &= Yr_{su} - bX + f_d(b)X + QX_{o,i}/V \\ &= 544.8 \text{ g VSS/m}^3 \cdot \text{d} \\ &\quad + (0.10 \text{ g VSS/g VSS})(0.10 \text{ g VSS/g VSS} \cdot \text{d})(2000 \text{ g VSS/m}^3) \\ &\quad + (1000 \text{ m}^3/\text{d})(50 \text{ g VSS/m}^3)/335 \text{ m}^3 \\ &= (544.8 + 20 + 149.3) \text{ g VSS/m}^3 \cdot \text{d} \\ &= 714 \text{ g VSS/m}^3 \cdot \text{d} \end{aligned}$$

3. Calculate the observed solids yield using Eq. (7–30).

$$\begin{aligned} Y_{\text{obs}} &= r_{X_r, \text{VSS}}/r_{su} \\ &= (714 \text{ g VSS/m}^3 \cdot \text{d})/(1655.2 \text{ g bsCOD/m}^3 \cdot \text{d}) \\ &= 0.43 \text{ g VSS/g bsCOD} \end{aligned}$$

4. Calculate the active biomass fraction in the MLVSS. Using Eq. (7–28).

$$\begin{aligned} F_{X,\text{act}} &= (Yr_{su} - bX)/r_{X_r, \text{VSS}} \\ &= (544.8 \text{ g VSS/m}^3 \cdot \text{d})/(714 \text{ g VSS/m}^3 \cdot \text{d}) \\ &= 0.76 \end{aligned}$$

Comment Thus, accounting for the nbVSS in the wastewater influent and cell debris produced, the MLVSS contains 76 percent active biomass.

7-6 MODELING SUSPENDED GROWTH TREATMENT PROCESSES

Before discussing the individual biological processes used for the treatment of wastewater as given in Secs. 7–8 through 7–15, it will be helpful to review the general application of the kinetics of biological growth and substrate removal. The purpose here is to illustrate (1) the development of biomass and substrate balances, (2) the prediction of effluent biomass and soluble substrate concentrations, (3) the prediction of the reactor biomass and MLSS/MLVSS concentrations and amount of waste sludge produced daily, and (4) the prediction of the oxygen requirements. Attached growth processes are considered in Sec. 7–7.

Description of Suspended Growth Treatment Processes

The complete-mix reactor with recycle will be considered in the following discussion as a model for suspended growth processes. The schematic flow diagrams shown on Fig. 7–12 include the nomenclature used in the following mass-balance equations. A similar complete-mix reactor may be used in laboratory studies to assess wastewater treatability and to obtain model kinetic coefficients.

All biological treatment reactor designs are based on using mass balances across a defined volume for each specific constituent of interest (i.e., biomass, substrate, etc.). The mass balance includes the flowrates for the mass of the constituent entering and/or leaving the system and appropriate reaction rate terms for the depletion or production of the constituent within the system. The units for a mass balance are usually given in mass per volume per time. For all mass balances a check of the units is recommended to assure that the mass-balance equations are correct.

Solids Retention Time

Before proceeding with the mass balance for biomass and substrate in the completely mixed activated sludge (CMAS) process shown on Fig. 7–12, it is important to first select the system solids retention time (SRT) that will impact the solids production and is a key

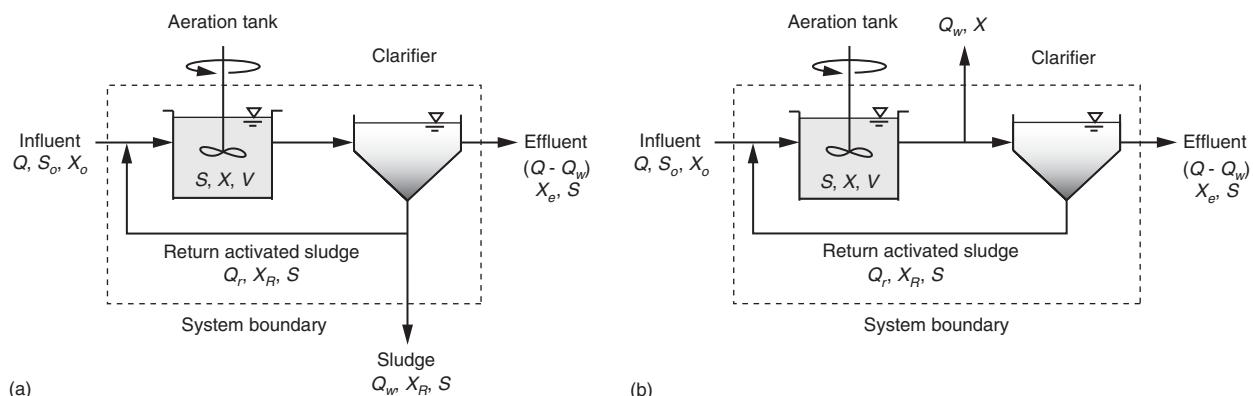


Figure 7-12

Schematic diagram of activated sludge process with model nomenclature: (a) with wasting from the sludge return line and (b) with wasting from the aeration tank.

operating and design parameter for activated sludge processes (Lawrence and McCarty, 1970). The SRT is the average time the activated sludge solids are in the system. Assuming that the solids inventory in the clarifier shown on Fig. 7–12(a) is negligible compared to that in the aeration tank, the SRT is determined by dividing the mass of solids in the aeration tank by the solids removed daily via the effluent and by wasting for process control. For many activated sludge processes, where good flocculation occurs and the clarifier is designed properly, the effluent VSS is typically less than 15 g/m³. Where the effluent VSS is low, excess solids must be removed from the system by wasting. Wasting is accomplished most commonly by removing biomass (sludge) from the clarifier underflow recycle line as shown on Fig. 7–12(a). Alternatively, wasting can be accomplished from the aeration tank as shown on Fig. 7–12(b).

The average SRT for the process flow diagram shown on Fig. 7–12(a) is given as

$$\text{SRT} = \frac{VX}{(Q - Q_w)X_e + Q_wX_R} \quad (7-31)$$

where SRT = solids retention time, d

V = reactor volume (i.e., aeration tank), m³

Q = influent flowrate, m³/d

X = concentration of biomass in the aeration tank, g VSS/m³

Q_w = waste sludge flowrate, m³/d

X_e = concentration of biomass in the effluent, g VSS/m³

X_R = concentration of biomass in the return activated sludge line from the clarifier, g VSS/m³

Based on Eq. (7–31), the SRT can be controlled by the wasting rate. Increasing the value for Q_w in Eq. (7–31) results in a lower SRT. Similarly, it can be shown that by wasting from the aeration tank, the SRT can be controlled by wasting a given percentage of the aeration tank volume each day.

The inverse of the SRT is the solids wasted per day divided by the solids present.

$$\frac{1}{\text{SRT}} = \frac{(Q - Q_w)X_e + Q_wX_R}{VX} \quad (7-32)$$

At steady state operation, where the influent flowrate and substrate concentration is constant, the reactor biomass concentration is constant, and the net biomass growth rate per day is equal to the solids wasting rate, [the numerator in Eq. (7–32)]. If the product of r_x , the net biomass growth rate per unit volume (g/m³·d) [see Eq (7–21)] and the volume, V , is substituted for the numerator in Eq. (7–32), it can be shown that the inverse of the SRT is the net specific biomass growth rate.

$$\frac{1}{\text{SRT}} = \frac{Vr_x}{VX} = \frac{r_x}{X} = \mu_{\text{net}} \quad (7-33)$$

Thus, based on Eq. (7–33), controlling the SRT by sludge wasting affects the net specific biomass growth rate, and the reactor substrate concentration. For a CMAS system the reactor effluent dissolved substrate concentration is equal to the reactor concentration.

Biomass Mass Balance

A mass balance for the mass of microorganisms in the complete-mix reactor shown on Fig. 7–12(a) can be written as follows:

1. General word statement:

$$\frac{\text{Rate of accumulation of microorganism within the system}}{\text{boundary}} = \frac{\text{rate of flow of microorganism into the system}}{\text{boundary}} - \frac{\text{rate of flow of microorganism out of the system}}{\text{boundary}} + \frac{\text{net growth of microorganism within the system}}{\text{boundary}} \quad (7-34)$$

2. Simplified word statement:

$$\text{Accumulation} = \text{inflow} - \text{outflow} + \text{net growth} \quad (7-35)$$

3. Symbolic representation:

$$\frac{dX}{dt}V = QX_o - [(Q - Q_w)X_e] - (Q_wX_R) + r_xV \quad (7-36)$$

where dX/dt = rate of change of biomass concentration in reactor measured as
g VSS/m³•d

V = reactor volume (i.e., aeration tank), m³

Q = influent flowrate, m³/d

X_o = concentration of biomass in influent, g VSS/m³

Q_w = waste sludge flowrate, m³/d

X_e = concentration of biomass in effluent, g VSS/m³

X_R = concentration of biomass in return line from clarifier, g VSS/m³

r_x = net rate of biomass production, g VSS/m³•d

If it is assumed that the concentration of microorganisms in the influent can be neglected and that steady-state conditions exist ($dX/dt = 0$), Eq. (7-36) can be simplified to

$$(Q - Q_w)X_e + Q_wX_R = r_xV \quad (7-37)$$

If Eq. (7-37) is combined with Eq. (7-21), the result is

$$\frac{(Q - Q_w)X_e + Q_wX_R}{VX} = Y\frac{r_{su}}{X} - b \quad (7-38)$$

where X = concentration of the biomass in the reactor, g/m³

The inverse of the term on the left-hand side of Eq. (7-38) is defined as the average solids retention time (SRT) as given above in Eq. (7-31). Thus,

$$\frac{1}{\text{SRT}} = Y\frac{r_{su}}{X} - b \quad (7-39)$$

The term, r_{su} , defined as the substrate utilization rate per unit of reactor volume [see Eq. (7-12)] is the amount of substrate removed in the reactor divided by the reactor volume:

$$r_{su} = \frac{Q(S_o - S)}{V} \quad (7-40)$$

Combining Eq. (7-39) and (7-40):

$$\frac{1}{\text{SRT}} = \frac{YQ(S_o - S)}{XV} - b \quad (7-41)$$

If Eq. (7-41) is rearranged as given by Eq. (7-42), the reactor biomass concentration is a function of the system SRT, the aerobic aeration tank hydraulic retention time, $\tau (V/Q)$, the

synthesis yield coefficient, the amount of substrate removed ($S_o - S$), and the specific endogenous decay coefficient.

$$X = \left(\frac{SRT}{\tau} \right) \left[\frac{Y(S_o - S)}{1 + b(SRT)} \right] \quad (7-42)$$

A substrate mass balance is needed to determine the effluent substrate concentration as a function of the reactor influent feed and operating conditions.

Substrate Mass Balance

The mass balance for substrate utilization in the aeration tank [see Fig. 7-12(a)] is

Accumulation = inflow – outflow + generation – utilization

$$\frac{dS}{dt}V = QS_o - QS + r_{su}V \quad (7-43)$$

where S_o = influent soluble substrate concentration, g/m³

Substituting the value for r_{su} [Eq. (7-12)] and assuming steady-state conditions ($dS/dt = 0$), Eq. (7-43) can be rewritten as

$$S_o - S = \left(\frac{V}{Q} \right) \left(\frac{kXS}{K_s + S} \right) \quad (7-44)$$

If Eq. (7-41) is substituted for X in Eq. (7-44), the resulting expression is

$$S_o - S = \left(\frac{V}{Q} \right) \left(\frac{kS}{K_s + S} \right) \left(\frac{SRT}{V} \right) \left[\frac{QY}{1 + b(SRT)} \right] S_o - S \quad (7-45)$$

Eliminating terms and solving for S yields

$$S = \frac{K_s[1 + b(SRT)]}{SRT(Yk - b) - 1} \quad (7-46)$$

It should be noted that in Eq. (7-46), the effluent soluble substrate concentration for a complete-mix activated sludge process is only a function of the SRT and the kinetic coefficients for growth and decay. The effluent substrate concentration is not related to the influent soluble substrate concentration, but as shown previously in Eq. (7-42), the influent substrate concentration affects the reactor biomass concentration.

The same equations can be applied to describe an activated sludge process with no clarifier and thus no return sludge flow. For the case with no return sludge, all of the solids produced are present in the effluent from the aeration tank, and the SRT equals the τ .

$$SRT = VX/QX = \tau \quad (7-47)$$

The importance of the system SRT in determining the effluent soluble substrate concentration and aeration tank biomass concentration is clear from an examination of Eqs. (7-46) and (7-42).

Mixed Liquor Solids Concentration and Solids Production

The solids production from a biological reactor represents the mass of material that must be removed each day to maintain the process. It is of interest to quantify the solids

production in terms of TSS, VSS, and biomass. By definition, the SRT also provides a convenient expression to calculate the total sludge produced daily from the activated sludge process:

$$P_{X_r,VSS} = \frac{X_r V}{SRT} \quad (7-48)$$

where $P_{X_r,VSS}$ = total solids wasted daily, g VSS/d

X_r = total MLVSS concentration in aeration tank, g VSS/m³

V = volume of reactor, m³

SRT = solids retention time, d

Because the 1/SRT in Eq. (7-32) represents the fraction of solids wasted per day and the mixed liquor can be assumed to be a homogeneous mixture of biomass and other solids, Eq. (7-48) can be used to calculate the amount of solids wasted for any of the mixed liquor components. For the amount of biomass wasted per day (P_X), the biomass concentration X can be used in place of X_r in Eq. (7-48).

Mixed Liquor Solids Concentration. The total MLVSS in the aeration tank equals the biomass concentration X plus the nbVSS concentration X_i :

$$X_r = X + X_i \quad (7-49)$$

A mass balance is needed to determine the nbVSS concentration in addition to the active biomass VSS concentration. The MLVSS nbVSS concentration is affected by the amount of nbVSS in the influent wastewater, the amount of nbVSS wasted per day, and the amount of cell debris produced from cell decay. A materials balance on the inert material is as follows:

Accumulation = inflow – outflow + generation

$$(dX_i/dt)V = QX_{o,i} - X_i V/SRT + r_{X,i}V \quad (7-50)$$

where $X_{o,i}$ = nbVSS concentration in influent, g/m³

X_i = nbVSS concentration in aeration tank, g/m³

$r_{X,i}$ = rate of nbVSS production from cell debris, g/m³·d

At steady-state ($dX_i/dt = 0$) and substituting Eq. (7-26) for $r_{X,i}$ in Eq. (7-50) yields

$$0 = QX_{o,i} - X_i V/SRT + (f_d)(b)XV \quad (7-51)$$

$$X_i = X_{o,i}(SRT)/\tau + (f_d)(b)X(SRT) \quad (7-52)$$

Substituting Eq. (7-42) for X and Eq. (7-52) for X_i in Eq. (7-49) produces the following equation that can be used to determine the total MLVSS concentration:

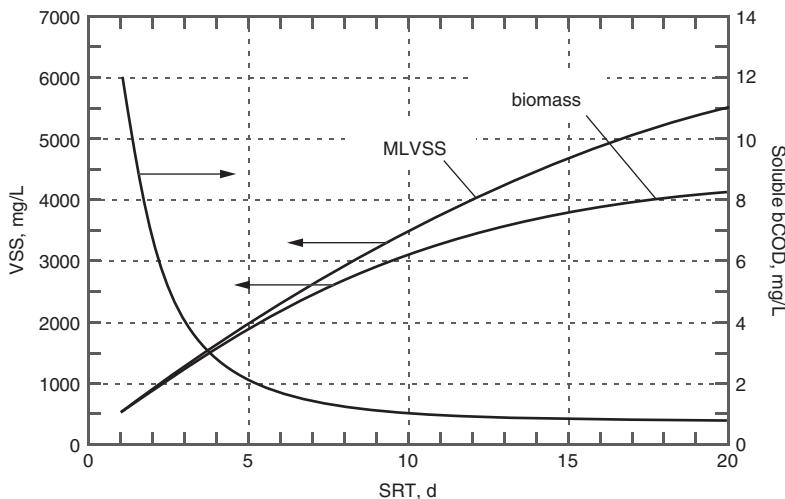
$$X_r = \left(\frac{SRT}{\tau} \right) \left[\frac{Y(S_o - S)}{1 + b(SRT)} \right] + (f_d)(b)(X)(SRT) + \frac{(X_{o,i})(SRT)}{\tau} \quad (7-53)$$

$\begin{array}{c} (A) \\ \text{Heterotrophic} \\ \text{biomass} \end{array}$
 $\begin{array}{c} (B) \\ \text{Cell debris} \end{array}$
 $\begin{array}{c} (C) \\ \text{Nonbiodegradable} \\ \text{VSS in influent} \end{array}$

Solids Production. The amount of VSS produced and wasted daily can be estimated by substituting Eq. (7-53) into Eq. (7-48), and replacing τ with V/Q . The resulting

Figure 7-13

Biodegradable soluble COD, biomass, and MLVSS concentrations versus SRT for complete-mix activated sludge process.



equation is expressed as a function of substrate degraded, influent nbVSS, and kinetic coefficients.

$$P_{X,VSS} = \frac{QY(S_o - S)}{1 + b(\text{SRT})} + \frac{(f_d)(b)YQ(S_o - S)\text{SRT}}{1 + b(\text{SRT})} + QX_{o,i} \quad (7-54)$$

(A) (B) (C)
 Heterotrophic Cell debris Nonbiodegradable
 Biomass VSS in influent

The effect of SRT on the performance of an activated sludge system for soluble substrate removal is illustrated on Fig. 7-13. In addition to the soluble substrate concentration, the total VSS concentration which includes nbVSS is also shown. As the SRT increases, more biomass decays and thus more cell debris accumulates, so that the difference between MLVSS and biomass VSS concentration increases with SRT. Also illustrated on Fig. 7-13 is the fact that the soluble substrate concentration is very low ($\text{bsCOD} < 5 \text{ mg/L}$) at SRTs above 2 d. The low substrate concentration is typical of the activated sludge process when used for the treatment of municipal wastewaters and illustrates how effectively the organic compounds are degraded in the activated sludge process. As will be shown in Chap. 8, organic substrate degradation is usually not the major factor in selecting a design SRT value.

The total mass of dry solids wasted per day is based on the TSS, which includes the VSS plus inorganic solids. Inorganic solids are present in the influent wastewater (TSS – VSS) and in the biomass, which contains 10 to 15 percent inorganic solids by dry weight. The influent inorganic solids are not soluble, and are assumed captured in the mixed liquor solids and removed in the wasted solids. To calculate the solids production in terms of TSS Eq. (7-54) is modified by adding the influent inorganic solids and by calculating the biomass in terms of TSS by assuming a typical biomass VSS/TSS ratio of 0.85. The ratio of VSS/TSS may vary from 0.80 to 0.90.

$$P_{X,TSS} = \frac{A}{0.85} + \frac{B}{0.85} + C + Q(\text{TSS}_o - \text{VSS}_o) \quad (7-55)$$

where $P_{X,TSS}$ = net waste activated sludge produced each day, measured in terms of total suspended solids, kg/d

TSS_o = influent wastewater TSS concentration, g/m³

VSS_o = influent wastewater VSS concentration, g/m³

A, B, and C are as defined in Eq. (7-54).

The mass of MLVSS and MLSS can be obtained by using Eqs. (7-54) and (7-55), respectively, with Eq. (7-48) as follows:

$$\text{Mass of MLVSS} = (X_{\text{VSS}})(V) = (P_{X,\text{VSS}}) \text{ SRT} \quad (7-56)$$

$$\text{Mass of MLSS} = (X_{\text{TSS}})(V) = (P_{X,\text{TSS}}) \text{ SRT} \quad (7-57)$$

By selecting an appropriate MLSS concentration, the aeration volume can be determined from Eq. (7-57). Typical MLSS concentrations in the range of 2000 to 4000 mg/L may be selected, and they must be compatible with the sludge settling characteristics and clarifier design as discussed in Secs. 8–10 and 8–11 in Chap. 8.

The Observed Yield

The observed yield Y_{obs} is based on the amount of solids production measured relative to the substrate removal, and may be calculated in terms of g TSS/g bsCOD or g BOD, or relative to VSS as g VSS/g bsCOD or g BOD. The measured solids production is the sum of the solids in the system effluent flow and the solids intentionally wasted, which equals the term P_X defined in Eqs. (7-54) and (7-55). The observed yield for VSS can be calculated by dividing Eq. (7-54) by the substrate removal rate, which is $Q(S_o - S)$:

$$Y_{\text{obs}} = \frac{Y}{1 + b(\text{SRT})} + \frac{(f_d)(b)(Y)(\text{SRT})}{1 + b(\text{SRT})} + \frac{X_{o,i}}{S_o - S} \quad (7-58)$$

(A)	(B)	(C)
Heterotrophic biomass	Cell debris	Nonbiodegradable VSS in influent

where $Y_{\text{obs}} = \text{g VSS/g substrate removed}$

For wastewaters with no nbVSS in the influent the solids production consists of only active biomass and cell debris, and the observed yield for VSS is as follows:

$$Y_{\text{obs}} = \frac{Y}{1 + b(\text{SRT})} + \frac{(f_d)(b)(Y)(\text{SRT})}{1 + b(\text{SRT})} \quad (7-59)$$

The impact of nonbiodegradable influent VSS in Eq. (7-58) on the observed yield depends on the wastewater characteristics and the type of pretreatment. The effluent substrate concentration is generally very low compared to S_o , and the term $X_{o,i}/(S_o - S)$ can be approximated by $X_{o,i}/S_o$, which is the g nbVSS/g BOD in the influent. For municipal wastewater $X_{o,i}/S_o$ values range from 0.10 to 0.30 g nbVSS/g BOD with primary treatment and 0.30 to 0.50 g nbVSS/g BOD without primary treatment. Sludge production is also addressed in Chap. 8.

Oxygen Requirements

The oxygen required for the biodegradation of carbonaceous material is determined from a mass balance using the bCOD concentration of the wastewater treated and the amount of biomass wasted from the system per day. If all of the bCOD were oxidized to CO_2 and H_2O , the oxygen demand would equal the bCOD concentration, but bacteria only oxidize a portion of the bCOD to provide energy and use a portion of the bCOD for cell growth. Oxygen is also consumed for endogenous respiration, and the amount will depend on the system SRT. For a given SRT, a mass balance on the system can be done where the bCOD removal equals the oxygen used plus the biomass VSS remaining (in terms of an oxygen equivalent), as given by Eq. (7-7). Thus, for a suspended growth process, the oxygen used is

$$\text{Oxygen used} = \text{bCOD removed} - \text{COD of waste sludge} \quad (7-60)$$

$$R_o = Q(S_o - S) - 1.42P_{X,\text{bio}} \quad (7-61)$$

where R_o = oxygen required, kg/d

$P_{X,\text{bio}}$ = biomass as VSS wasted per day, kg/d

It is important to note that $P_{X,\text{bio}}$ includes active biomass and cell debris derived from cell growth and is thus the sum of terms A and B in Eq. (7-54).

EXAMPLE 7-6 Design of a Complete-mix Suspended Growth Process A complete-mix suspended growth activated sludge process with recycle is used to treat municipal wastewater after primary sedimentation. The characteristics of the primary effluent are: flow = 1000 m³/d, bsCOD = 192 g/m³, nbVSS = 30 g/m³, and inert inorganics = 10 g/m³. The aeration tank MLVSS = 2500 g/m³. Using these data and the kinetic coefficients given below, design a system with a 6-d SRT and determine the following:

1. What is the effluent bsCOD concentration?
2. What value of τ should be used so that the MLVSS concentration is 2500 g/m³?
3. What is the daily sludge production in kg/d as VSS and TSS?
4. What is the fraction of biomass in the MLVSS?
5. What is the observed solids yield in g VSS/g bsCOD and g TSS/g bsCOD?
6. What is the oxygen requirement in kg/d?

Kinetic coefficients:

$$k = 12.5 \text{ g COD/g VSS}\cdot\text{d} \quad K_s = 10 \text{ g COD/m}^3$$

$$Y = 0.40 \text{ g VSS/g COD used} \quad f_d = 0.15 \text{ g VSS/g VSS}$$

$$b = 0.10 \text{ g VSS/g VSS}\cdot\text{d} \quad \text{Biomass VSS/TSS} = 0.85$$

Solution

1. Determine the effluent bsCOD concentration using Eq. (7-46).

$$\begin{aligned} S &= \frac{K_s[1 + b(\text{SRT})]}{\text{SRT}(Yk - b) - 1} \\ &= \frac{(10 \text{ g bsCOD/m}^3)[1 + (0.10 \text{ g VSS/g VSS}\cdot\text{d})(6 \text{ d})]}{(6 \text{ d})[(0.40 \text{ g VSS/g COD})(12.5 \text{ g COD/g VSS}\cdot\text{d}) - (0.10 \text{ g VSS/g VSS}\cdot\text{d})] - 1} \\ &= 0.56 \text{ g bsCOD/m}^3 \end{aligned}$$

2. Determine τ for 2500 g/m³ MLVSS concentration.

Solve for τ in Eq. (7-53).

$$X_T = Y(S_o - S)\text{SRT}/[1 + b(\text{SRT})](\tau) + (f_d)(b)(X)\text{SRT} + (X_{o,i})\text{SRT}/\tau$$

$$\begin{aligned} 2500 \text{ g VSS/m}^3 &= (0.40 \text{ g VSS/g COD})[(192 - 0.56) \text{ g COD/m}^3](6 \text{ d})/ \\ &\quad [(1 + 0.10 \text{ g VSS/g VSS}\cdot\text{d})(6 \text{ d})(\tau)] \\ &\quad +(0.15 \text{ g VSS/g VSS})(0.10 \text{ g VSS/g VSS}\cdot\text{d})(X)(6 \text{ d}) \\ &\quad + 30 \text{ g bsCOD/m}^3(6 \text{ d}/\tau) \end{aligned}$$

$$2500 = 287.2/\tau + 0.09(X) + 180/\tau$$

The biomass concentration X is determined using Eq. (7-42).

$$\begin{aligned} X &= [Y(S_o - S)]SRT/[1 + b(SRT)](\tau) \\ &= \frac{(0.40 \text{ g VSS/g COD})[(192 - 0.56) \text{ g COD/m}^3](6 \text{ d})}{[1 + (0.10 \text{ g VSS/g COD})(6 \text{ d})](\tau)} \\ &= (287.2 \text{ g/m}^3 \cdot \text{d})/\tau \end{aligned}$$

Substituting for X in the above expression yields:

$$2500 = 287.2/\tau + 180/\tau + 25.8/\tau = 493/\tau$$

and solving the above expression for τ yields

$$\tau = 0.197 \text{ d}$$

$$\text{Aeration tank volume} = \tau(Q) = 0.197 \text{ d} (1000 \text{ m}^3/\text{d}) = 197 \text{ m}^3$$

3. Determine the total sludge production as kg VSS/d using Eq. (7-48).

$$\begin{aligned} P_{X_r, \text{VSS}} &= X_r(V)/(SRT) \\ &= (2500 \text{ g VSS/m}^3)(197 \text{ m}^3)(1 \text{ kg}/10^3 \text{ g})/6 \text{ d} = 82.1 \text{ kg VSS/d} \end{aligned}$$

4. Determine the total sludge production as kg TSS/d using Eq. (7-55) and the assumed coefficients.

$$\begin{aligned} P_{X_r, \text{TSS}} &= \frac{QY(S_o - S)}{1 + (b)SRT} \left(\frac{1}{0.85} \right) + \frac{(f_d)(b)YQ(S_o - S)SRT}{1 + (b)SRT} \left(\frac{1}{0.85} \right) + QX_{o,i} + Q(\text{TSS}_o - \text{VSS}_o) \\ &= \frac{(1000 \text{ m}^3/\text{d})(0.40 \text{ g VSS/g COD})[(192 - 0.56) \text{ g COD/m}^3]}{[1 + (0.10 \text{ g VSS/g VSS} \cdot \text{d})(6 \text{ d})](0.85)} \\ &\quad + \frac{(0.15)(0.10)(1000 \text{ m}^3/\text{d})(0.40)[(192 - 0.56) \text{ g COD/m}^3](6 \text{ d})}{[1 + (0.10 \text{ g VSS/g VSS} \cdot \text{d})(6 \text{ d})](0.85)} \\ &\quad + (1000 \text{ m}^3/\text{d})(30 \text{ g/m}^3) + (1000 \text{ m}^3/\text{d})(10 \text{ g/m}^3) \\ &= (56.3 + 5.1 + 30 + 10)(10^3 \text{ g/d}) = 101.4 \times 10^3 \text{ g/d} = 101.4 \text{ kg/d} \end{aligned}$$

5. Determine the biomass fraction from the values for X and X_r .

$$X = (287.2 \text{ g/m}^3 \cdot \text{d})/\tau = (287.2 \text{ g/m}^3 \cdot \text{d})/0.197 \text{ d} = 1458 \text{ g VSS/m}^3$$

$$\text{Biomass fraction} = X/X_r = 1458/2500 = 0.58$$

6. Calculate the observed solids yield, g VSS/g bsCOD removed and g TSS/g bsCOD removed.

$$\text{Solids wasted/d} = P_{X_r} = 82.1 \text{ kg VSS/d} \text{ and } 101.4 \text{ kg TSS/d}$$

$$\begin{aligned} \text{bsCOD removed/d} &= Q(S_o - S) \\ &= (1000 \text{ m}^3/\text{d})[(192 - 0.56) \text{ g COD/m}^3](1 \text{ kg}/10^3 \text{ g}) \\ &= 191,440 \text{ g COD/d} = 191.4 \text{ kg/d} \end{aligned}$$

$$\text{As VSS, } Y_{\text{obs}} = 82.1/191.4 = 0.43 \text{ g VSS/g bsCOD}$$

$$\text{As TSS, } Y_{\text{obs}} = 101.4/191.4 = 0.53 \text{ g TSS/g bsCOD}$$

7. Determine the oxygen required using Eq. (7-61).

$$R_o = Q(S_o - S) - 1.42 P_{X,\text{bio}}$$

$$\begin{aligned} P_{X,\text{bio}} &= P_{X,VSS} - P_{nbVSS} \\ &= 82.1 \text{ kg/d} - (1000 \text{ m}^3/\text{d})(30 \text{ g VSS/m}^3)(1 \text{ kg}/10^3 \text{ g}) = 52.2 \text{ kg/d} \end{aligned}$$

$$\begin{aligned} R_o &= (1000 \text{ m}^3/\text{d})[(192 - 0.56)\text{g COD/m}^3](1 \text{ kg}/10^3 \text{ g}) - 1.42(52.2 \text{ kg VSS/d}) \\ &= 117.7 \text{ kg O}_2/\text{d} \end{aligned}$$

Comment The same approach can be used to treat wastewater with particulate biodegradable COD by assuming it is equal to bsCOD. For complete-mix suspended growth designs if the SRT is 3 d or more, essentially all of the degradable particulate COD will be converted to bsCOD.

Design and Operating Parameters

In the mass balance for the complete mix reactor presented above, the SRT was introduced as the fundamental process parameter that affects the treatment efficiency and general performance for the activated sludge process. Two other activated sludge process parameters used for the design and operation of the activated sludge process, the food to microorganism ratio and the volumetric loading rate, are introduced below.

Food to Microorganism (F/M) Ratio. The F/M ratio is defined as the rate of BOD or COD applied per unit volume of mixed liquor:

$$F/M = \frac{\text{total applied substrate rate}}{\text{total microbial biomass}} = \frac{QS_o}{VX} \quad (7-62)$$

and

$$F/M = \frac{S_o}{\tau X} \quad (7-63)$$

where F/M = food to biomass ratio, g BOD or bsCOD/g VSS·d

Q = influent wastewater flowrate, m^3/d

S_o = influent BOD or bsCOD concentration, g/m^3

V = aeration tank volume, m^3

X = mixed liquor biomass concentration in the aeration tank, g/m^3

τ = hydraulic retention time of aeration tank, V/Q , d

The F/M ratio is useful for understanding the effect of transient loads on a system. The higher the specific BOD loading rate ($\text{g BOD/g VSS}\cdot\text{d}$), the faster is the substrate utilization rate and thus the reactor would have a higher substrate concentration.

F/M ratio and SRT. The F/M ratio can be related to the system SRT by noting that there is a given substrate removal efficiency at a given F/M. The process removal efficiency E as percent BOD or bsCOD removed across the activated sludge system is defined as

$$E = \frac{S_o - S}{S_o}(100) \quad (7-64)$$

Thus E/100 times F/M equals

$$\frac{E}{100} \left(\frac{F}{M} \right) = \frac{QS_o(S_o - S)}{VX(S_o)} = \frac{Q(S_o - S)}{VX} \quad (7-65)$$

Substituting Eq. (7-40) in Eq. (7-65) yields

$$\frac{E}{100} \left(\frac{F}{M} \right) = \frac{r_{su}}{X} \quad (7-66)$$

In Eq. (7-66), the term (r_{su}/X) is known as the *specific substrate utilization rate*, U . Rearranging Eq. (7-39) it can be shown that U is related to the SRT and the bacteria growth and decay coefficients.

$$U = \frac{r_{su}}{X} = \frac{\left(\frac{1}{SRT} + b \right)}{Y} \quad (7-67)$$

where terms are as defined earlier.

Substituting Eq. (7-67) into Eq. (7-66) yields

$$\frac{1}{SRT} = Y(F/M) \frac{E}{100} - b \quad (7-68)$$

From Eq. (7-68) it can be seen that systems operated with a higher F/M ratio result in a lower steady-state SRT value. For systems designed for the treatment of municipal wastewater with activated sludge SRT values in the 20- to 30-d range, the corresponding F/M values may range from 0.10 to 0.05 g BOD/g VSS·d, respectively. At SRTs in the range of 5 to 7 d, the F/M values may range from 0.3 to 0.5 g BOD/g VSS·d, respectively.

Organic Volumetric Loading Rate. The organic volumetric loading rate, defined as the amount of BOD or COD applied to the aeration tank volume per day, is

$$L_{org} = \frac{(Q)(S_o)}{(V)(10^3 \text{ g/l kg})} \quad (7-69)$$

where L_{org} = volumetric organic loading rate, kg BOD/m³·d

Q = influent wastewater flowrate, m³/d

S_o = influent BOD concentration, g/m³

V = aeration tank volume, m³

Process Performance and Stability

The effects of the kinetics considered above on the performance and stability of the system shown on Fig. 7-14 will now be examined further. It was shown previously that $1/SRT$, the net microorganism specific growth rate, and U , the specific substrate utilization rate, are related directly [see Eqs. (7-67) and (7-39)]. For a specified waste, a given biological community, and a particular set of environmental conditions, the kinetic coefficients Y , k , K_s , and b are fixed. It is important to note that domestic wastewater may have significant variability in its composition and may not always be treated as a single waste type in evaluating the kinetic coefficients. For given values of the coefficients, the steady state effluent substrate concentration from the reactor is a direct function of the SRT, as given by Eq. (7-46). Setting the SRT value fixes the values of U and m and also defines the efficiency of biological waste stabilization. Equation (7-46) for substrate is plotted on Fig. 7-14(a) for a growth-specified complete-mix system with recycle. As shown, the

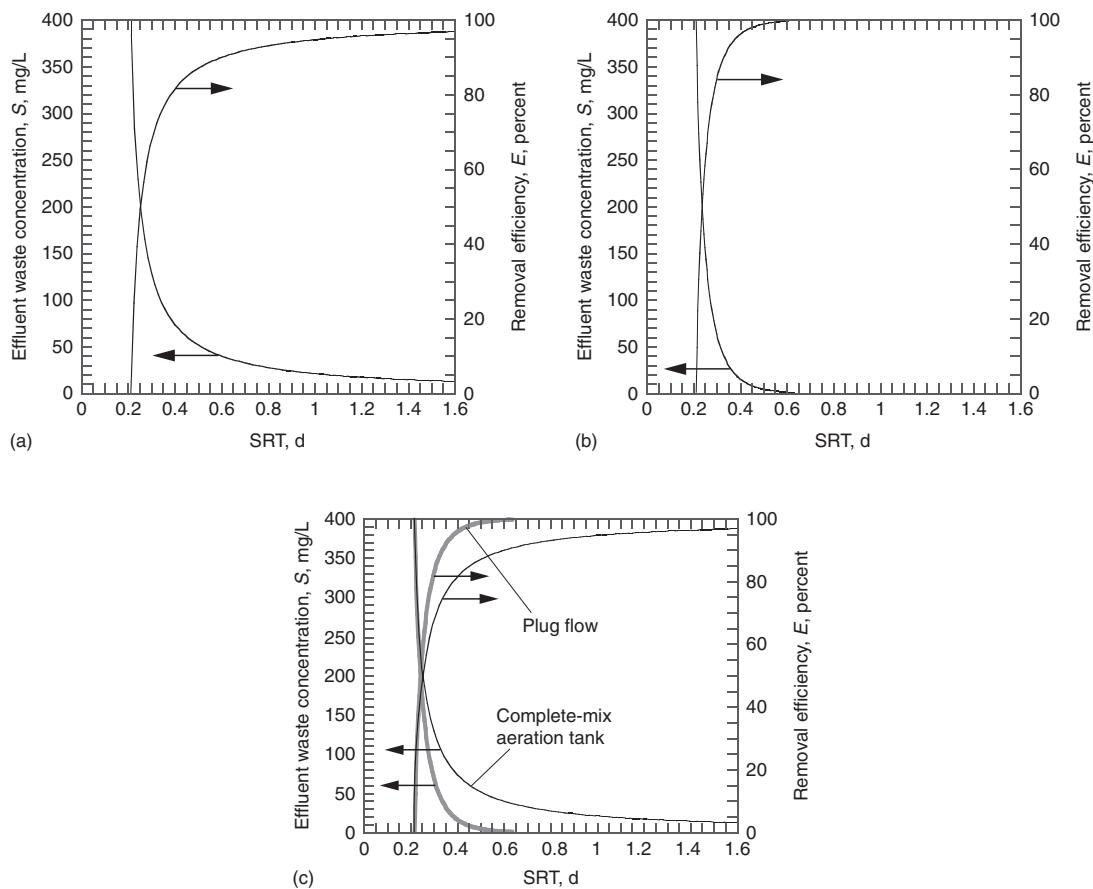


Figure 7-14

Effluent substrate concentration and removal efficiency versus SRT for: (a) complete-mix reactors with recycle, (b) plug flow reactor with recycle, and (c) plug flow and complete-mix plotted on the same graph for comparison.

treatment efficiency and the substrate concentration are related directly to the SRT, and the reactor hydraulics (i.e., complete-mix or plug-flow).

It can also be seen from Fig. 7-14(a) that there is a certain value of SRT below which waste stabilization does not occur. The critical SRT value is called the minimum solids retention residence time SRT_{min} . Physically, SRT_{min} is the residence time at which the cells are washed out or wasted from the system faster than they can reproduce. The minimum SRT can be calculated using Eq. (7-23), in which $S = S_o$. When washout occurs, the influent concentration S_o is equal to the effluent waste concentration S .

$$\frac{1}{SRT_{min}} = \frac{YkS_o}{K_s + S_o} - b \quad (7-70)$$

In many situations encountered in waste treatment, S_o is much greater than K_s so that Eq. (7-70) can be rewritten to yield

$$\frac{1}{SRT_{min}} \approx Yk - b \quad (7-71)$$

or

$$\frac{1}{SRT_{min}} \approx \mu_m - b \quad (7-72)$$

Equations (7-71) and (7-72) can be used to determine the SRT_{min} . Typical kinetic coefficients that can be used to solve for SRT_{min} for BOD removal systems are given in Table 7-8. Biological treatment processes should not be designed with SRT values equal to SRT_{min} . To ensure adequate waste treatment, biological treatment processes are usually designed and operated with a design SRT value from 2 to 20 times SRT_{min} . In effect, the ratio of the design SRT (SRT_{des}) to SRT_{min} can be considered to be a process safety factor, SF, against system failure (Lawrence and McCarty, 1970).

$$SF = \frac{SRT_{des}}{SRT_{min}} \quad (7-73)$$

Modeling Plug-Flow Reactors

The plug-flow system with biomass recycle can be used to model certain forms of the activated sludge process. The distinguishing feature of this recycle system is that the hydraulic regime of the reactor is of a plug-flow nature. In a true plug-flow model, all the particles entering the reactor stay in the reactor an equal amount of time. Some particles may make more passes through the reactor because of recycle, but while they are in the tank, all pass through in the same amount of time.

A kinetic model of the plug-flow system is mathematically complex, but Lawrence and McCarty (1970) have made two simplifying assumptions that lead to a useful kinetic model of the plug-flow reactor:

1. The concentration of microorganisms in the influent to the reactor is approximately the same as that in the effluent from the reactor. This assumption applies only if $SRT/\tau > 5$. The resulting average concentration of microorganisms in the reactor is symbolized as \bar{X} .
2. The rate of change of substrate concentration as the waste passes through the reactor is given by the following expression:

$$\frac{dS}{dt} = -\frac{kS\bar{X}}{K_s + S} \quad (7-74)$$

Integrating Eq. (7-74) over the retention time of the wastewater in the aeration tank, substituting Eq. (7-42) for \bar{X} , and simplifying, the following expression is obtained:

$$\frac{1}{SRT} = \frac{Yk(S_o - S)}{(S_o - S) + (1 + R)K_s \ln(S_i/S)} - b \quad (7-75)$$

where S_o = influent concentration

S = effluent concentration

S_i = influent concentration to reactor after dilution with recycle flow

$$= \frac{S_o + RS}{1 + R}$$

R = clarifier return sludge recycle ratio (return flowrate divided by the influent wastewater flowrate)

Other terms are as defined previously.

The effluent substrate concentration in Eq. (7–75) is a function of both the influent concentration and SRT, whereas for a complete-mix system [see Eq. (7–46)], the effluent substrate concentration was only a function of the SRT. A version of Eq. (7–75) in which Eq. (7–42) is not substituted for \bar{X} is shown in Chap. 8 in the design of sequencing batch reactors.

The true plug-flow recycle system is theoretically more efficient in the stabilization of most soluble wastes than in the continuous-flow stirred-tank recycle system. A graphical representation is shown on Fig. 7–14(b). In actual practice, a true plug-flow regime is essentially impossible to obtain because of longitudinal dispersion caused by aeration and mixing. By dividing the aeration tank into a series of reactors, the process approaches plug-flow kinetics with improved treatment efficiency compared to a complete-mix process. Because of the greater dilution with the influent wastewater, the complete-mix system can handle shock loads better than staged reactors in series. Reactor selection is discussed further in Chap. 8.

7–7 SUBSTRATE REMOVAL IN ATTACHED GROWTH TREATMENT PROCESS

In an attached growth treatment process, a biofilm consisting of microorganisms, particulate material, and extracellular polymers is attached and covers the support packing material, which may be plastic, rock, or other material (see Fig. 7–15). The growth and substrate utilization kinetics described for the suspended growth process were related to the dissolved substrate concentration in the bulk liquid. For attached growth processes, substrate is consumed within a biofilm.



(a)



(b)

Figure 7–15

Typical packing for trickling filters: (a) rock with fixed spray nozzles and (b) plastic media in tower trickling filter.

8

Suspended Growth Biological Treatment Processes

8-1	INTRODUCTION TO THE ACTIVATED SLUDGE PROCESS 700 <i>Historical Development of Activated Sludge Process</i> 701 <i>Basic Process Description</i> 701 <i>Evolution of the Conventional Activated Sludge Process</i> 702 <i>Nutrient Removal Processes</i> 706
8-2	WASTEWATER CHARACTERIZATION 707 <i>Key Wastewater Constituents for Process Design</i> 707 <i>Measurement Methods for Wastewater Characterization</i> 712 <i>Recycle Flows and Loadings</i> 716
8-3	FUNDAMENTALS OF PROCESS SELECTION, DESIGN, AND CONTROL 717 <i>Overall Considerations in Treatment Process Implementation</i> 717 <i>Important Factors in Process Selection and Design</i> 717 <i>Process Control</i> 726 <i>Operational Problems in Activated Sludge Systems with Secondary Clarifiers</i> 732 <i>Operational Problems with MBR Systems</i> 738
8-4	SELECTOR TYPES AND DESIGN CONSIDERATION 738 <i>Selector Types and Design Considerations</i> 739 <i>Poor Settling Even With Use of Selector</i> 741
8-5	ACTIVATED SLUDGE PROCESS DESIGN CONSIDERATIONS 742 <i>Steady-State Design Approach</i> 742 <i>Use of Simulation Model</i> 744 <i>Model Matrix Format, Components, and Reactions</i> 747 <i>Other Simulation Model Applications</i> 751
8-6	PROCESSES FOR BOD REMOVAL AND NITRIFICATION 752 <i>Overview of BOD Removal and Nitrification Processes</i> 752 <i>General Process Design Considerations</i> 754 <i>Complete-Mix Activated Sludge Process Design</i> 754 <i>Sequencing Batch Reactor Process Design</i> 771 <i>Staged Activated Sludge Process Design</i> 782 <i>Alternative Processes for BOD Removal and Nitrification</i> 786
8-7	PROCESSES FOR BIOLOGICAL NITROGEN REMOVAL 795 <i>Process Development</i> 796 <i>Overview of Types of Biological Nitrogen-Removal Processes</i> 797 <i>General Process Design Considerations</i> 802 <i>Preanoxic Denitrification Processes</i> 804 <i>Postanoxic Denitrification Processes</i> 831 <i>Low DO and Cyclic Nitrification/Denitrification Processes</i> 833 <i>Alternative Process Configurations for Biological Nitrogen Removal</i> 838

	<i>Denitrification with External Carbon Addition</i>	848
	<i>Process Control and Performance</i>	860
8-8	PROCESSES FOR ENHANCED BIOLOGICAL PHOSPHORUS REMOVAL	861
	<i>Process Development</i>	861
	<i>Overview of Enhanced Biological Phosphorus Removal Processes</i>	862
	<i>General Process Design Considerations</i>	864
	<i>Operational Factors That Affect Enhanced Biological Phosphorus Removal</i>	878
	<i>Enhanced Biological Phosphorus Removal Process Design</i>	880
	<i>Provision for Chemical Addition</i>	883
	<i>Process Control and Performance Optimization</i>	884
8-9	AERATION TANK DESIGN FOR ACTIVATED SLUDGE PROCESSES	885
	<i>Aeration System</i>	885
	<i>Aeration Tanks and Appurtenances</i>	886
8-10	ANALYSIS OF LIQUID-SOLIDS SEPARATION FOR ACTIVATED SLUDGE PROCESSES WITH CLARIFIERS	889
	<i>Solids Separation by Secondary Clarifiers</i>	889
	<i>Assessing Sludge Thickening Characteristics</i>	891
	<i>Clarifier Design Based on Solids Flux Analysis</i>	893
	<i>Clarifier Design Based on State Point Analysis</i>	900
8-11	DESIGN CONSIDERATIONS FOR SECONDARY CLARIFIERS	906
	<i>Types of Sedimentation Tank</i>	906
	<i>Sidewater Depth</i>	910
	<i>Flow Distribution</i>	910
	<i>Tank Inlet Design</i>	910
	<i>Weir Placement and Loading</i>	912
	<i>Scum Removal and Management</i>	912
8-12	SOLIDS SEPARATION FOR MEMBRANE BIOREACTORS	913
	<i>Design Parameter</i>	913
	<i>Membrane Properties</i>	914
	<i>Membrane Design and Operating Characteristics</i>	917
	<i>Membrane Usage</i>	917
	<i>Membrane Fouling Issues</i>	917
	PROBLEMS AND DISCUSSION TOPICS	919
	REFERENCES	934

WORKING TERMINOLOGY

Term	Definition
Activated sludge process	Biological treatment process that involves the conversion of organic matter and/or other constituents in the wastewater to gases and cell tissue by a large mass of aerobic microorganisms maintained in suspension by mixing and aeration. The microorganisms form flocculent particles that are separated from the process effluent in a sedimentation tank (clarifier) and are returned subsequently to the aeration process or wasted.

Term	Definition
Aerobic (oxic) processes	Biological treatment processes that occur in the presence of free dissolved oxygen; oxygen is consumed by aerobic microorganisms in oxidation/reduction reactions to produce energy for cell growth and cell maintenance.
Anaerobic processes	Biological treatment processes that occur in the absence of oxygen.
Anoxic process	Biological treatment process that occurs in the absence of free dissolved oxygen where nitrate and nitrite are used as the main electron acceptors in biological oxidation/reduction reactions; denitrification is an example of an anoxic process.
Biomass	The total mass of solids in a reactor consisting mainly of organic matter and microorganisms.
Biological nutrient removal (BNR)	The term applied to the removal of nitrogen and phosphorus in biological treatment processes.
Denitrification	The biological process by which nitrate or nitrite is reduced to nitrogen and other gaseous end products.
Enhanced biological phosphorus removal (EBPR)	Removal of phosphorus by extraordinary storage in bacteria selected in anaerobic/aerobic process configuration and subsequent solids separation.
Hindered settling	Settling which occurs when the activated sludge flocs interfere with each other as they settle.
Facultative processes	Biological treatment processes in which the organisms can function in the presence or absence of molecular oxygen.
Fermentation	The conversion of organic matter to volatile fatty acids in the absence of oxygen, nitrate, and nitrite.
Membrane bioreactor (MBR)	A process that combines a suspended growth process with a membrane separation system within the process aeration tank; membrane separation is accomplished by either microfiltration or ultrafiltration.
Membrane flux	The rate of flow across a membrane per unit of surface area, L/m ² ·h.
Mixed liquor suspended solids (MLSS)	The biomass contained in a treatment reactor used to bring about treatment of the organic material in wastewater.
Nitrification	The two-step biological process by which nitrogen (mostly in the form of ammonia) is converted to nitrite and then to nitrate.
Nocardioform foam	A thick layer of brown, biological foam caused by a filamentous bacteria that forms on the top of aeration tanks and secondary clarifiers.
Nonbiodegradable volatile suspended solids (nbVSS)	These are suspended solids contained in influent wastewater to activated sludge processes that are organic but not biodegradable. They impact sludge production.
Phosphorus accumulating organisms (PAOs)	Heterotrophic bacteria selected in EBPR processes that have the ability for high intracellular phosphorus storage.
Readily biodegradable COD (rbCOD)	Dissolved biodegradable organic substrates which are removed by bacteria much faster than colloidal or particulate degradable COD. The rbCOD impacts spatial oxygen demand, EBPR removal efficiency, and denitrification rates.
Sequencing batch reactor (SBR)	An SBR is a batch fill and draw activated sludge treatment process. It involves a treatment sequence of fill, react, settling, supernatant decanting, and idle. Activated sludge aeration and liquid solids separation occurs in the same tank.
Simulation models	Mathematical models, based on a set of equations, used to assess the effects of kinetics and changes in the wastewater characterizes on process performance.
Simultaneous nitrification and denitrification (SNdN)	Nitrogen removal occurs in same activated sludge floc or in a biofilm due to nitrification in aerobic outer layer and denitrification in interior due to the lack of dissolved oxygen and presence of nitrate or nitrite.

Term	Definition
Sludge production	The amount of solids produced during the biological processing of wastewater including influent nonbiodegradable solids and the biomass resulting from the conversion of organic.
Sludge yield	The amount of solids produced relative to the amount of BOD or COD removed during the biological processing of wastewater.
Solids flux analysis	A method used to determine the area required for hindered settling based on an analysis of the solids (mass) flux.
Solids retention time (SRT)	The average period of time in which solids remain in a suspended growth process (also called sludge age).
Staged process	Processes which occur with more than one independent reactor or compartment in series.
Surface overflow rate	The hydraulic flowrate applied relative to the clarifier surface area ($\text{m}^3/\text{m}^2\cdot\text{d}$).
Suspended growth processes	Biological treatment processes in which microorganisms responsible for the conversion of organic matter or other constituents in the wastewater to gases and cell tissue are maintained in suspension within the liquid.
Volumetric organic loading rate	The amount of BOD or COD applied to the aeration tank volume per day (e.g., kg BOD or $\text{COD}/\text{m}^3\cdot\text{d}$).

The theory of biological wastewater treatment is presented and discussed in detail in Chap. 7. Biological treatment processes, as noted in Chap. 7, may be classified as aerobic and anaerobic suspended growth, attached growth, and various combinations thereof. The focus of this chapter is on suspended growth treatment processes as exemplified by the activated sludge process for BOD and nitrification and for nitrogen and phosphorus removal. Attached growth and combined processes are discussed in Chap. 9, and suspended and attached growth anaerobic processes are considered in Chap. 10. Included in this chapter are (1) introduction to the activated sludge process, (2) wastewater characterization, (3) fundamentals of process selection, design, and control, (4) selector types and design considerations, (5) use of simulation models for activated sludge process design considerations, (6) processes for BOD removal and nitrification, (7) processes for biological nitrogen removal, (8) processes for enhanced biological phosphorus removal, (9) aeration tank design for activated sludge processes, (10) analysis of liquid separation for activated sludge processes with clarifiers, (11) design considerations for secondarily clarifiers, and (12) solids separation for membrane bioreactors. Aerated lagoons, non-aerated lagoons, and stabilization ponds are not covered in this text, as they are used mainly for small rural communities where sufficient land is available and discharge requirements may not be as stringent as in urban areas. Detailed design information on aerated lagoons and stabilization ponds may be found in the 4th edition of this textbook (Tchobanoglous et al., 2003). Additional sources of information may be found in Crites and Tchobanoglous (1998) and Reed et al. (1995).

8–1 INTRODUCTION TO THE ACTIVATED SLUDGE PROCESS

To provide a basis for the process designs presented in the subsequent sections of this chapter, it will be useful to consider (1) a brief summary of the historical development of the activated sludge process, (2) a description of the basic process, (3) a brief review of the evolution of the activated sludge process, and (4) an overview of recent process developments.

Historical Development of Activated Sludge Process

The activated sludge process is now used routinely for the biological treatment of municipal and industrial wastewaters. The antecedents of the activated sludge process date back to the early 1880s in England, to the work of Dr. Angus Smith, who investigated the aeration of wastewater in tanks, which hastened the oxidation of the organic matter. The aeration of wastewater was studied subsequently by a number of investigators, and in 1910 Black and Phelps reported that a considerable reduction in putrescibility could be secured by forcing air into wastewater in basins. In experiments with aerated wastewater, conducted at the Lawrence Experiment Station during 1912 and 1913, Clark and Gage found that growths of organisms could be cultivated in bottles and in tanks partially filled with roofing slate spaced about 25 mm (1 in.) apart and that these growths greatly increased the degree of purification obtained (Clark and Adams, 1914).

The results of the work at the Lawrence Experiment Station, with respect to the treatment of wastewater, were so striking that knowledge of them led Dr. G. J. Fowler of the University of Manchester, England to suggest that experiments along similar lines be conducted at the Manchester Sewage Works where Ardern and Lockett carried out valuable research on the subject. During the course of their experiments, Ardern and Lockett found that the sludge played an important part in the results obtained by aeration, as announced in their paper of May 3, 1914 (Ardern and Lockett, 1914). The process was named *activated sludge* by Ardern and Lockett because it involved the production of an activated mass of microorganisms capable of aerobic stabilization of organic material in wastewater (Metcalf & Eddy, 1935).

Basic Process Description

By definition, the basic activated sludge treatment process, as illustrated on Figs. 8-1(a) and (b), consists of the following three basic components: (1) a reactor in which the microorganisms responsible for treatment are kept in suspension and aerated; (2) liquid-solids separation unit, usually in a sedimentation tank; and (3) a recycle system for returning solids removed from the liquid-solids separation unit back to the reactor. Numerous process configurations have evolved employing these components. An important feature of the activated sludge process is the formation of flocculent settleable solids that can be removed by gravity settling in sedimentation tanks. In most cases, the activated sludge process is employed in conjunction with physical and chemical processes that are used for the preliminary and primary treatment of wastewater (discussed in Chap. 5), and post treatment, including disinfection (Chap. 12), and possibly filtration (Chap. 11).

Historically, most activated sludge plants have been used to treat wastewaters that have been pretreated by primary sedimentation, as shown on Figs. 8-1(a) and (b). Primary sedimentation is most efficient at removing settleable solids, whereas the biological processes are essential for removing soluble, colloidal, and particulate (suspended) organic substances; for nitrification and denitrification; and for biological phosphorus removal. For applications such as treating wastewater from smaller-sized communities, primary treatment is often not used as more emphasis is placed on simpler and less operator-intensive treatment methods. Primary treatment is omitted frequently in areas of the world that have hot climates, where odor problems from primary tanks and primary sludge can be significant. For these applications, various modifications of conventional activated sludge processes are used, including sequencing batch reactors, oxidation ditch systems, and membrane bioreactors.

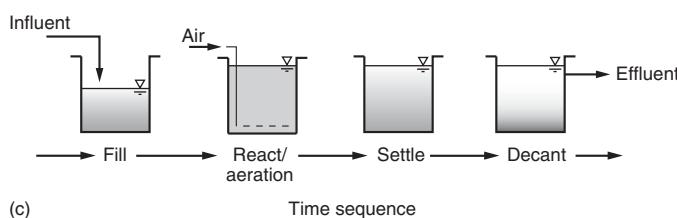
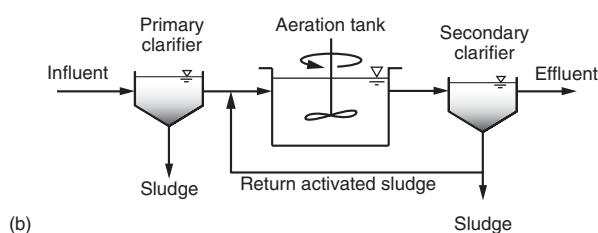
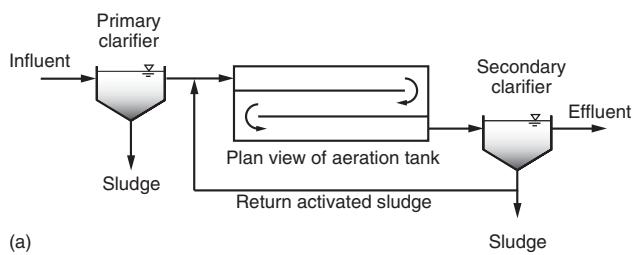


Figure 8-1

Typical activated sludge processes with different types of reactors: (a) schematic flow diagram of plug-flow process and view of plug-flow reactor, (b) schematic flow diagram of complete-mix process and view of complete-mix activated sludge reactor, and (c) schematic diagram of sequencing batch reactor process and view of sequencing batch reactor.

Evolution of the Conventional Activated Sludge Process

Prior to the 1980s, the principal objective of activated sludge process designs was aimed mainly at achieving a “secondary treatment” standard of 85 percent BOD and TSS removal. Since then, more emphasis has been placed on meeting more stringent discharge limits as well as the removal of nutrients (nitrogen and phosphorus). Thus, a number of activated sludge processes and design configurations have evolved in response to (1) the need for higher-quality effluents from wastewater treatment plants; (2) the need to remove nutrients; (3) increased discoveries and understanding of microbial processes and fundamentals; (4) technological advances in equipment, materials, electronics, and process control; and (5) the continual need to reduce capital and operating and energy costs for municipalities and industries. Many activated sludge processes used today and expected to be used in the future may incorporate nitrification, biological nitrogen removal, and/or biological phosphorus removal. Typically, reactors in series, operated under aerobic, anoxic, and anaerobic conditions are used. The general types of activated sludge processes used (i.e., plug flow, complete mix, and sequencing batch reactor), illustrated on Fig. 8-1, are considered in the following discussion.

Plug-Flow Process Configurations. Since the process came into common use in the early 1920s and up until the late 1970s, the type of activated sludge process used most commonly was the one in which a plug-flow reactor with large length to width ratios (typically $> 10:1$) was used [see Fig. 8-1(a)]. In considering the evolution of the activated sludge process, it is important to note that the discharge of industrial wastes to domestic wastewater collection systems increased in the late 1960s. The use of a plug-flow process became problematic when industrial wastes were introduced because of the toxic effects of some of the discharges.

Complete-Mix Process Configurations. The complete-mix reactor was developed, in part, because the larger volume allowed for greater dilution and thus mitigated the effects of toxic discharges. The more common type of activated sludge process in the 1970s and early 1980s tended to be single-stage, complete-mix activated sludge (CMAS) processes [see Fig. 8-1(b)], as advanced by McKinney (1962). For some nitrification applications, two-stage systems (each stage consisting of an aeration tank and clarifier) were used with the first stage designed for BOD removal, followed by a second stage for nitrification.

Comparing Plug Flow and Complete Mix Process Configurations. In comparing the plug-flow [see Fig. 8-1(a)] and complete-mix activated sludge (CMAS) [see Fig. 8-1(b)] processes, the mixing regimes and tank geometry are quite different. In the CMAS process, the mixing of the tank contents is sufficient so that ideally the concentrations of the mixed-liquor constituents, soluble substances (i.e., COD, BOD, $\text{NH}_4\text{-N}$), and colloidal and suspended solids do not vary with location in the aeration basin. The plug-flow process involves relatively long, narrow aeration basins, so that the concentration of soluble substances and colloidal and suspended solids varies along the reactor length. Although process configurations employing long, narrow tanks are commonly referred to as plug-flow processes, in reality, true plug flow does not exist. Depending on the type of aeration system, back mixing of the mixed liquor can occur and, depending on the layout of the reactor and the system reaction kinetics, nominal plug flow may be described more appropriately by the series of complete-mix reactors as discussed in Chap. 4.

Sequencing Batch Process Configuration. With the development of simple inexpensive program logic controllers (PLCs) and the availability of level sensors and automatically operated valves, the sequencing batch reactor (SBR) process [see Fig. 8-1(c)] became used more widely by the late 1970s, especially for smaller communities and industrial installations with intermittent flows. In recent years, however, SBRs are being used for larger cities. The SBR is a fill-and-draw type of reactor system involving a single complete-mix reactor in which all steps of the activated sludge process occur. Mixed liquor remains in the reactor during all cycles, thereby eliminating the need for separate sedimentation tanks.

Other Activated Sludge Processes. Other activated sludge processes that have found application, with their dates of major interest in parentheses, include the oxidation ditch (1950s), contact stabilization (1950s), Krause process (1960s), pure oxygen activated sludge (1970s), Orbal process (1970s), deep shaft aeration (1970s), and sequencing batch reactor process (1980).

Development of Selectors. Activated sludge process designs before and until the late 1970s generally involved the configurations shown on Figs. 8-1(a) and (b). These designs very often suffered from solids settling problems in the secondary clarifiers due to

the proliferation of filamentous-type bacteria. In the early 1980s researchers and practicing engineers advanced the concept of a “biological selector” in activated sludge design, which was first introduced in a patent by Davidson (1957) to select for good settling “floc-forming” activated sludge over filamentous bacteria. Selectors are smaller single or multi-staged aerated reactors in front of the main activated sludge treatment aeration basin. The selector concept is also inherent in designs with single or multi-staged anoxic or anaerobic reactors before the main aeration tank to select for conditions for denitrification of nitrate/nitrite or for phosphorus-storing bacteria. The anoxic or anaerobic reactors also serve as selectors resulting in the development of good settling activated sludge. Selectors are considered in detail in Sec. 8–4.

Membrane Bioreactor Process Configuration. A membrane bioreactor (MBR) is an activated sludge system with membranes located at the end of the activated sludge basin(s) for liquid-solids separation in lieu of using secondary clarifiers (see Fig. 8–2). In the integrated MBR system shown on Fig. 8–2 the key component is the microfiltration or ultrafiltration membrane that is immersed directly into the activated sludge reactor. The membranes are mounted in modules (sometimes called cassettes) that can be lowered into the bioreactor. The modules are comprised of the membranes, support structure for the membranes, feed inlet and outlet connections, and an overall support structure. The membranes are subjected to a vacuum (less than 50 kPa) that

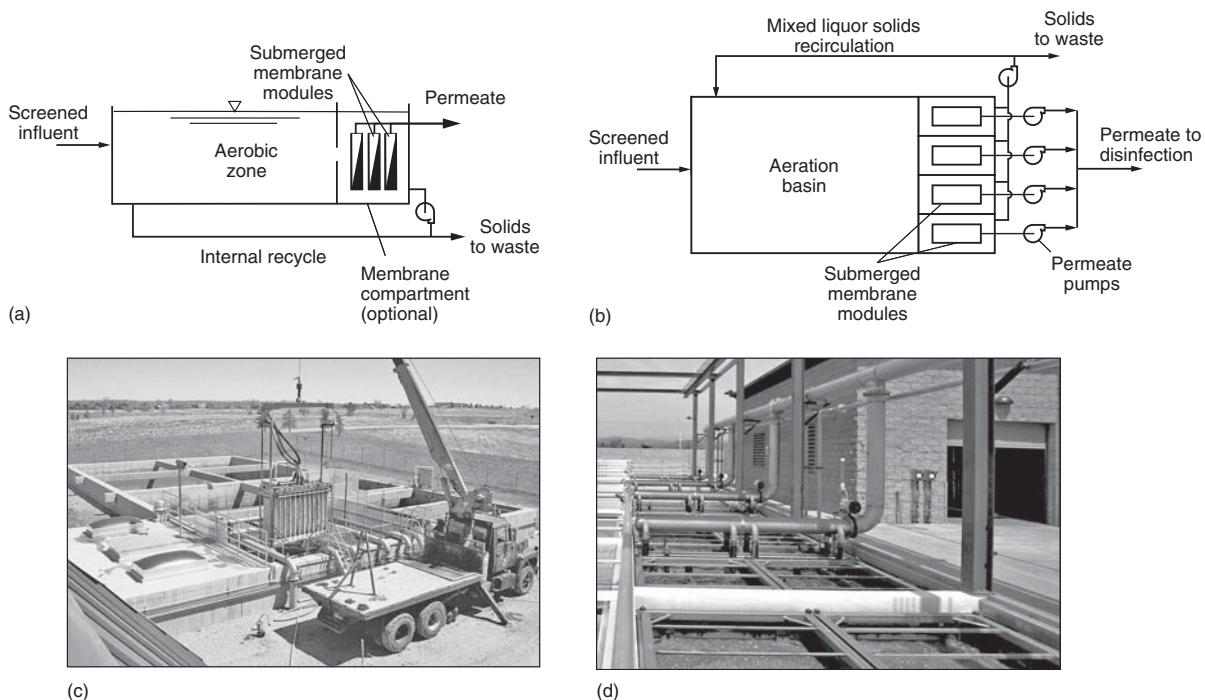


Figure 8–2

Membrane bioreactor (MBR). A multi-staged activated sludge system with membranes for liquid-solids separation: (a) section through MBR with separate compartment for the membranes, (b) plan view of MBR, (c) view of membrane cassettes being placed in separate compartment, and (d) view of separate membrane compartment.

important constituents, such as BOD, TSS/VSS, nitrogen compounds, and phosphorus should be done to account for all contributing flows and loads to the activated sludge process. The separate treatment of return flows is considered in Chap. 15.

8-3 FUNDAMENTALS OF PROCESS SELECTION, DESIGN, AND CONTROL

The purpose of this section is to introduce (1) overall considerations in treatment process implementation, (2) important factors in process selection and design, (3) process control issues, (4) operational problems associated with the activated sludge secondary clarifier process, and (5) operational problems associated with the MBR process. The information presented in this section is applied to the analysis and design of alternative activated sludge processes in the remainder of this chapter. Many of the equations presented in this chapter were derived previously in Chap. 7 and are summarized in this section for convenient reference.

Overall Considerations in Treatment Process Implementation

The selection of an activated sludge treatment process is always based on a review a number of local factors that will govern the final choice. The principal factors that must be considered are summarized in Table 8-4. The relative importance of the factors presented in Table 8-4 is site specific. Current and future treatment needs are typically driven by regulatory requirements with regard to the impact of the point discharge to surface or groundwater or reclaimed water quality. Wastewater characteristics were considered previously in Sec. 8-2. Flowrates and their variation are discussed in Chap. 3. Local environmental conditions, space constraints, and costs are site-specific. Energy considerations are discussed in Chap. 17. Important factors in the selection of a specific activated sludge process are discussed in this section.

Important Factors in Process Selection and Design

In the selection and design of the activated sludge process, consideration must be given to (1) the type of activated sludge process and reactor configuration, (2) applicable kinetic relationships, (3) solids retention time and loading, (4) sludge production rate, (5) oxygen demand rate and transfer, (6) nutrient requirements, (7) other chemical requirements, (8) activated sludge settling characteristics, (9) liquid-solids separation of mixed liquor, and (10) effluent characteristics.

Selection of Activated Sludge Process and Reactor Configuration. The many different types of activated sludge processes that can be selected are dependent on treatment needs required to meet effluent discharge limits. In general, the reactor types employed are plug flow, complete-mix, and batch (e.g., sequencing batch reactor). Regardless of the type of reactor or reactor combination used, a critical element in the performance of the various activated sludge processes that employ secondary clarification for liquids-solids separation is the settleability of the mixed liquor suspended solids (MLSS). In turn, the settleability of the MLSS depends on the nature of the microorganisms that comprise the MLSS. Occasionally, a proliferation of filamentous bacteria can occur. When a proliferation of filamentous bacteria occurs, the MLSS biological flocs do not settle well, which can result in a high solids levels in the secondary clarifiers and the loss of solids in the final clarifier overflow. The term *bulking sludge* is used to describe the poor settling sludge. Bulking sludge is considered further in Sec. 8-4 along with another type of bulking, known as *viscous bulking*.

Table 8-4**General considerations for the selection of the type of suspended growth reactor**

Factor	Description
Treatment needs	Treatment requirements and process selection can be categorized according to effluent discharge water quality needs, which may range from secondary treatment for BOD removal, nitrification to achieve low effluent ammonia concentration, anoxic-aerobic processes to provide nitrogen removal, and anaerobic-anoxic-aerobic processes to provide nitrogen and phosphorus removal.
Future treatment needs	Potential future treatment needs can have an impact on present process selection. For example, if water reuse is anticipated in the future, the process selection should favor designs that can easily accommodate nitrogen removal and effluent filtration.
Sludge settleability	Activated sludge selector designs can be used that control filamentous bacteria growth that leads to poor sludge settling and thickening in secondary clarifiers. Some selector designs are inherent in nitrogen and phosphorus removal processes.
Effect of reaction kinetics	Both completely-mixed and plug flow reactor configurations with similar volumes have been commonly used for BOD removal designs as both require a minimum SRT to provide acceptable sludge settling properties. Staged-reactors or plug flow designs can exploit reaction kinetic advantages for nitrification or preanoxic tanks to result in less volume than that for a single completely-mixed tank. Such designs require that the aeration equipment provides a high enough oxygen transfer rate in the first stage or at the front of a plug-flow tank to meet the oxygen demand for BOD removal and nitrification. The aeration equipment design must account for different oxygen demand rates along the length of the aeration tank. The oxygen demand is less variable and lower in completely-mixed tanks.
Wastewater characteristics	Wastewater characteristics are affected by contributions from domestic and industrial sources and inflow/infiltration flows. Large variations in wastewater concentrations due to wet weather or seasonal loads can affect process selection. Wastewater alkalinity and pH are also important for nitrification and enhance biological phosphorus removal processes.
Local environmental conditions	Temperature is an important environmental condition that affects treatment performance and lower rates occur at lower temperatures. The size of the facility and plant staffing are also important and smaller plants with less staffing favor processes that are simpler to operate and are more robust to influent wastewater variations. Concerns for aesthetics for facilities with close neighbors can affect process selection.
Toxic or inhibitory substances	Industrial pretreatment standards and enforcement provide substantial protection against biological process upsets from toxic or inhibitory substances disposed into the collection system. If potential exists for shock industrial toxic loads, completely-mixed activated sludge processes with greater design safety factors are considered.
Space	Space limitations for new or existing plant retrofits often limits the candidate processes that can be considered. Membrane bioreactors, integrated fixed film activated sludge, and biological aerated filter processes are good candidates for limited space.
Cost	Construction and operating costs are very important considerations in selecting the type and size of biological reactors. Because the associated settling facilities are an integral part of the activated sludge process, the selection of the reactor and the solids separation facilities must be considered as a unit.

Prior to the 1970s, filamentous bulking was considered an inevitable consequence of activated sludge treatment, but work by Chudoba et al. (1973) with staged versus complete-mix activated sludge reactors led to the concept that reactor configuration designs, now termed selectors, could be used to control filamentous bulking and improve sludge-settling characteristics. Because of their impact on the operation of the activated sludge process, the use of selectors is now a common design element of the activated sludge process. Selector types and designs are considered in detail in Sec. 8-4.

Kinetic Relationships. As developed in Chap. 7, kinetic relationships are used to determine biomass growth and substrate utilization rates, and to define process performance. The derivation of important kinetic relationships may be found in Chap. 7 and their application is demonstrated for various designs in this chapter.

Selection of Solids Retention Time and Loading Criteria. Certain design and operating parameters distinguish one activated sludge process from another. The common parameters used are the solids retention time (SRT), the food to biomass (F/M) ratio (also known as food to microorganism ratio), and the volumetric organic loading rate. While the SRT is the basic design and operating parameter, the F/M ratio and volumetric loading rate values are useful for comparison to historical data and typical observed operating conditions. The F/M and volumetric loading parameters are described in Chap. 7.

Solids Retention Time. The SRT, in effect, represents the average period of time during which the sludge has remained in the system. As presented previously in Chap. 7, SRT is the most critical parameter for activated sludge design and operation as SRT affects the treatment process performance, aeration tank volume, sludge production, and oxygen requirements. For BOD removal, SRT values generally range from 3 to 5 d, depending on the mixed-liquor temperature. At 18 to 25°C an SRT value close to 3 d is desired where only BOD removal is required and to discourage nitrification and eliminate the associated oxygen demand. To limit nitrification, some activated sludge plants have been operated at SRT values of 1 d or less. At 10°C, SRT values of 5 to 6 d are common for BOD removal only. Temperature and other factors that affect SRT in various treatment applications are summarized in Table 8-5.

Table 8-5
Typical minimum SRT ranges for activated sludge treatment^a

Treatment goal	SRT range, d	Factors affecting SRT
Removal of soluble BOD in domestic wastewater	1–2	Temperature
Conversion of particulate organics in domestic wastewater	2–5	Temperature
Develop flocculent biomass for treating domestic wastewater	2–3	Temperature
Provide complete nitrification	3–18	Temperature/inhibitory substances
Biological phosphorus removal	2–4	Temperature
Aerobic digestion of waste activated sludge	20–40	Temperature
Degradation of xenobiotic compounds	5–50	Temperature/specific bacteria/compounds

^a SRT is based on aerobic volume.

SRT Values for Nitrification. Because nitrification performance is temperature-dependent, the design SRT for nitrification must be selected with caution as variable nitrification growth rates have been observed at different sites, presumably due to the presence of inhibitory substances (Barker and Dold, 1997; Fillos et al., 2000). For desktop nitrification designs, which are based on constant influent flow and TKN concentration, a safety factor is used to increase the SRT above that calculated from nitrification kinetics and the required effluent NH₄-N concentration. A factor of safety is used for two reasons: (1) to allow flexibility for operational variations in controlling the SRT, and (2) to provide for additional nitrifying bacteria to handle peak TKN loadings. The influent TKN concentration and mass loading can vary throughout the day (a peak to average TKN loading of 1.3 to 1.5 is not unusual, depending on plant size) and can also be affected by return flows from digested and dewatered biosolids processing. By increasing the design SRT, the inventory of nitrifying bacteria is increased to meet the NH₄-N concentration at the peak load so that the effluent NH₄-N concentration requirement is achieved.

Typical SRT Factors of Safety for Nitrification. Typically, the value of the factor of safety is equal to the peak/average TKN load. Because use of the peak/average TKN load is conservative, the NH₄-N concentration during the normal loading period will be lower with the net effect of a composite effluent NH₄-N concentration that is somewhat lower than the design goal. Dynamic simulation models can be used to optimize the design SRT value to meet target effluent NH₄-N concentrations, subject to changing influent flow and TKN concentrations (Barker and Dold, 1997). The steady-state solution approach described in Sec. 8–6 has resulted in reasonable designs, and can provide a starting point for using simulation models to analyze and design activated sludge nitrification processes.

Sludge Production. The design of the sludge-handling and disposal/reuse facility depends on the prediction of sludge production for the activated sludge process. If the sludge-handling facilities are undersized, treatment process performance may be compromised. Sludge will accumulate in the activated sludge process if it cannot be processed fast enough by an undersized sludge-handling facility. Eventually, the sludge inventory capacity of the activated sludge system will be exceeded and excess solids will exit in the secondary clarifier effluent, potentially violating TSS discharge limits. The sludge production relative to the amount of BOD removed also affects the aeration tank size. Two methods are used to determine sludge production as a function of SRT. The first method is based on an estimate of an observed sludge production yield from published data for domestic water, and the second is based on wastewater characterization information with consideration to the various sources of sludge production.

Sludge Production Based on Observed Yield. The use of observed yield is often satisfactory for determining an initial activated sludge process design and for estimating sludge production rate. The quantity of sludge produced daily (and thus wasted daily at steady state) can be estimated using Eq. (8–19). For a given wastewater, the Y_{obs} value will vary depending on whether the substrate is defined as BOD, bCOD, or COD.

$$P_{X,\text{VSS}} = Y_{\text{obs}}(Q)(S_o - S)(1 \text{ kg}/10^3 \text{ g}) \quad (8-19)$$

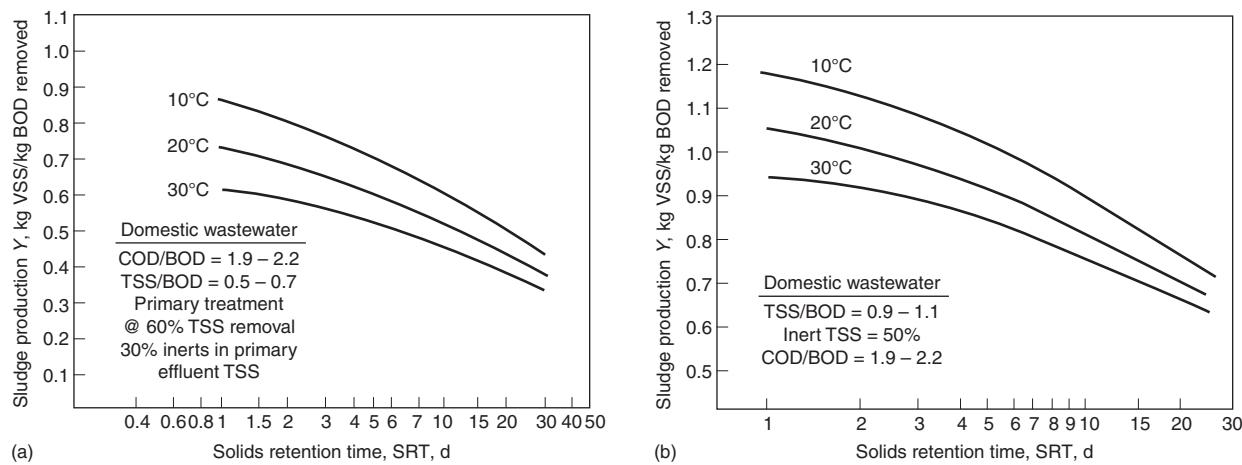
where $P_{X,\text{VSS}}$ = net waste activated sludge produced each day, kg VSS/d

Y_{obs} = observed yield, g VSS/g substrate removal

Q = influent flowrate, m³/d

S_o = influent substrate concentration, mg/L

S = effluent substrate concentration, mg/L

**Figure 8-7**

Net solids production as a function of solids retention time (SRT) and temperature: (a) with primary treatment and (b) without primary treatment.

Observed volatile suspended solids yield values, based on BOD, are illustrated on Fig. 8-7. The observed yield decreases as the SRT is increased due to biomass loss by more endogenous respiration. The yield is lower with increasing temperature as a result of a higher endogenous respiration rate at higher temperature. The yield is higher when no primary treatment is used as more nbVSS remains in the influent wastewater. The temperature correction value θ for endogenous respiration [see Eq. (2-25)] is 1.04 between 20 and 30°C, and 1.12 between 10 and 20°C. A θ value of 1.04 has been adopted in this text for the temperature effect on endogenous decay.

Sludge Production Based on Wastewater Characteristics. With sufficient wastewater characterization, a more accurate prediction of sludge production can be made. The following equation, based on Eq. (7-54) in Chap. 7, accounts for the heterotrophic biomass growth, cell debris from endogenous decay, nitrifying bacteria biomass, and nonbiodegradable volatile suspended solids and can be used to estimate sludge production. The subscripts H and n are used to distinguish the synthesis yield and decay coefficients between heterotrophic bacteria and nitrifying organisms.

$$\begin{aligned}
 P_{X,VSS} = & \frac{QY_H(S_o - S)(1 \text{ kg}/10^3 \text{ g})}{1 + b_H(\text{SRT})} + \frac{(f_d)(b_H)QY_H(S_o - S)\text{SRT}(1 \text{ kg}/10^3 \text{ g})}{1 + b_H(\text{SRT})} \\
 & \quad \text{(A)} \qquad \qquad \qquad \text{(B)} \\
 & \quad \text{Heterotrophic} \qquad \qquad \qquad \text{Cell} \\
 & \quad \text{Biomass} \qquad \qquad \qquad \text{debris} \\
 & + \frac{QY_n(\text{NO}_x)(1 \text{ kg}/10^3 \text{ g})}{1 + b_n(\text{SRT})} + Q(\text{nbVSS})(1 \text{ kg}/10^3 \text{ g}) \\
 & \quad \text{(C)} \qquad \qquad \qquad \text{(D)} \\
 & \quad \text{Nitrifying} \qquad \qquad \qquad \text{Nonbiodegradable} \\
 & \quad \text{bacteria} \qquad \qquad \qquad \text{VSS in influent}
 \end{aligned} \tag{8-20}$$

where NO_x = concentration of $\text{NH}_4\text{-N}$ in the influent flow that is nitrified, mg/L

b_n = endogenous decay coefficient for nitrifying organisms, g VSS/g VSS·d

Other terms as defined previously.

The total mass of dry solids wasted/day includes TSS and not just VSS. The TSS includes the VSS plus inorganic solids. Inorganic solids in the influent wastewater ($TSS_o - VSS_o$) contribute to inorganic solids and are an additional solids production term that must be added to Eq. (8–20). The biomass terms in Eq. (8–20) (A, B, and C) contain inorganic solids and the VSS fraction of the total biomass is about 0.85, based on the cell composition given in Table 7–4. Thus, Eq. (8–20) is modified as follows to calculate the solids production in terms of TSS:

$$P_{X,TSS} = \frac{A}{0.85} + \frac{B}{0.85} + \frac{C}{0.85} + D + Q(TSS_o - VSS_o) \quad (8-21)$$

(E)
Influent Inert TSS

where TSS_o = influent wastewater TSS concentration, mg/L

VSS_o = influent wastewater VSS concentration, mg/L

The daily mass of solids in the aeration tanks is determined from the SRT. The daily sludge production can be computed by Eqs. (7–56) and (7–57).

$$(X_{VSS})(V) = (P_{X,VSS}) SRT \quad (7-56)$$

$$(X_{TSS})(V) = (P_{X,TSS}) SRT \quad (7-57)$$

By selecting an appropriate MLSS concentration, the aeration volume can be determined using Eq. (7–57). Concentrations of MLSS selected range from 1200 to 4000 mg/L, but must be compatible with the sludge settling characteristics and clarifier design as discussed later in Sec. 8–10.

Oxygen Requirements. The oxygen required for the biodegradation of carbonaceous material is for the amount of bCOD oxidized to provide energy during bCOD consumption for cell synthesis plus the oxygen consumed during the endogenous respiration by the biomass produced. The calculation of the oxygen required for a completely mixed activated sludge system [see Fig. 8–8(a)] involves a simple mass balance on the bCOD removal across the system in which the bCOD removed is equal to the oxygen used plus the

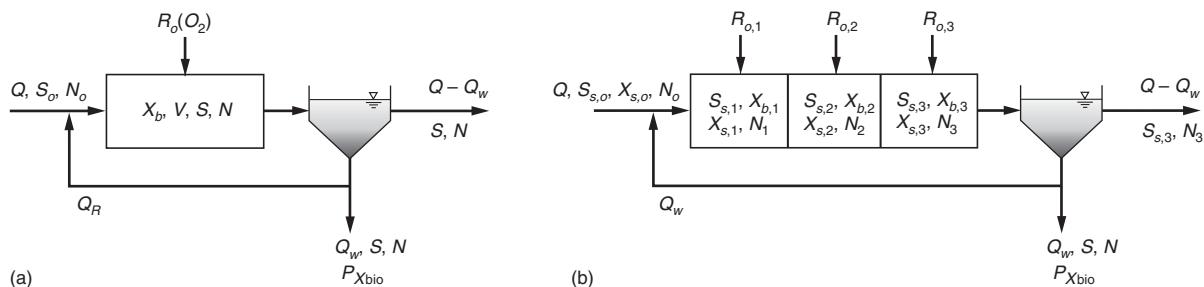


Figure 8–8

Schematic showing components used in oxygen demand rate analysis for completely-mixed activated sludge and staged activated sludge processes: (a) single stage reactor and (b) reactors in series.

bCOD in the excess biomass produced. The biomass produced ($P_{x,\text{bio}}$) is component A plus B in Eq. 8–20 and thus the total oxygen required, R_o , is as follows as shown in Chap. 7:

$$R_o = Q(S_o - S) - 1.42P_{x,\text{bio}} \quad (7-61)$$

$$R_o = Q(S_o - S) - 1.42 \left[\frac{QY_H(S_o - S)}{1 + b_H(\text{SRT})} + \frac{f_d(b_H)QY_H(S_o - S)\text{SRT}}{1 + b_H(\text{SRT})} \right] \quad (8-22)$$

As an approximation, for BOD removal only, the oxygen requirement will vary from 0.90 to 1.3 kg O₂/kg BOD removed for SRTs from 5 to 20 d, respectively (WEF, 2010).

Oxygen Required for Nitrification. When nitrification is included in the process, the total oxygen requirements will include the oxygen required for removal of carbonaceous material plus the oxygen required for ammonia and nitrite oxidation to nitrate (see Sec. 7–9 in Chap. 7) as follows:

$$R_o = Q(S_o - S) - 1.42 P_{x,\text{bio}} + 4.57 Q(\text{NO}_x) \quad (8-23)$$

where R_o = total oxygen required, g/d

$P_{x,\text{bio}}$ = biomass as VSS wasted, g/d [parts A, B, and C of Eq. (8–20)]

NO_x = Amount of NO₃-N produced from nitrification of NH₄-N, g/m³

Other terms as defined previously.

As shown in Eq. (8–23), NO_x is the amount of TKN oxidized to nitrate. A nitrogen mass balance for the system that accounts for the influent TKN, nitrogen removed for biomass synthesis, and unoxidized effluent nitrogen is done to determine NO_x. Unless a careful wastewater characterization study is done to determine the nonbiodegradable particulate and soluble nitrogen (nbpON and nbsON), these components are ignored. Ignoring these terms results in predicting a slightly higher NO_x concentration (5 to 15 percent) and a more conservative oxygen requirement estimate using Eq. (8–23). The nitrogen mass balance, based on the assumption that biomass (which can be represented by the formula C₅H₇NO₂) contains 0.12 g N/g biomass, is as follows:

$$\begin{aligned} \text{Nitrogen} &= \text{nitrogen in} - \text{nitrogen in} - \text{nitrogen in} \\ \text{Oxidized} &\quad \text{influent} \quad \text{effluent} \quad \text{cell mass} \\ Q(\text{NO}_x) &= Q(\text{TKN}_o) - QN_e - 0.12P_{x,\text{bio}} \end{aligned}$$

$$\text{NO}_x = \text{TKN} - N_e - 0.12P_{x,\text{bio}}/Q \quad (8-24)$$

where NO_x = nitrogen oxidized, mg/L

TKN_o = influent TKN concentration, mg/L

N_e = effluent NH₄-N concentration, mg/L

Other terms as defined previously.

Equation (8–24) can be solved for the NO_x concentration by estimating the effluent NH₄-N concentration from the nitrification process design.

Oxygen Required for Staged Systems. For activated sludge systems with aerobic reactors in series, the calculation for the oxygen requirement in each reactor is different than above and more complex. The oxygen requirement is highest in the first stage of activated sludge systems with reactors in series [see Fig. 8–8(b)] and decreases in subsequent stages. The amount needed in each stage is a function of (1) the rate of soluble and particulate bCOD removed, (2) the rate of NH₄-N oxidized, and (3) the rate of oxygen used for

endogenous respiration. For example, the R_o for reactor 2 shown on Fig. 8–8(b) is as follows:

$$R_{o,2} = (Q + Q_R)(1 - Y_H)[(S_{s,1} - S_{s,2}) + (X_{s,1} - X_{s,2})] + (Q + Q_R)4.57 (\text{NO}_2 - \text{NO}_1) + 1.42b_H(X_{b,2})V_2 \quad (8-25)$$

where $R_{o,2}$ = oxygen demand rate in reactor 2, g/d

S_s = soluble bCOD concentration, g/m³

X_s = particulate bCOD concentration, g/m³

Y_H = synthesis yield, g biomass COD/g bCOD removed

NO = $\text{NO}_3\text{-N}$ concentration, g/m³

X_b = biomass concentration, g VSS/m³

V = reactor volume, m³

Q_R = return activated sludge recycle flowrate, m³/d

The oxygen demand for each stage cannot be determined without first calculating the soluble and particulate bCOD, biomass and the $\text{NO}_3\text{-N}$ concentration in each stage (assumes that no appreciable $\text{NO}_2\text{-N}$ is present). These values can be obtained from mass balance equations for each of these constituents, but simulation models, discussed in Sec. 8–4, provide a more efficient solution method.

As an estimate for the design of a 3-stage activated sludge system treating domestic wastewater, Eq.(8–23) can be used to estimate the total oxygen demand rate for the system. Then a distribution of that demand as 60, 25, and 15 percent for reactors 1, 2, and 3, respectively, may be used.

The oxygen demand rate for an activated sludge aeration basin will vary during the day due to diurnal loading changes. The hourly rate can be 1.3 to 1.8 times the rate based on the average daily loading depending on the magnitude of the diurnal changes in influent BOD and TKN concentrations and flowrate.

Nutrient Requirements. If a biological system is to function properly, nutrients must be available in adequate amounts. As discussed in Chaps. 2 and 7, the principal nutrients are nitrogen and phosphorus. Using the formula $C_5H_7NO_2$, for the composition of cell biomass, about 12.4 percent by weight of nitrogen will be required. The phosphorus requirement is 1.5 to 2.0 percent by weight of the cell biomass. These are typical values, not fixed quantities, because it has been shown that the percentage distribution of nitrogen and phosphorus in cell tissue varies with the system SRT and environmental conditions. The amount of nutrients required can be estimated based on the daily biomass production rate [terms A, B, and C in Eq. (8–21)]. It should be noted that nutrient limitations can occur when the concentrations of inorganic nitrogen as N and orthophosphate as P are less than 0.1 mg/L (de Barbadillo et al., 2006). As a general rule, for SRT values greater than 7 d, about 5 g nitrogen and 1 g phosphorus will be required per 100 g of BOD to provide an excess of nutrients.

Other Chemical Requirements. In addition to the nutrient requirements, alkalinity is a major chemical requirement needed for nitrification. The amount of alkalinity required for nitrification, taking into account cell growth, is about 7.14 g CaCO_3 /g $\text{NH}_4\text{-N}$ [see Eq. (7–91) in Chap. 7]. In addition to the alkalinity required for nitrification, additional alkalinity must be available to maintain the pH in the range from 6.8 to 7.4. Typically the amount of residual alkalinity required to maintain pH near a neutral point (i.e., pH ~ 7) is between 70 and 80 mg/L as CaCO_3 , based on the equilibrium relationship between the gas phase CO_2 , bicarbonate alkalinity, and pH.

Liquid-Solids Separation of Mixed Liquor. Liquid-solids separation of the activated sludge mixed liquor and return of activated sludge to the activated sludge treatment reactors is critical to the process function and performance. The two methods used for liquid-solids separation are gravity settling in secondary clarifiers and membrane separation. Both methods provide solids thickening for return activated sludge flows and sludge wasting, but differ in the quality of the secondary treatment effluent TSS concentration. With well designed biological nutrient removal facilities secondary clarifier effluent TSS concentrations may range from 4 to 10 mg/L. The effluent from a membrane separation system is termed *permeate* and has no measurable TSS concentration after the liquid passes through the membrane separation unit which have membrane pore sizes of about 0.02 or 0.40 μm , depending on the choice of membranes. Each method has issues with regard to the effect of activated sludge mixed liquor characteristics on performance and design. Specific information on the design of secondary clarifiers and membrane separation is presented in Secs. 8–10 and 8–12, respectively.

Effluent Characteristics. The major parameters of interest that determine the effluent quality from biological treatment processes consist of organic compounds, suspended solids, and nutrients as indicated in Table 8–6. The biodegradable soluble organic concentration is minimal in effluents from systems with complete nitrification and SRT values above 5.0 d, and is likely within the detection limit of the BOD test, which is about 2.0 mg/L. Most of the BOD is in particulate form associated with biomass contained in the effluent VSS concentration.

Table 8-6
Effluent characteristics for biological wastewater treatment processes

Category	Characteristics	Key constituents
Soluble COD	Biodegradable	Remaining influent bsCOD Metabolic intermediates bsCOD from cell lysis
	Nonbiodegradable	Influent nbsCOD Metabolic products Nonbiodegradable colloids
Particulate COD	Biodegradable	Biomass VSS Non-captured influent VSS
	Nonbiodegradable	Biomass cell debris Non-captured influent nbVSS
Nitrogen	Inorganic	$\text{NH}_4\text{-N}$, $\text{NO}_3\text{-N}$, $\text{NO}_2\text{-N}$
	Organic	Dissolved organic N Particulate organic N in VSS
Phosphorus	Inorganic	$\text{PO}_4\text{-P}$
	Organic	Dissolved organic P Particulate organic P in VSS

With a proper secondary clarifier design and good settling sludge, the effluent suspended solids will typically be in the range of 4 to 10 mg/L. Assuming a sBOD of 2.0 mg/L, a VSS/TSS ratio of 0.85, and an effluent TSS of 6 mg/L, the final effluent BOD concentration, BOD_e, can be estimated as follows:

$$\text{BOD}_e = \text{sBOD} + \left(\frac{0.60 \text{ g BOD}}{\text{g UBOD}} \right) \left(\frac{1.42 \text{ g UBOD}}{\text{g VSS}} \right) \left(\frac{0.85 \text{ g VSS}}{\text{g TSS}} \right) (\text{TSS, mg/L}) \quad (8-26)$$

$$\text{BOD}_e = 2 \text{ mg/L} + (0.60)(1.42)(0.85)(6 \text{ mg/L})$$

$$\text{BOD}_e = 10.2 \text{ mg/L}$$

Because the MBR processes produce an effluent with non-detectable TSS, the BOD concentration is also minimal.

Effluent nitrogen contains inorganic and organic forms. The effluent dissolved organic nitrogen (DON) may range from 0.5 to 2.0 mg/L (Urgun-Demirtas et al., 2007), and thus can be a very significant fraction of the effluent total nitrogen concentration for system confronted with permits with stringent effluent TN concentrations (typically below 3.0 mg/L).

Process Control

To maintain high levels of treatment performance with the activated sludge process under a wide range of operating conditions, special attention must be given to process control. The principal approaches to process control are (1) maintaining a target SRT, (2) maintaining target dissolved oxygen levels in the aeration tanks, and (3) regulating the return activated sludge (RAS) flowrate. The waste activated sludge (WAS) rate is selected to meet the SRT. The SRT is the most common control parameter, but in some cases the WAS rate is adjusted to meet a target MLSS concentration. Thus, higher WAS rates result in a lower SRT and vice versa. Return activated sludge is important in maintaining the MLSS concentration and controlling the sludge blanket level in the secondary clarifier. Aeration tank oxygen uptake rates (OURs) are useful for understanding the process operating conditions and oxygen transfer requirements, and in some cases, have been used in process control algorithms. Routine microscopic observations are important for monitoring the microbial characteristics and for early detection of changes that might negatively impact sludge settling and process performance.

SRT Control. To maintain a given SRT, the excess activated sludge produced each day must be wasted. The most common practice for both activated sludge/secondary clarifier and MBR systems is to waste sludge from the return sludge line because RAS is more concentrated and requires smaller waste sludge pumps than if wasting was done by removing mixed liquor from the aeration tank. In some activated sludge/secondary clarifier applications, such as for smaller flow systems with minimal operating staff, WAS is taken directly from the aeration tank. Though this method requires a larger wasting volume, advantages are that the waste solids concentration is more uniform and the SRT control can be based on volumetric wasting without the need for measuring suspended solids in the aeration tank, effluent and return sludge. The waste sludge may be discharged to various types of thickening processes alone or combined with sludge removed from primary clarification tanks. The actual amount of liquid that must be pumped to achieve SRT process control depends on the method used and the location from which the wasting is to be accomplished.

Wasting from Return Line. For example, if SRT is used for process control and wasting is from the return sludge line, the wasting rate can be computed by modifying the terms of Eq. (7-31) (see Table 8-10).

$$SRT = \frac{VX}{(Q_w X_R + Q_e X_e)} \quad (8-27)$$

where V = volume of the reactor, m^3

X = aeration tank solids concentration, mg/L

Q_w = waste sludge flowrate from the return sludge line, m^3/d

X_R = concentration of sludge in the return sludge line, mg/L

Q_e = effluent flowrate from the secondary clarifier, m^3/d

X_e = effluent TSS concentration, mg/L

The daily manual wasting flowrate out of the RAS line is then

$$Q_w = \frac{VX}{X_R(SRT)} - \frac{Q_e X_e}{X_R} \quad (8-28)$$

If it is assumed that the concentration of solids in the effluent from the settling tank is low, then Eq. (8-28) reduces to

$$SRT \approx \frac{VX}{Q_w X_R} \quad (8-29)$$

and

$$Q_w \approx \frac{VX}{X_R(SRT)} \quad (8-30)$$

At higher operating SRT values, the effect of solids loss in the effluent flow on the SRT is less significant. To determine the waste flowrate using Eq. (8-30), the solids concentration in both the aeration tank and the return line must be measured.

Wasting from Aeration Tank. If wasting is done from the aeration tank and the solids in the settled effluent are again neglected, then the rate of pumping can be estimated using the following relationship:

$$SRT = \frac{V}{Q_w} \quad (8-31)$$

or

$$Q_w = \frac{V}{SRT} \quad (8-32)$$

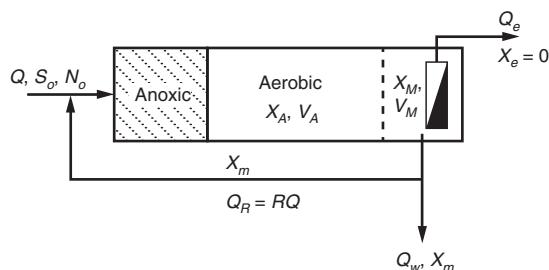
where Q_w = waste sludge flowrate from the aeration tank, m^3/d

Thus, the process may be controlled by daily wasting of a quantity of flow equal to the volume of the aeration tank divided by the SRT.

Wasting from MBR. The WAS flowrate needed to maintain a target aerobic SRT in an MBR system can be set strictly on the system aerobic volumes and return sludge recycle ratio. Such systems typically have an aerobic tank before the aerobic membrane

Figure 8–9

Schematic showing waste activated sludge flow from an anoxic/aerobic MBR.



liquid-solids separation tank (see Fig. 8–9). In some cases the aerobic tank prior to the membrane separation tank is referred to as a preaeration tank. The aerobic SRT is by definition:

$$\text{SRT} = \frac{X_A V_A + X_M V_M}{Q_W X_M} \quad (8-33)$$

where V_A = volume of preaeration tank(s) before membrane tank, m^3

V_M = volume of aerated membrane separation tank, m^3

X_A = solids concentration in preaeration tank, mg/L

X_M = solids concentration in membrane tank, mg/L

The solids concentration is thicker in the membrane tank due to the removal of permeate. The preaeration and membrane tank solids concentration can be related by mass balance, and ignoring the relatively small increase in solids concentration in the flow through the tank due to the removal of influent solids and bsCOD.

$$(RQ)X_M + Q(0) = (Q + RQ)X_A \quad (8-34)$$

and thus

$$X_A = \left(\frac{R}{1 + R} \right) X_M \quad (8-35)$$

substituting for X_A in Eq. (8–33) yields

$$\text{SRT} = \frac{\left(\frac{R}{1 + R} \right) V_A + V_M}{Q_W} \quad (8-36)$$

and

$$Q_W = \frac{\left(\frac{R}{1 + R} \right) V_A + V_M}{\text{SRT}} \quad (8-37)$$

Using Eq. (8–37), the daily WAS flowrate from an MBR system can be set for a given return activated sludge recycle ratio and SRT.

It should be noted that the SRT can be treated as an average value for a system operation and need not be maintained with an exact wasting rate each day. The SRT can be averaged over a time period equal to the target SRT value.

Dissolved Oxygen Control. Aeration equipment must be designed to supply the oxygen demand rate of the microorganisms in the activated sludge system under a wide range of flowrates and loads, while at the same time assuring that some minimum DO concentration is met. In DO control systems, the output of the aerator is adjusted to meet changing demands based on DO concentration measurements in the aeration tank. For example, when the influent BOD and ammonia load drops, the aeration tank DO concentration rises. The control system should be designed to sense DO concentration changes and makes appropriate adjustments to change the aeration rate to lower the DO to its target range so that energy is not wasted at lower loads. At higher loads the DO concentration will start to drop and the aeration output must be increased to keep the desired elevated DO concentration.

When aeration occurs at a lower DO concentration in the aeration basin, less energy is needed because of the higher driving force between the saturated DO concentration and the aeration basin DO concentration. However, if the DO concentration is too low, filamentous organisms may predominate and the settleability and quality of the activated sludge may be poor. In general, the dissolved oxygen concentration in the aeration tank should be maintained at about 1.5 to 2.0 mg/L in all areas of the aeration tank. A minimum DO concentration of about 0.7 mg/L is required to initiate nitrification. Operation at DO concentrations below 1.0 mg/L can save energy and is done in some designs to provide simultaneous nitrification and denitrification. However, the aerobic biological reaction rates are lower, requiring greater tank volume. Higher DO concentrations (2.0 to 3.0 mg/L) provide a small additional increase in nitrification rates. Values of DO above 4 mg/L result in little or no improvement in performance, but do increase aeration costs significantly, and can potentially result in the growth of foaming organisms.

Return Activated Sludge Control. The purpose of the return of activated sludge is to maintain the necessary concentration of activated sludge in the aeration tank and to keep the sludge blanket in the secondary clarifier at a low safe depth. Sufficient return sludge pumping capacity is needed, along with sufficient clarifier depth (3.7 to 6.5 m), to maintain the blanket below the effluent weirs. Return sludge pumping rates of 50 to 75 percent of the average design wastewater flowrate are typical, and the design average capacity is typically 100 to 150 percent of the average design flowrate (recycle ratio of 1.0 to 1.5). The use of variable speed pump drives allows operation between 50 to 150 percent of the influent flowrate.

Impact of SVI on RAS. Higher return activated sludge (RAS) recycle ratios (4.0 to 6.0) are used in MBR systems (Fig. 8–9) to take advantage of the fact that the system can operate at higher MLSS concentrations (8000–12,000 mg/L) than for activated sludge/secondary clarifier systems. Clarifier thickener capabilities typically limit the return sludge concentrations to between 6000 and 12,000 mg/L.

When the activated sludge has good settling properties (see Fig. 8–10) thickening occurs readily in the clarifier and a wide range of RAS ratios are possible, while keeping the clarifier sludge blanket below 0.15 to 0.30 m. Sludges that settle well typically have a sludge volume index (SVI) value equal to or less than 120, although the exact value will vary from plant to plant. For poorer settling sludge, higher recycle rates may be needed. The SVI test is considered in greater detail in Sec. 8–10 which deals with the liquid-solids separation for activated sludge with clarifiers.

Relationship between RAS and MLSS. The relationship between the RAS ratio and aeration tank MLSS concentration can be evaluated by mass balance. A certain MLSS

Figure 8-10

Field test for determining the sludge volume index (SVI).



concentration is necessary for a given aeration tank volume and SRT as shown by Eq. (7-57). The appropriate boundaries for two mass-balance analyses are illustrated on Fig. 8-11. Assuming the sludge-blanket level in the settling tank remains constant and that the solids in the effluent from the settling tank are negligible, the mass balance around the settling tank shown on Fig. 8-11(a) is as follows:

$$\text{Accumulation} = \text{inflow} - \text{outflow}$$

$$0 = X(Q + Q_R) - Q_R X_R - Q_W X_R - Q_e X_e \quad (8-38)$$

where X = mixed-liquor suspended solids, mg/L

Q = secondary influent flowrate, m³/s

Q_R = return sludge flowrate, m³/s

X_R = return activated sludge suspended solids, mg/L

Q_W = waste activated sludge flowrate, m³/s

Q_e = effluent flowrate, m³/s

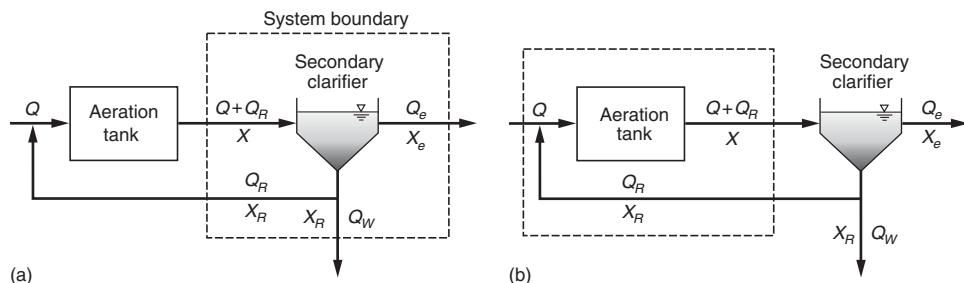
X_e = effluent suspended solids, mg/L

Assuming X_e is negligible and that $Q_W X_R$ is related to the SRT [Eq. (8-28)], solving Eq. (8-38) for Q_R yields

$$Q_R = \frac{[XQ - (XV/\text{SRT})]}{X_R - X} \quad (8-39)$$

Figure 8-11

Definition sketch for suspended solids mass balances for return sludge control: (a) secondary clarifier mass balance and (b) aeration tank mass balance.



The recycle ratio ($Q_R/Q = R$) is then

$$R = \frac{1 - (\tau/\text{SRT})}{(X_R/X) - 1} \quad (8-40)$$

The required RAS pumping rate can also be estimated by performing a mass balance around the aeration tank [see Fig. 8-11(b)]. The solids entering the tank will equal the solids leaving the tank if new cell growth can be considered negligible. At SRT above 8–10 d, this is a reasonable assumption. Solids enter the aeration tank in the return sludge and in the influent to the secondary process. However, if the influent solids are negligible compared to the MLSS, the mass balance around the aeration tank results in the following expression:

Accumulation = inflow – outflow

$$0 = X_R Q_R - X(Q + Q_R) \quad (8-41)$$

Solving for the return activated sludge ratio R yields X

$$Q_R/Q = R = \frac{X}{X_R - X} \quad (8-42)$$

Thus, based on Eqs. (8-40) and Eq. (8-42) for a given RAS solids concentration (X_R), a certain RAS ratio is needed to meet the required aeration tank solids concentration (X) for the target SRT. If the solids do not settle and thicken well in the clarifier, X_R is lower and a higher RAS ratio is needed. The SVI sludge settleability test can be used to approximate X_R for adjusting the RAS ratio:

$$X_R = \frac{1}{\text{SVI}} \left(\frac{1 \text{ g}}{1 \text{ mL}} \right) \left[\frac{(10^3 \text{ mg}/1 \text{ g})}{(1 \text{ L}/10^3 \text{ mL})} \right] = \frac{10^6}{\text{SVI}} \quad (8-43)$$

where X_R = estimated RAS concentration, mg/L

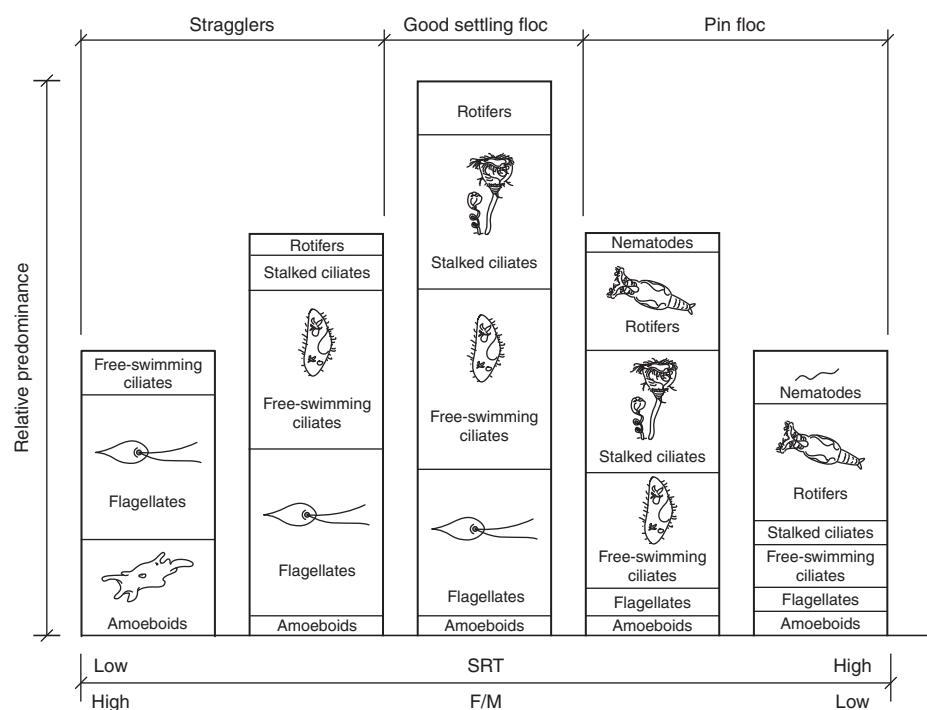
Sludge Blanket Level. Sludge blanket depth is a useful operational parameter to assess changes in the settling characteristics. Optimal depths usually range between 0.3 and 0.6 m (1 and 2 ft). The sludge blanket method of control requires considerable operator attention because of the diurnal flow and sludge production variations and changes in the settling characteristics of the sludge. The most common method of determining the blanket level is the use of a long tube core sampler.

Oxygen Uptake Rates. Microorganisms in the activated sludge process use oxygen as they consume the substrate. The rate at which they use oxygen, known as the oxygen uptake rate (OUR), is a measure of the biological activity due to the loading on the aeration tank. Values for the OUR are obtained by performing a series of DO measurements over a period of time with mixing but no aeration, and the measured results are reported conventionally as mg O₂/L·min or mg O₂/L·h. Oxygen uptake is most valuable for plant operations when combined with VSS data. The combination of OUR with MLVSS yields a value termed the specific oxygen uptake rate (SOUR) or respiration rate and is reported as mg O₂/g MLVSS·h. Changes in SOUR values may also be used to assess the presence of toxic or inhibitory substances in the influent wastewater or load changes.

Microscopic Observations. Routine microscopic observations provide valuable monitoring information about the condition of the microbial population in the activated sludge process. Specific information gathered includes changes in floc size and density, the status of filamentous organism growth, the presence of *Nocardioform* bacteria, and the

Figure 8-12

Relative predominance of microorganisms versus solids retention time (SRT) and F/M ratio. (Adapted in part from WEF, 1996.)



type and abundance of higher life-forms such as protozoans and rotifers. Changes in these characteristics can provide an indication of changes in the wastewater characteristics or of an operational problem. Examples of the changes in predominance of microorganisms versus F/M ratio and SRT are shown on Fig. 8-12. A decrease in the protozoan population may be indicative of DO limitations, operation at a lower SRT, or inhibitory substances in the wastewater. At very high SRT, small-diameter *pin floc* may develop to lead to a higher effluent suspended solids concentration. Early detection of filamentous or *Nocardioform* growth, as discussed below, will allow time for corrective action to be taken to minimize potential problems associated with excessive growth of these organisms.

Operational Problems in Activated Sludge Systems with Secondary Clarifiers

The most common problems encountered in the operation of an activated sludge/secondary clarifier plant are filamentous bulking sludge, viscous bulking sludge, *Nocardioform foaming*, and rising sludge. Because few plants have escaped these problems, it is appropriate to discuss their nature and possible plant operations responses.

Bulking Sludge. The issue of bulking sludge, which was introduced in Sec. 7-8, is always of concern for activated sludge/secondary clarifier systems. In extreme bulking sludge conditions, the sludge blanket cannot be contained and large quantities of MLSS are carried along with the clarifier effluent, potentially resulting in violation of permit requirements, inadequate disinfection, and clogging of effluent filters. The two principal types of sludge bulking problems are related to *filamentous bulking*, caused by the growth of filamentous organisms, and *viscous bulking*, caused by the presence of an excessive amount of extracellular biopolymer. The occurrence of both types of bulking, along with other forms of filamentous bulking, is considered in the following discussion. However,

Table 8-7

Filamentous bacteria found in activated sludge and associated process conditions^a

Filament type identified	Cause of filament growth
<i>Sphaerotilus natans</i> , <i>Halsicomenebacter hydrossis</i> , <i>Microthrix parvicella</i> , type 1701.	Low dissolved oxygen concentration
<i>M. parvicella</i> , types 0041, 0092, 0675, 1851	Low F/M
<i>H. hydrossis</i> , <i>Nocardia spp.</i> , <i>Nostocoida limicola</i> , <i>S. natans</i> , <i>Thiothrix spp.</i> , types 021N, 0914	Complete mix reactor conditions
<i>Beggiatoa</i> , <i>Thiothrix spp.</i> , types 021N, 0914	Septic wastewater/sulfide available
<i>S. natans</i> , <i>Thiothrix spp.</i> , type 021N, possible <i>H. hydrossis</i> , types 0041, 0675	Nutrient deficiency
Fungi	Low pH

^a From Eikelboom (1975).

before discussing the occurrence of bulking, it will be helpful to consider the general characteristics of filamentous bacteria.

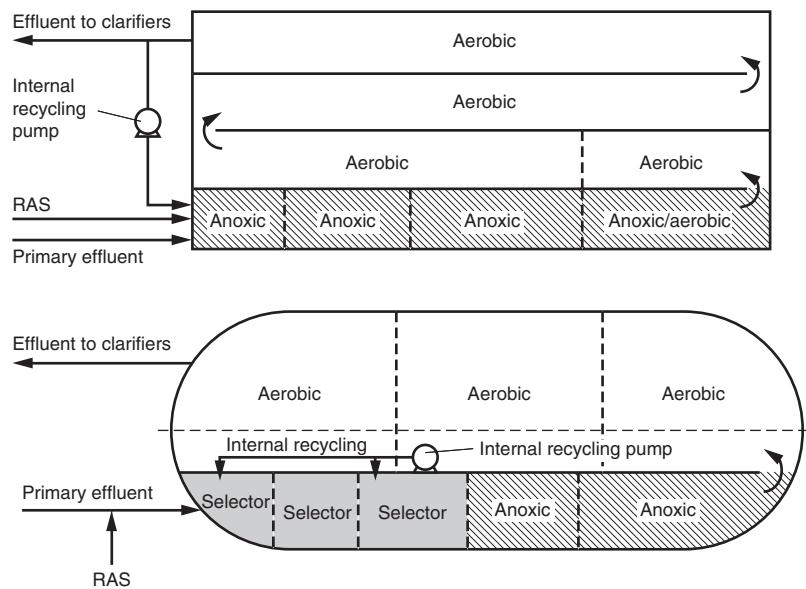
Characteristics of Filamentous Bacteria. In filamentous growth, bacteria form filaments of single-cell organisms that attach end-to-end, and the filaments normally protrude out of the sludge floc. This structure, in contrast to the preferred dense floc with good settling properties, has an increased surface area to mass ratio, which results in poor settling. Methods have been developed for the identification and classification of filamentous bacteria found commonly in activated sludge systems (Eikelboom, 2000). The classification system is based on morphology (size and shape of cells, length and shape of filaments), staining responses, and cell inclusions. Common filamentous organisms are summarized in Table 8-7, along with the operating conditions that favor their growth. Identifying the specific type of filamentous organism which may help identify an operating or design condition that encourages their growth (Jenkins et al., 2004). Examples of good and poor settling with floc containing filamentous bacteria are illustrated on Fig. 8-13.

Occurrence of Filamentous Sludge Bulking. Many types of filamentous bacteria exist, but the types that occur most frequently are related to wastewater characteristics, reactor design limitations, and operational issues. Individual items associated with each of these categories are identified in Table 8-8. Activated sludge reactor operating conditions (low DO, low F/M, and complete-mix operation) clearly have an effect on the development of filamentous populations. One of the kinetic features of filamentous organisms that relates to these conditions is that they are very competitive at low substrate concentrations whether it be organic substrates, DO, or nutrients. Thus, lightly loaded complete-mix activated sludge systems or low DO (<0.5 mg/L) operating conditions provide an environment more favorable to filamentous bacteria than to the desired floc-forming bacteria.

Occurrence of Viscous Sludge Bulking. Another type of bulking that can occur, known as *viscous bulking*, is caused by the presence of an excessive amount of extracellular biopolymer, which results in a sludge with a slimy, jellylike consistency (Wanner, 1994). As the biopolymers are hydrophilic, the activated sludge is highly water-retentive. The resultant sludge has a low density with low settling velocities and poor compaction. Viscous bulking is usually found with nutrient-limited systems or in a very high F/M loading condition with wastewater having a high amount of rbCOD.

Figure 8-17

Examples of full-scale wastewater treatment plant modifications to anoxic/aerobic treatment for nitrogen removal with staged anoxic selector zones.



8-5 ACTIVATED SLUDGE PROCESS DESIGN CONSIDERATIONS

Activated sludge design involves performing mass balances on key constituents and the application of fundamental kinetic relationships. These calculations can be done using desktop calculations, spreadsheets and/or computer simulation models. It is common today to use steady-state spreadsheet calculations to aid in the understanding of the process to be designed and to provide a basic starting point in the design to be used in computer simulation models for dynamic process analysis and design optimization. The purpose of this section is to provide a summary of steady-state design approaches and an introduction computer simulation modeling, including common model parameters and reactions and the universal matrix model format that is used to describe reaction components, stoichiometry, and kinetics.

Steady-State Design Approach

The fundamental principles of wastewater characterization, biological treatment and process analysis were presented in Chap. 7 and in Secs. 8-2 and 8-3. The activated sludge system SRT has been presented as a fundamental process parameter that can be used to determine the effluent substrate concentration, sludge wasting rate, and total oxygen demand rate. The principal relationships between SRT and the basic design and performance parameters presented in the previous sections are summarized in Table 8-10, along with equations to determine the aeration tank volume as a function of SRT and MLSS concentration. These equations are appropriate for determining process design and effluent concentrations for steady-state operating conditions at constant flow and influent wastewater constituent concentrations.

Impact of Diurnal Variations in Wastewater Characteristics. As reported in Chap. 3, wastewater flowrates and concentrations are not constant, but vary diurnally. When operating at a given SRT, the effluent substrate concentration will be higher

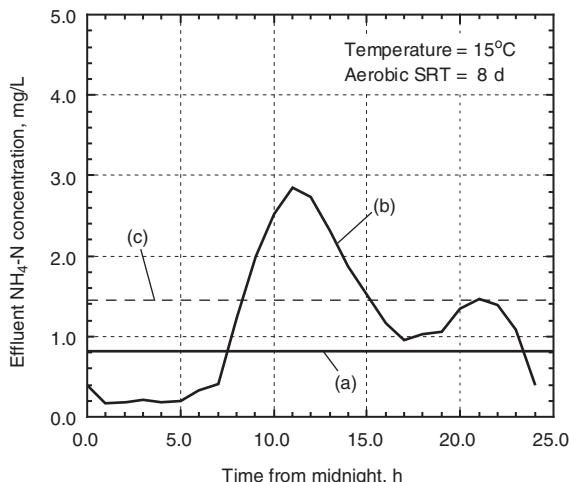
Table 8-10**Summary of equations used in the analysis of suspended growth processes**

Application	Equation	Eq. No.
Temperature	$k_T = k_{20}\theta^{(T-20)}$	1-44
Rate of sCOD Utilization	$r_{SU} = \frac{kXS}{K_S + S}$	7-12
	$\mu_{\max} = Yk$	7-16
Rate of $\text{NH}_4\text{-N}$ Oxidation	$r_{\text{NH}_4} = \left(\frac{\mu_{\max, \text{AOB}}}{Y_{\text{AOB}}}\right) \left(\frac{S_{\text{NH}_4}}{S_{\text{NH}_4} + K_{\text{NH}_4}}\right) \left(\frac{S_o}{S_o + K_{o, \text{AOB}}}\right) X_{\text{AOB}}$	7-101
Rate of $\text{NO}_3\text{-N}$ Utilization	$r_{\text{NO}_3} = \left(\frac{1 - 1.42Y_H}{2.86}\right) \left[\frac{\mu_{H, \max} S_s}{Y_H(K_s + S_s)}\right] \left(\frac{S_{\text{NO}_2}}{K_{\text{NO}_2} + S_{\text{NO}_2}}\right) \left(\frac{K'_o}{K'_o + S_o}\right) (\eta) X$	7-133
Specific Growth Rate and SRT	$\mu_{\text{AOB}} = \mu_{\max, \text{AOB}} \left(\frac{S_{\text{NH}_4}}{S_{\text{NH}_4} + K_{\text{NH}_4}}\right) \left(\frac{S_o}{S_o + K_{o, \text{AOB}}}\right) - b_{\text{AOB}}$	7-94
	$\text{SRT} = \frac{1}{\mu_{\text{AOB}}}$	7-98
	$\text{SF} = \text{SRT}_{\text{des}}/\text{SRT}_{\min}$	7-73
Biomass Production, Heterotrophs (VSS)	$P_{X, \text{bio}} = \frac{QY_H(S_o - S)}{1 + b_H(\text{SRT})} + \frac{(f_d)(b_H)QY_H(S_o - S)\text{SRT}}{1 + b_H(\text{SRT})}$	8-20 (A+B)
Sludge Production ($P_{x, \text{VSS}}$)	$P_{x, \text{VSS}} = P_{X, \text{bio}} + \frac{QY_n(\text{NO}_X)}{1 + b_n(\text{SRT})} + Q(\text{nbVSS})$	8-20
Sludge Production ($P_{x, \text{TSS}}$)	$P_{x, \text{TSS}} = \frac{P_{X, \text{bio}}}{0.85} + \frac{QY_n(\text{NO}_X)}{0.85[1 + b_n(\text{SRT})]} + Q(\text{nbVSS}) + Q(\text{TSS}_o - \text{VSS}_o)$	8-21
Reactor Mass and Volume	$\text{Mass} = X_{\text{VSS}}(V) = (P_{X, \text{VSS}})\text{SRT}$	7-56
	$\text{Mass} = X_{\text{TSS}}(V) = (P_{X, \text{TSS}})\text{SRT}$	7-57
SRT	$\text{SRT} = \frac{VX}{(Q - Q_w)X_e + Q_w X_R}$	8-27
	$\text{SRT} = \frac{V}{Q_w}$	8-31
	$\text{SRT} = \frac{\left(\frac{R}{1+R}\right)V_A + V_M}{Q_w}$	8-36
CMAS Effluent bsCOD	$S = \frac{K_s[1 + b_H(\text{SRT})]}{\text{SRT}(Y_Hk - b_H) - 1}$	7-46
CMAS Biomass	$X = \left(\frac{\text{SRT}}{\tau}\right) \left[\frac{Y_H(S_o - S)}{1 + b_H(\text{SRT})}\right]$	7-42
CMAS Oxygen Req'd	$R_o = Q(S_o - S) - 1.42P_{X, \text{bio}} + 4.57Q(\text{NO}_X)$	8-23
Ammonia oxidized	$\text{NO}_X = \text{TKN} - N_e - 0.12P_{X, \text{bio}}/Q$	8-24
Stage Reactor Oxygen Req'd	$R_{o,2} = (Q_1)(1 - Y_H)[(S_{s,1} - S_{s,2}) + (X_{s,1} - X_{s,2})] + (Q_1)4.57(\text{NO}_2 - \text{NO}_1) + 1.42b_H(X_{b,2})V_2$	8-25
Food to Mass Ratio	$F/M = \frac{QS_o}{VX}$	7-62
Organic Loading	$L_{\text{org}} = \frac{(Q)(S_o)}{(V)}$	7-69

All terms as defined previously.

Figure 8-18

Comparison of effluent $\text{NH}_4\text{-N}$ concentrations from nitrification activated sludge systems operated at the same temperature and SRT with the same average influent flowrate and TKN loading: (a) effluent $\text{NH}_4\text{-N}$ concentration with constant influent conditions, (b) effluent $\text{NH}_4\text{-N}$ concentration versus time with diurnal variations in influent flowrate and concentration, and (c) value of the 24-h composite $\text{NH}_4\text{-N}$ concentration with diurnal variations in influent flowrate and concentration.



at the higher loading periods and lower at the lower loading periods. An example of the effect of diurnal load variations on effluent $\text{NH}_4\text{-N}$ concentrations for a nitrification activated sludge process operated with an 8-d aerobic SRT and 15°C temperature is illustrated on Fig. 8-18. The average influent BOD and TKN concentrations used for calculations in this example are 220 mg/L and 35 mg/L, respectively. Assuming constant flow and loadings, the steady state effluent $\text{NH}_4\text{-N}$ concentration, calculated using Eqs. (7-94) and (7-98), is 0.82 mg/L. The diurnal load was based on the diurnal flowrate and BOD variations for domestic wastewater shown on Fig. 3-11. Using the same kinetic parameters and coefficient values in a dynamic simulation model, the effluent $\text{NH}_4\text{-N}$ concentration will vary from 0.2 to 2.8 mg/L over the 24 h period, and the flow-weighted composite effluent $\text{NH}_4\text{-N}$ concentration is 1.45 mg/L. To assure that the daily composite effluent sample substrate concentration is at or below the desired design value, the design SRT that yields the target steady state effluent substrate concentration is multiplied by a design safety factor (typically 1.3 to 1.5 for domestic wastewater treatment) so that a sufficient biomass is available to handle the higher loadings.

Impact of Using Staged Reactors. Use of steady-state equations is satisfactory for the design of completely mixed activated sludge (CMAS) processes, but is not used as easily to determine substrate concentrations and oxygen demand rates in each stage of activated sludge process designs with multiple reactors in series. Staged reactors are common for biological nutrient removal and nitrification process designs. As indicated on Fig. 8-8(b), the bsCOD and pbCOD concentrations vary from stage to stage and accordingly their degradation rates vary. The necessary calculations of process behavior under varying load conditions for single-stage CMAS systems or CSTRs in series, including aerobic, anoxic, and anaerobic conditions, can be accomplished more readily with computer simulation models.

Use of Simulation Models

Simulation models can be used to account for the kinetics and changes in constituent concentrations in each mixed tank of an activated sludge process. Computer modeling provides the tool to incorporate the large number of components and reactions to evaluate activated sludge performance under both dynamic and steady-state conditions, and to easily design multiple-staged processes as well as single-stage complete-mix processes.

Historical Development of Activated Sludge Model. The development of the first comprehensive activated sludge model (ASM1) by the International Water Association (IWA) task group (Henze et al., 1987) was a major step forward in the use of computer simulation models for activated sludge process analysis and design. Limited to carbon oxidation, nitrification and denitrification, the ASM1 model was followed by the ASM2 and ASM2d models, which included fermentation, enhanced biological phosphorus removal, and chemical phosphorus removal (Henze et al., 1995, Barker and Dold, 1997). Further model structure modifications have been made in ASM3 (Gujer et al., 1999, and Henze et al., 2000). Commercial software packages that include these models are used commonly by wastewater process engineers for the process design of different activated sludge system configurations.

General Model Characteristics. The components included in activated sludge simulation models are termed *state variables*. The principal components are summarized in Table 8-11 along with brief descriptions of the types of reactions causing their production or depletion. Activated sludge process models used today are much more complex and now include BOD removal, nitrification, denitrification, and phosphorus removal. The reactions involved in these processes are carried out by different types of bacteria that include a mixture of heterotrophic bacteria that are phosphorus-storing and non-phosphorus-storing, and among these, bacteria that can and cannot use nitrate as an electron acceptor, as well as autotrophic nitrifying bacteria. The impact of wastewater components such as fermentable soluble COD, acetate, biodegradable particulate COD, inert particulate and soluble COD, inorganic nitrogen, and soluble phosphorus on oxygen consumption and sludge production rates are now included.

Important Model Features. The models are based on growth as opposed to substrate utilization. Monod specific growth rate kinetics are used to model the growth of autotrophic or heterotrophic bacteria. Substrate, oxygen, and nutrient utilization rates are related to the growth rates by stoichiometric factors. Another important feature of the models is that COD is used as the common measure of organic substrate and biomass so that a COD balance exists for substrate utilization, biomass growth, and oxygen consumption. The models also use the lysis-regrowth model for endogenous respiration instead of a net endogenous decay coefficient as presented in this chapter for activated sludge design. In the lysis-regrowth model, endogenous decay results in the release of biomass particulate material, some of which is biodegradable and is hydrolyzed to provide a source of rbCOD. Another portion remains as cell debris, similar to what has been incorporated for endogenous respiration in the activated sludge design model presented in this chapter. To compute the same amount of sludge production as the endogenous respiration model, the decay coefficient value in the lysis-regrowth model is higher.

Inclusion of Nitrite. The ASM1, ASM2, ASM2d, and ASM3 models do not include nitrite as a state variable, because most of the ammonia oxidized in many activated sludge processes is complete to nitrate with very little nitrite present. However, nitrite is included in many simulation models today because it may represent an important fraction of ammonia oxidation under certain process conditions, including higher temperatures ($>25^{\circ}\text{C}$), the initial stages of a multi-staged nitrification system, and low dissolved oxygen concentration.

Matrix Model Format. A long list of complex equations would be needed to describe the various reactions in an activated sludge process involving numerous components such as organic substrates (soluble and particulate), inorganic substrates (ammonia, nitrate, and

Table 8-11**Key process components in ASM2d and types of reactions affecting their reactor concentration**

Model component	Symbol	Production or input	Reactions or input
			Depletion
Dissolved O ₂	S_{O_2}	<ul style="list-style-type: none"> Influent wastewater Aeration 	<ul style="list-style-type: none"> Consumption by X_H, X_{AUT}, X_{PAO}
rbCOD	S_F	<ul style="list-style-type: none"> Influent wastewater Hydrolysis of X_S 	<ul style="list-style-type: none"> Biodegradation by X_H Fermentation by X_H
Acetate	S_A	<ul style="list-style-type: none"> Influent WWT Fermentation of S_F 	<ul style="list-style-type: none"> Uptake by X_{PAO} Biodegradation by X_H
Ammonia	S_{NH_4}	<ul style="list-style-type: none"> Influent wastewater Hydrolysis of organic N Hydrolysis of cell decay products 	<ul style="list-style-type: none"> Oxidation by autotrophic bacteria (X_{AUT}) Synthesis uptake by X_H, X_{AUT}, X_{PAO}
Nitrate	S_{NO_3}	<ul style="list-style-type: none"> Oxidation of S_{NH_4} by X_{AUT} 	<ul style="list-style-type: none"> Synthesis
Phosphorus	S_{PO_4}	<ul style="list-style-type: none"> Influent wastewater Hydrolysis of organics 	<ul style="list-style-type: none"> Synthesis uptake by X_H, X_{AUT}, X_{PAO} Anoxic and aerobic uptake by X_{PAO}
Alkalinity	S_{ALK}	<ul style="list-style-type: none"> Influent wastewater During biological reduction of S_{NO_3} 	<ul style="list-style-type: none"> During S_{NH_4} oxidation by X_{AUT}
Biodegradable particulate COD	X_I	<ul style="list-style-type: none"> Influent wastewater Cell decay 	
Slowly biodegradable COD	X_S	<ul style="list-style-type: none"> Influent wastewater Cell decay 	<ul style="list-style-type: none"> Hydrolysis by X_H
Ordinary heterotrophs	X_H	<ul style="list-style-type: none"> Growth from S_F, S_A 	<ul style="list-style-type: none"> Cell decay
Phosphorus accumulating heterotrophs	X_{PAO}	<ul style="list-style-type: none"> Growth from using X_{PHA} 	<ul style="list-style-type: none"> Cell decay
Stored PHA	X_{PHA}	<ul style="list-style-type: none"> Production in anaerobic zone by X_{PAO} from S_A 	<ul style="list-style-type: none"> Biodegradation by X_{PAO} in aerobic and anoxic zones
Stored polyphosphate	X_{PP}	<ul style="list-style-type: none"> During oxidation of X_{PHA} 	<ul style="list-style-type: none"> Release in anaerobic conditions by X_{PAO}
Ammonia oxidizing bacteria	X_{AUT}	<ul style="list-style-type: none"> Growth during S_{NH_4} oxidation 	<ul style="list-style-type: none"> Cell decay

phosphorus), dissolved oxygen, and various heterotrophic and autotrophic bacteria. Instead of presenting the model in terms of numerous equations, a more general matrix model approach has been adopted. Process reactions and the stoichiometric factors that link the components to the various reactions are presented in a matrix model format. The advantage of the matrix format is that a relatively simple, concise format can be used to describe the process. The purpose of this section is to provide a basic introduction to the matrix model approach showing the components, reactions, and stoichiometric coefficients for the activated sludge process model and how the matrix format can be used to describe the process. In addition, the approach can also be used to illustrate how the matrix

model can be interpreted to describe a complete set of equations for a given process component. For example, the Activated Sludge Model No. 2 (ASM2d) (Henze et al., 1995) is used to describe basic features of a comprehensive activated sludge model.

Model Matrix Format, Components, and Reactions

A convenient matrix format is used to describe the model without having to present the large number of equations involved. The ASM2d model includes 19 components and accounts for 21 process reactions. Some of the component and reaction terms are described here to illustrate the basic model format.

Process Reactions and Stoichiometric Coefficients. Fourteen key components related to the biological processes are described in Table 8–11. An example of some of the process reactions and the corresponding stoichiometric rate coefficients are given in Tables 8–12 and 8–13, respectively. The stoichiometric coefficients are used to relate

Table 8-12

Example of process rate equations selected from ASM2d

<i>i</i> ^a	<i>i</i> Process	Process rate equations, <i>r_i</i>
Hydrolysis Processes		
1	Aerobic Hydrolysis	$K_h \left(\frac{S_{O_2}}{K_{O_2} + S_{O_2}} \right) \left(\frac{X_s/X_H}{K_X + X_s/X_H} \right) X_H$
2	Anoxic Hydrolysis	$K_h \left(\frac{K'_{O_2}}{K_{O_2} + S_{O_2}} \right) \left(\frac{S_{NO_3}}{K_{NO_3} + S_{NO_3}} \right) \left(\frac{X_s/X_H}{K_X + X_s/X_H} \right) (\eta_{NO_3}) X_H$
Heterotrophic Organisms, <i>X_H</i>		
4	Aerobic Growth on <i>S_F</i>	$\mu_H \left(\frac{S_{O_2}}{K_{O_2} + S_{O_2}} \right) \left(\frac{S_F}{K_F + S_F} \right) \left(\frac{S_F}{S_A + S_F} \right) (Growth_{lim})(X_H)$
5	Aerobic Growth on <i>S_A</i>	$\mu_H \left(\frac{S_{O_2}}{K_{O_2} + S_{O_2}} \right) \left(\frac{S_A}{K_A + S_A} \right) \left(\frac{S_A}{S_A + S_F} \right) (Growth_{lim})(X_H)$
6	Anoxic Growth on <i>S_F</i>	$\mu_H (\eta_{NO_3}) \left(\frac{K'_{O_2}}{K'_{O_2} + S_{O_2}} \right) \left(\frac{S_{NO_3}}{K_{NO_3} + S_{NO_3}} \right) \left(\frac{S_F}{K_F + S_F} \right) \left(\frac{S_F}{S_A + S_F} \right) (Growth_{lim})(X_H)$
7	Anoxic Growth on <i>S_A</i>	$\mu_H (\eta_{NO_3}) \left(\frac{K'_{O_2}}{K'_{O_2} + S_{O_2}} \right) \left(\frac{S_{NO_3}}{K_{NO_3} + S_{NO_3}} \right) \left(\frac{S_A}{K_A + S_A} \right) \left(\frac{S_A}{S_A + S_F} \right) (Growth_{lim})(X_H)$
8	Growth _{lim} Equation	$\left(\frac{S_{NH_4}}{K_{NH_4} + S_{NH_4}} \right) \left(\frac{S_{PO_4}}{K_{PO_4} + S_{PO_4}} \right) \left(\frac{S_{ALK}}{K_{ALK} + S_{ALK}} \right)$
9	Cell Lysis	$b_H(X_H)$
Ammonia Oxidizing Bacteria, <i>X_{AUT}</i>		
18	Aerobic Growth on <i>S_{NH₄}</i>	$\mu_{AUT} \left(\frac{S_{O_2}}{K_{O_2} + S_{O_2}} \right) \left(\frac{S_{NH_4}}{K_{NH_4} + S_{NH_4}} \right) \left(\frac{S_{PO_4}}{K_{PO_4} + S_{PO_4}} \right) \left(\frac{S_{ALK}}{K_{ALK} + S_{ALK}} \right) (X_{AUT})$
19	Cell Lysis	$b_{AUT}(X_{AUT})$

^a *i* = ASM2d equation number.

Table 8-13**Stoichiometric matrix for selected components in ASM2d**

<i>i</i>	<i>iProcess</i>	Component								
		<i>S_F</i>	<i>S_A</i>	<i>S_I</i>	<i>S_{NH₄}</i>	<i>S_{O₂}</i>	<i>S_{NO₃}</i>	<i>X_S</i>	<i>X_I</i>	<i>X_H</i>
Hydrolysis processes										
1	Aerobic hydrolysis	1							-1	
2	Anoxic hydrolysis	1							-1	
Heterotrophic organisms, <i>X_H</i>										
4	Aerobic growth on <i>S_F</i>	$-\frac{1}{Y_H}$			$\frac{-i_N}{X_H}$	$1 - \frac{1}{Y_H}$				1
5	Aerobic growth on <i>S_A</i>		$-\frac{1}{Y_H}$		$\frac{-i_N}{X_H}$	$1 - \frac{1}{Y_H}$				1
6	Anoxic growth on <i>S_F</i>	$-\frac{1}{Y_H}$			$\frac{-i_N}{X_H}$		$-\frac{(1 - Y_H)}{2.86 Y_H}$			1
7	Anoxic growth on <i>S_A</i>		$-\frac{1}{Y_H}$		$\frac{-i_N}{X_H}$		$-\frac{(1 - Y_H)}{2.86 Y_H}$			1
9	Cell lysis							$1 - f_{X_I}$	f_{X_I}	-1
Phosphorus accumulating organisms										
Ammonia oxidizing bacteria, <i>X_{AUT}</i>										
18	Aerobic growth on <i>S_{NH₄}</i>			$-\frac{i_N}{X_{AUT}}$	$-\frac{1}{Y_{AUT}}$	$\frac{4.57 - Y_{AUT}}{Y_{AUT}}$				1
19	Cell lysis							$1 - f_{X_I}$	f_{X_I}	-1

Simultaneous precipitation of phosphorus with ferric hydroxide.

changes in model components to the growth rate. The process reactions in ASM2d are divided into the following five groups dealing with:

- Hydrolysis processes
- Heterotrophic organisms (including aerobic oxidation, denitrification, fermentation, and cell lysis)
- Phosphorus accumulation organisms
- Ammonia oxidizing organisms
- Simultaneous precipitation of phosphorus with ferric hydroxide

For simplicity, the process equations related to enhanced biological phosphorus removal including a description of PHA and phosphorus storage under anaerobic, anoxic, and aerobic conditions; phosphorus accumulating organism (*X_{PAO}*) growth under anoxic and aerobic conditions; and *X_{PAO}* cell lysis and release of phosphorus and PHA are not shown in Table 8-12.

For example, as given in Table 8-12, the change in the heterotrophic bacteria (*X_H*) concentration is related to process reactions 4, 5, 6, 7, and 9. The term η represents the fraction of heterotrophic bacteria that can use nitrate in place of dissolved oxygen.

The corresponding stoichiometric rate coefficients for these biomass growth reactions are 1, 1, 1, 1 as given in Table 8–13. For cell decay, the death-lysis model is used, which is given by process reactions 9 and 19 for heterotrophic and autotrophic bacteria, respectively (see Table 8–12). The cell debris material produced during biomass decay is indicated by X_l and the production of degradable particulate substrate (X_s) is indicated by a stoichiometric coefficient $(1 - f_{Xl})$. The hydrolysis of the particulate organic substrates under either aerobic or anoxic conditions is described by processes 1 and 2, respectively.

Rate Expressions. The rate expression for each component for a specific reaction is the product of the stoichiometric coefficient given in Table 8–13 and the corresponding process reaction given in Table 8–12. An equation for each component (e.g., S_F , S_A , etc.) can be written by summing the product of the stoichiometric coefficients given in Table 8–13 for that term times the corresponding rate expressions given in Table 8–12. The summation rate equation is:,

$$R_C = \sum_{i=1}^n C_i(j_i) \quad (8-44)$$

where R_C = rate of change for the process component C (e.g., S_F , S_A , X_l , X_s , etc.)

C_i = stoichiometric coefficient for component C , reaction i

j_i = rate reaction i

For example, for component S_A under aerobic growth (reaction 5, Table 8–12), the stoichiometric coefficient from Table 8–13 is $-(1/Y_H)$. Thus, under aerobic conditions, the rate of change of S_A is given by the following expression:

$$R_{S_A} = -\frac{1}{Y_H} \mu_H \left(\frac{S_{O_2}}{K_{O_2} + S_{O_2}} \right) \left(\frac{S_A}{K_A + S_A} \right) \left(\frac{S_A}{S_A + S_F} \right) (\text{Growth}_{\text{Lim}})(X_H) \quad (8-45)$$

$$\text{where } \text{Growth}_{\text{Lim}} = \left(\frac{S_{NH_4}}{K_{NH_4} + S_{NH_4}} \right) \left(\frac{S_{PO_4}}{K_{PO_4} + S_{PO_4}} \right) \left(\frac{S_{ALK}}{K_{ALK} + S_{ALK}} \right)$$

The stoichiometric coefficients for oxygen consumption are described as follows: for heterotrophic growth, the term $(1 - Y_H)$ is the g O₂ used/g COD removed. The term $(1 - Y_H)$ is divided by Y_H (g cell COD/g COD used) to obtain the stoichiometric coefficient as g O₂/g cell COD produced to fit to the matrix format. The stoichiometric term for autotrophic growth contains the factor 4.57. The term is required because ammonia, the substrate for the nitrifying bacteria, is expressed as nitrogen in the matrix S_{NH_4} , and oxygen is expressed as COD. The oxygen equivalent for ammonia is 4.57 g O₂/g NH₄-N. The amount in the numerator is lower by Y_A , which accounts for the ammonia used in cell synthesis. Application of Eqs. (8–44) and (8–45) is demonstrated in Example 8–2. It is intended to illustrate how the model matrix is interpreted to describe the rate of concentration change of a model component.

Example 8–2 Apply ASM2d Model Matrix Use the ASM2d model matrix to describe the concentration of the readily biodegradable COD component, S_F , due to nonphosphorus accumulating heterotrophic bacteria in reactor number 2 of the staged reactor shown on Fig 8–8(b). Substitute $S_F + S_A$ for S_s in figure for the rbCOD components. For simplicity in demonstrating the use of the matrix information the PAO reactions are not included here.

Solution

1. Develop mass balance equation for S_F and S_A for reactor 2.
- a. The general word statement is

Rate of change = rate in – rate out + rate of production + rate of depletion

- b. Write the symbolic representation.

- i. The incoming and outgoing mass of S_F with the flow is $(Q + Q_R) S_{F,1}$, and $(Q + Q_R) S_{F,2}$ expressed in g/d, respectively.
- ii. The rate of production of S_F due to aerobic and anoxic hydrolysis, given by reactions 1 and 2 in Table 8–12, can be represented as R_1 and R_2 , respectively.
- iii. The rate of depletion of S_F due to aerobic and anoxic growth, given by reactions 4 and 6 in Table 8–12, can be represented as R_4 and R_6 , respectively.
- iv. Substituting the terms given above in the mass balance equation yields the following expression

$$V_2 \frac{dS_{F,2}}{dt} = (Q + Q_R) S_{F,1} - (Q + Q_R) S_{F,2} + R_1 V_2 + R_2 V_2 + R_4 V_2 + R_6 V_2$$

2. Write the mass balance equation with the appropriate rate expressions including the process reactions from Table 8–12 and the corresponding stoichiometric coefficients from Table 8–13. For example, the rate expression for aerobic hydrolysis, R_1 , for component S_F (rbCOD) is:

$$R_1 = 1 \times K_h \left(\frac{S_{O_2}}{K_{O_2} + S_{O_2}} \right) \left[\frac{X_{S,2}/X_{H,2}}{K_X + (X_{S,2}/X_{H,2})} \right] X_{H,2}$$

Thus, the mass balance S_A for the reactor 2 can be expressed as:

$$\begin{aligned} V_2 \frac{dS_{F,2}}{dt} &= (Q + Q_R) S_{F,1} - (Q + Q_R) S_{F,2} \\ &\quad + K_h \left(\frac{S_{O_2,2}}{K_{O_2} + S_{O_2,2}} \right) \left[\frac{X_{S,2}/X_{H,2}}{K_X + (X_{S,2}/X_{H,2})} \right] X_{H,2}(V_2) \\ &\quad + K_h \left(\frac{K'_{O_2}}{K'_{O_2} + S_{O_2}} \right) \left(\frac{S_{NO_3}}{K_{NO_3} + S_{NO_3}} \right) \left[\frac{X_{S,2}/X_{H,2}}{K_X + (X_{S,2}/X_{H,2})} \right] (\eta_{NO_3}) X_{H,2}(V_2) \\ &\quad + \left(\frac{-1}{Y_H} \right) \mu_H \left(\frac{S_{O_2,2}}{K_{O_2} + S_{O_2,2}} \right) \left(\frac{S_{F,2}}{K_F + S_{F,2}} \right) \left(\frac{S_{F,2}}{S_{A,2} + S_{F,2}} \right) (\text{Growth}_{\text{Lim},2})(X_{H,2})(V_2) \\ &\quad + \left(\frac{-1}{Y_H} \right) \mu_H (\eta_{NO_3}) \left(\frac{K'_{O_2}}{K'_{O_2} + S_{O_2,2}} \right) \left(\frac{S_{NO_3,2}}{K_{NO_3} + S_{NO_3,2}} \right) \left(\frac{S_{F,2}}{K_F + S_{F,2}} \right) \left(\frac{S_{F,2}}{S_{A,2} + S_{F,2}} \right) \\ &\quad (\text{Growth}_{\text{Lim},2})(X_{H,2})(V_2) \end{aligned}$$

Comment

The solution for S_F depends on the concentrations of (1) non-PAO heterotrophic bacteria, (2) slowly biodegradable COD, X_s , (3) dissolved oxygen, (4) nitrate-nitrogen, (5) acetate COD, S_A , (6) ammonia-nitrogen, (7) phosphate, and (8) alkalinity. Similar sets of equations can be provided for these components using the matrix information shown in Table 8–13.

Other Simulation Model Applications

In addition to process design and analysis, models may be used as (1) a research tool to evaluate biological processes and to better understand important parameters that affect a certain type of performance, and (2) as a means to evaluate the treatment capacity of a given facility. For item 2, the availability of accurate and representative wastewater characterization data is critical where dynamic simulations are used to assess the effect of variable flow and concentrations with time.

Evaluation of Plant Capacity. To evaluate the capacity of an existing plant, the model is calibrated using wastewater characterization and plant performance data. Calibrations based only on comparing the model predictions to the plant effluent concentration are not necessarily valid. Low effluent concentrations normally exist for all soluble degradable components, and thus the ability of the model to predict the plant performance is clouded by analytical accuracy and practical considerations. Intermediate soluble substrate concentrations from aerobic stages and/or anoxic and anaerobic stages provide a more reliable indication of the ability to describe the kinetics for the site. Oxygen uptake rate data are more meaningful for model calibration, as the data reflect different factors in the model including kinetic rates for different reactions and stoichiometric ratios for cell yield and decay. Oxygen uptake rate data for staged systems are valuable for model calibration. Nitrification rates and sludge production rates are also important parameters for model calibration.

Use of Model Default Values. Default values for 45 kinetic parameters in ASM2d have been selected and are summarized in the model report (Henze et al., 1995). Using these values does not ensure that the model can be used to predict the performance of an activated sludge process accurately, as some of the coefficient values can be different at different sites. One of the parameters found to vary the most at different sites and often adjusted during model calibration is the maximum specific growth rate of nitrifying bacteria μ_{AUT} . Variations in nitrification kinetics may be due to differences in wastewater characteristics, population selection, and inhibitors on nitrification or may also reflect other kinetic changes for which μ_{AUT} adjustments improve the overall fit.

Evaluation of Different Process Configuration. Process design engineers can take advantage of the powerful tool provided by simulation models to study various design configurations and possible operating conditions on expected system performance. Because the selection of process reaction equations and coefficient values used in different commercial software packages varies, the design engineer must be familiar with all model assumptions and model structure. Simple desktop designs, as will be presented in the next three sections, can be used to obtain reactor sizing and configuration for use in the simulation models and to also gain a sense for the expected results as a function of SRT, number of stages, DO conditions, and recycle streams. At a minimum the designer should be comfortable with the oxygen demand rates and sludge production rates predicted by the simulation models relative to alternate calculation methods.

Recent Simulation Models. The model presented in this section is consistent with ASM2d, but the latest commercial computer simulation models apply additional process models that are not included in ASM2d or ASM3. For example, ammonium oxidation to nitrite and nitrite oxidation to nitrate is not separated in the activated sludge models discussed above. In most commercial computer simulation programs, these two reactions are treated separately to describe more accurately the nitrogen removal processes, and also allow a modeling of nitritation/denitritation. Models for deammonification have also been developed and used by computer modeling programs.

8–6 PROCESSES FOR BOD REMOVAL AND NITRIFICATION

Important considerations for the design of activated sludge processes were presented in Secs. 8–3, 8–4, and 8–5. The purpose of this section is to illustrate in detail the design procedure for three common but different activated sludge processes for BOD removal and nitrification. The objective of the three different activated sludge process design examples presented in this section is to demonstrate the application of the fundamental principles discussed previously for BOD removal and nitrification processes and to provide insight into their behavior along with key design features. The material in this section is organized into the following topics: (1) overview of BOD removal and nitrification processes, (2) general process design considerations, (3), (4), (5) process designs for three different activated sludge processes, and (6) a summary of alternative processes used for BOD removal and nitrification, typical process design parameters, and process selection considerations. Biological nitrogen and phosphorus removal can be incorporated into most of the processes used for BOD removal and nitrification, but because additional design factors must be considered, nitrogen and phosphorus removal processes are covered separately in Secs. 8–7 and 8–8, respectively. Details for the selection and design of physical facilities may be found in Secs. 8–9, 8–11, and 8–12.

Overview of BOD Removal and Nitrification Processes

All of the considerations involved in BOD removal and nitrification process design can be applied to a BOD removal-only design by modifying the SRT and removing items that deal with nitrification. The design methodology presented is based fundamentally on using appropriate SRT values, and thus can be applied to the broad range of processes described at the end of this section. As an introduction to BOD removal and nitrification, descriptions of the following three process configurations that are used most commonly are presented below.

1. A complete-mix activated sludge process without and with nitrification
2. A sequencing batch reactor (SBR) with nitrification
3. A multi-staged nitrification process.

Each of these process configurations is considered in the following discussion.

Complete-Mix Activated Sludge Process. A typical complete-mix activated sludge (CMAS) process is shown on Fig. 8–19. Effluent from the primary sedimentation tank and recycled return activated sludge are introduced typically at several points in the reactor. Because the tank contents are mixed thoroughly, the organic load, oxygen demand, and substrate concentration are uniform throughout the entire aeration tank and the F/M ratio is low. Care should be taken to assure that the contents of CMAS reactor are well mixed and that influent feed and effluent withdrawal points are selected to prevent short-circuiting of untreated or partially treated wastewater. The complete-mix reactor is usually configured in square, rectangular, or round shapes. Tank dimensions depend mainly on the size, type, and mixing pattern of the aeration equipment.

Sequencing Batch Reactor Process. The sequencing batch reactor (SBR) process utilizes a fill-and-draw reactor with complete mixing during the batch reaction step (after filling) and where the subsequent steps of aeration and clarification occur in the same tank. All SBR systems have five steps in common, carried out in following sequence: (1) fill, (2) react (aeration), (3) settle (sedimentation/clarification), (4) draw (decant), and (5) idle. Each of these steps is illustrated on Fig. 8–20 and described later in Table 8–16 on page 771. For continuous-flow applications, at least two SBR tanks must be provided so that one tank receives flow

9

Attached Growth and Combined Biological Treatment Processes

9-1	INTRODUCTION TO ATTACHED GROWTH PROCESSES 943 <i>Types of Attached Growth Processes</i> 943 <i>Mass Transfer Limitations in Attached Growth Processes</i> 947
9-2	NONSUBMERGED ATTACHED GROWTH PROCESSES 947 <i>General Process Description</i> 947 <i>Trickling Filter Classification and Applications</i> 950 <i>Advantages and Disadvantages of Trickling Filters</i> 953 <i>Physical Facilities for Trickling Filters</i> 954 <i>Design Considerations for Physical Facilities</i> 957 <i>Process Design Considerations for BOD Removal</i> 968 <i>Process Analysis for BOD Removal</i> 972 <i>Process Analysis for Nitrification</i> 978
9-3	SEQUENTIAL COMBINED TRICKLING FILTER AND SUSPENDED SOLIDS PROCESSES 987 <i>Process Development</i> 987 <i>Process Applications</i> 987 <i>Trickling Filter/Solids Contact Process</i> 988 <i>Trickling Filter/Activated Sludge Process</i> 990 <i>Series Trickling Filter and Activated Sludge Process</i> 997
9-4	INTEGRATED FIXED FILM ACTIVATED SLUDGE PROCESS 997 <i>Process Development</i> 998 <i>Process Applications</i> 1000 <i>IFAS Process Advantages and Disadvantages</i> 1002 <i>Design of Physical Facilities</i> 1003 <i>IFAS Process Design Analysis</i> 1005 <i>BOD and Nitrification Design Approach</i> 1008
9-5	MOVING BED BIOFILM REACTOR (MBBR) 1015 <i>Background</i> 1015 <i>MBBR Process Applications</i> 1016 <i>MBBR Process Advantages and Disadvantages</i> 1016 <i>Design of Physical Facilities</i> 1019 <i>MBBR Process Design Analysis</i> 1020 <i>BOD Removal and Nitrification Design</i> 1021
9-6	SUBMERGED AEROBIC ATTACHED GROWTH PROCESSES 1026 <i>Process Development</i> 1026 <i>Process Applications</i> 1027 <i>Process Advantages and Disadvantages</i> 1027 <i>Design of Physical Facilities</i> 1029 <i>BAF Process Design Analysis</i> 1031 <i>FBBR Process Design Analysis</i> 1034

9-7	ATTACHED GROWTH DENITRIFICATION PROCESSES	1034
	<i>Process Development</i>	1034
	<i>Description and Application of Attached Growth Denitrification Processes</i>	1035
	<i>Process Design Analysis of Postanoxic Attached Growth Denitrification</i>	1037
	<i>Operational Considerations for Postanoxic Attached Growth Denitrification</i>	1041
9-8	EMERGING BIOFILM PROCESSES	1045
	<i>Membrane Biofilm Reactors</i>	1045
	<i>Biofilm Airlift Reactors</i>	1046
	<i>Aerobic Granules Reactor</i>	1046
	PROBLEMS AND DISCUSSION TOPICS	1046
	REFERENCES	1052

WORKING TERMINOLOGY

Term	Definition
Attached growth aerobic process	Aerobic treatment processes in which the biomass responsible for treatment is attached to some type of medium (packing material).
Biofilm	A layer of biomass that develops on the surface of media used in attached growth biological processes.
Biological aerated filter (BAF)	An aerobic downflow or upflow submerged growth process that uses an inert media to support the growth of a biofilm for the removal of dissolved organic matter. Filtration also occurs for suspended solids removal.
Denitrification filter	An anoxic packed bed reactor that contains granular or synthetic media to support a biofilm for nitrate and nitrite reduction and to filter effluent suspended solids.
Diffusion limited substrate removal	The substrate removal rate in a biofilm is limited by the rate of diffusion across a stagnant layer between the biofilm and mixed bulk liquid and diffusion into the unmixed biofilm layer.
Fluidized-bed bioreactor (FBBR)	A submerged attached growth process that can be used for either aerobic or anaerobic applications where wastewater is fed upward to a bed of sand or activated carbon; the fluid velocity causes the bed to expand and form a biofilm that removes soluble organic matter.
Hybrid processes	Those processes that use a combination of attached-growth and suspended-growth processes.
Integrated fixed film activated sludge (IFAS)	An activated sludge process in which a portion of the biomass is maintained on fixed or suspended media that is separated from the reactor effluent by screening. Return activated sludge is required.
Moving-bed bioreactor (MBBR)	A submerged attached growth biological process in which most of the biomass is maintained on suspended media, which is separated from the reactor effluent by screening. No return activated sludge is used.
Packed bed filter	A process that makes use of biofilm microbial communities attached to fixed packing materials.
Oxygen limited substrate removal	Rate of substrate removal in a biofilm may be limited by the rate that oxygen can be supplied by diffusion from the bulk liquid. Higher bulk liquid DO concentrations than used in suspended growth processes must be provided to realize the full removal rate potential of an attached growth biofilm.
Recirculation rate	The rate at which treated wastewater is returned to the process influent to dilute the organic concentration and to provide more optimal hydraulics to maintain the viability of the microorganisms in nonsubmerged attached growth processes.

Term	Definition
Rotating biological contactor (RBC)	A fixed-film biological treatment device in which microorganisms are grown on circular plastic disks mounted on a horizontal shaft that rotates slowly while partially immersed in wastewater.
Submerged attached growth processes	Aerobic submerged fixed film processes involving three phases: a support material, the attached biofilm, and liquid to be treated.
Trickling filter	A nonsubmerged aerobic fixed-film biological reactor that uses rock or a plastic packing over which wastewater is distributed for treatment.
Trickling filter/activated-sludge process (TF/AS)	A sequential combined attached and suspended growth biological process in which the influent wastewater is treated in a highly loaded trickling filter prior to activated sludge treatment for production of a high quality effluent.
Trickling filter/solids-contact process (TF/SC)	A sequential combined attached and suspended growth biological process. The main differences between the TF/SC and TF/AS processes are in the trickling filter loading and activated sludge SRT values.

The concept of attached growth processes was introduced in Chap. 7, along with the fundamental mechanisms of mass transfer of substrate and electron acceptors into the biofilms that develop in attached growth systems. In this chapter, various aerobic attached growth processes used for wastewater treatment are introduced and discussed. Following a brief introduction to the general characteristics of attached growth processes, separate sections are devoted to (1) nonsubmerged attached growth aerobic processes, (2) sequential combined trickling filter-activated sludge processes, (3) activated sludge processes with various media for attached growth, (4) submerged attached growth aerobic processes with suspended media, (5) submerged fixed bed attached growth processes, and (6) attached growth processes used for biological denitrification.

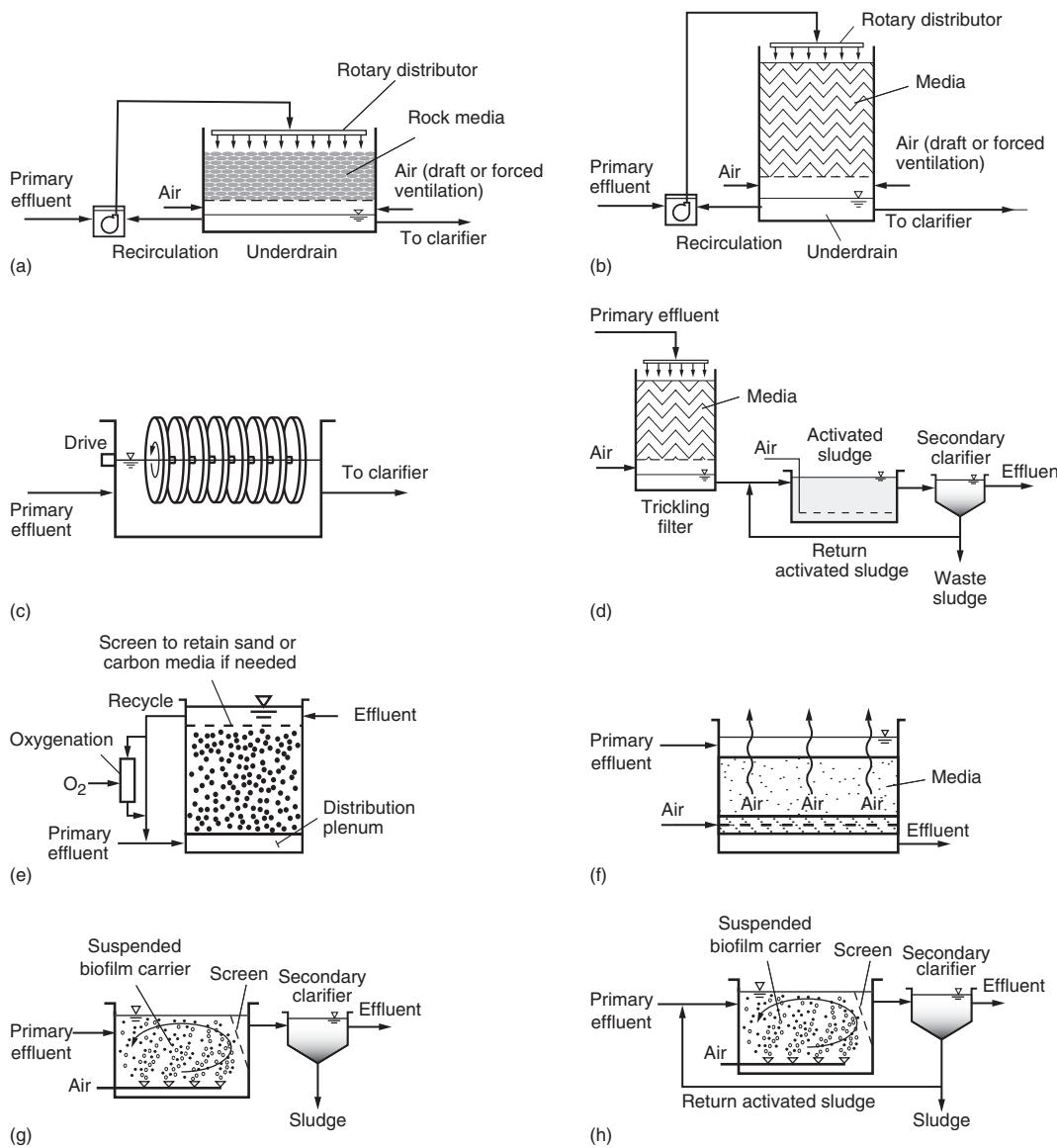
9-1 INTRODUCTION TO ATTACHED GROWTH PROCESSES

To introduce attached growth processes, it will be helpful to review briefly the types of processes and their evolution, and to consider the importance of mass transfer on their performance.

Types of Attached Growth Processes

Attached growth aerobic processes can be grouped into five general categories as illustrated on Fig. 9-1: (1) nonsubmerged attached growth aerobic processes, (2) partially submerged attached growth aerobic processes, (3) sequential nonsubmerged attached growth-activated sludge process (4) submerged attached growth aerobic processes, and (5) activated sludge processes with biofilm carriers. Features that are common to each of these processes include:

1. Growth of the biomass on a fixed media
2. Removal of excess solids by sedimentation after solids sloughing off the fixed film media or by backwashing of the media
3. Need to provide oxygen by either air movement through the void volume in nonsubmerged processes or by air sparging into fixed or moving submerged media, or oxygenation of recycle flow in a fluidized bed reactor
4. Need to provide distribution and contact of the influent flow with the media surface area
5. Need for an underdrain or other methods of collecting the treated effluent

**Figure 9-1**

Schematics of common attached growth processes: (a) nonsubmerged attached growth shallow depth rock trickling filter, (b) nonsubmerged attached growth tower type trickling filter with plastic media, (c) nonsubmerged attached growth rotating biological contactor, (d) combined trickling filter/activated sludge process, (e) submerged upflow fixed media, (f) submerged downflow fixed media, (g) submerged moving bed attached growth bioreactor, and (h) combined submerged attached growth-activated sludge.

As the media size gets smaller, the area for biomass growth per unit of reactor volume increases and the reactor volume can become smaller. However, the oxygen transfer rate needed per unit volume increases, which cannot be supplied by forced air ventilation or natural draft air flow. Thus, a submerged media with air sparging is used in systems with media that have high surface area to volume ratios. A brief historical background is provided below for these processes.

Nonsubmerged Attached Growth Processes. The principal processes considered under this heading are the many variations of the trickling filter process [see Figs. 9–1(a) and (b)]. Trickling filters with rock packing [see Fig. 9–1(a)] have been a common, simple, and low-energy process used for secondary treatment since the late 1800s. A trickling filter is a non-submerged fixed film biological reactor using rock or plastic packing over which wastewater is distributed continuously. Treatment occurs as the liquid flows over the attached biofilm. The concept of a trickling filter grew from research at the Lawrence Experimental Station in Massachusetts, established in 1887, and the use of contact filters in England in the early 1890s (Alleman, 1982). Work at Lawrence confirmed wastewater treatment in an intermittent dosed sand filter, while treatment in watertight basins filled with broken stones with a cyclic operating mode was demonstrated in England. The bed was filled with wastewater from the top, and the wastewater was allowed to contact the packing for a short time. The bed was then drained and allowed to rest before the cycle was repeated. A typical cycle required 12 h (6 h of operation and 6 h of resting). The limitations of the contact filter included a relatively high incidence of clogging, the long rest period required, and the relatively low loading that could be used. Because of the clogging problems, larger packing was used until a rock size 50 to 100 mm (2 to 4 in.) was reached. Rotary distributor designs driven by water jets were developed at the Lawrence Experimental Station in 1894 to allow continuous uniform distribution of wastewater to the trickling filter (WEF, 2011).

In the early 1950s, Dow Chemical developed a plastic packing media, termed *Surfpak*, that enabled the use of taller filters (also known as biotowers), higher void volume, and better ventilation with less land area needs and improved process efficiency [see Fig. 9–1(b)] (Bryan, 1955). In the United States, plastic media has essentially replaced rock media in nonsubmerged attached growth processes; however, rock is still used in many parts of the world.

Partially Submerged Attached Growth Processes. In the mid-1960s, Allis Chalmers Corporation started investigating the use of metal rotating discs for wastewater treatment at the Jones Island treatment plant in Milwaukee. Soon after, they licensed a similar process termed *Bio-Disc* from a German company, which used polystyrene discs. However, in 1972 the process was passed to Autotrol Corporation, which introduced a new rotating biological contactor (RBC) media made from corrugated sheets of polyethylene. In the RBC attached growth process the packing is rotated in the wastewater treatment tank in contrast to pumping and applying the wastewater over a vertical static packing [see Fig. 9–1(c)].

The first RBC unit in the United States was for a small cheese factory in 1969 and its application became widespread in the United States through the 1970s (Alleman, 1982). However, problems with lower performance at design loadings, excess biomass accumulations on the discs due to the development of an undesirable type of biological growth, shaft breakage, and media deterioration on the disks has resulted in very few new installations over the last 20 years. Because of their relatively infrequent application, RBCs are not considered further in this chapter. The use of RBCs for the treatment of return flows is considered in Chap. 15. A complete discussion of the RBC process including design equations and examples may be found in the fourth edition of this textbook (Tchobanoglous et al., 2003). Process design guidelines are also provided in a U.S. EPA report on RBCs (U.S. EPA 1984).

Sequential Nonsubmerged Combined Attached Growth-Activated Sludge Process. Trickling filters have also been used before activated sludge in a coupled process to utilize the benefits of both processes, in terms of energy savings and

effluent quality [see Fig. 9–1(d)]. The first sequential *trickling filter/activated sludge* systems were applied for the treatment of industrial wastewaters or high strength combined domestic/industrial wastewaters. The nonsubmerged attached growth process was used for partial BOD removal without solids clarification followed by activated sludge treatment for final treatment. The upstream attached growth process greatly improved the activated sludge settling characteristics with low SVIs, and the energy needed for BOD removal in the attached growth process was 20 to 40 percent of that needed per kg of BOD removal in the activated sludge process (Biesinger et al., 1980). Other combined trickling filter processes are considered in Sec. 9–3.

Another sequential trickling filter and activated sludge system is the *trickling filter/solids contact process*, which was developed in the late 1970s from research in Corvallis, OR, aimed at a higher quality effluent after trickling filter treatment of domestic wastewater (Norris et al., 1982). In this case the trickling filter was designed to remove most of the soluble BOD, and was followed by an aerated solids contact channel also receiving return activated sludge from secondary clarifiers. The primary purpose of the aerated solids contact channel was to flocculate trickling filter effluent suspended solids in the activated sludge.

Submerged Attached Growth Aerobic Processes. Beginning in the 1970s and extending into the 1980s, a new class of aerobic attached growth processes became established alternatives for biological wastewater treatment. These are upflow, downflow, and fluidized bed reactors with fixed or moving media that do not use secondary clarification or sparged aeration reactors. Work by Jeris et al. (1977) developed an upflow fluidized bed reactor that was applied for BOD removal [see Fig. 9–1(e)] and also for denitrification in an anoxic reactor. The first downflow submerged fixed media bed system [see Fig. 9–1(f)] using a fired clay media was developed from research work in Paris, France (Leglise et al., 1980). Later, upflow submerged fixed media bed systems were developed. In the late 1980s a moving bed bioreactor (MBBR), containing a mixed and aerated high density polyethylene biofilm media in a submerged horizontal flow tank [see Fig. 9–1(g)] was developed in Norway (Ødegaard, 2006).

The unique advantage of the submerged attached growth system is the small footprint required with an area requirement that is a fraction (one-fifth to one-third) of that needed for activated sludge treatment. Other advantages compared to activated sludge are the ability to handle dilute wastewaters and the avoidance of activated sludge settling concerns. Though they are more compact, their capital costs are generally higher than that for activated sludge treatment. In addition to BOD removal, submerged attached growth processes have also been used for tertiary nitrification and denitrification following suspended or attached growth nitrification.

Activated Sludge Processes With Fixed Film Carriers. The placement of material for attached growth in the aeration tank of the activated sludge process dates back to the 1940s with the Hays and Griffith processes (WEF, 2011), in which cement asbestos or wood baffles were added to activated sludge aeration tanks. Present day designs use more engineered materials and include the use of synthetic media that are suspended in the aeration tank with the mixed liquor, fixed synthetic material placed in portions of the aeration tank, and submerged RBCs. This combined attached growth/activated sludge process [see Fig. 9–1(h)] has also been referred to as a *hybrid* process or *integrated fixed-film activated sludge* (IFAS) process. The advantages claimed for these activated sludge process enhancements are as follows:

1. Increased treatment capacity
2. Greater process stability

3. Reduced sludge production
4. Enhanced sludge settleability
5. Reduced solids loadings on secondary clarifier
6. No increase in operation and maintenance costs

Mass Transfer Limitations in Attached Growth Processes

A significant process feature of attached growth processes in contrast to activated sludge treatment is the fact that the performance of biofilm processes is often diffusion limited. Substrate removal and electron donor utilization occurs within the depth of the attached growth biofilm and subsequently the overall removal rates are a function of diffusion rates and the electron donor and electron acceptor concentrations at various locations in the biofilm. By comparison, the process kinetics for the activated sludge process are generally characterized by the bulk liquid concentrations.

The diffusion-limited concept is especially important when considering the measurable bulk liquid DO concentrations on attached growth process biological reaction rates. Where a DO concentration of 2 to 3 g/m³ is generally considered satisfactory for most suspended growth aerobic processes, such low DO concentrations can be limiting for attached growth processes. For uninhibited nitrification in the biofilm a much higher DO concentration may be required, as shown in Sec. 7-7, depending on the ammonia-N concentration.

The concept of diffusion limitations on nitrification rates and the ability to develop anaerobic layers within the biofilm may be exploited to accomplish both nitrification and denitrification in attached growth processes with positive bulk liquid DO concentrations. Investigators have shown how aerobic and anaerobic layers can be developed in the biofilm to accomplish nitrogen removal by nitrification and denitrification (Chui et al., 1996, Richter et al., 1994, and Meaney et al., 1994).

9-2 NONSUBMERGED ATTACHED GROWTH PROCESSES

Trickling filters and variants are the principal nonsubmerged attached growth processes used for aerobic biological treatment of wastewater. The objective of this section is to consider the fundamentals of the trickling process, important design considerations, and the implementation of trickling filters for BOD removal and nitrification. Topics considered in this section include (1) general process description, (2) trickling filter classification and application, (3) advantages and disadvantages of trickling filters, (4) physical facilities for trickling filters, (5) operational considerations (6) process analysis for BOD removal, (7) process analysis for BOD removal and nitrification design, and (8) process analysis for tertiary nitrification design. The first series of topics (1 through 5) deal with a general description of the characteristics and physical features of trickling filters. The focus of the last three topics (6, 7, and 8) is with the important process analysis considerations for trickling filters and their application for the removal of BOD and nitrification with or following BOD removal.

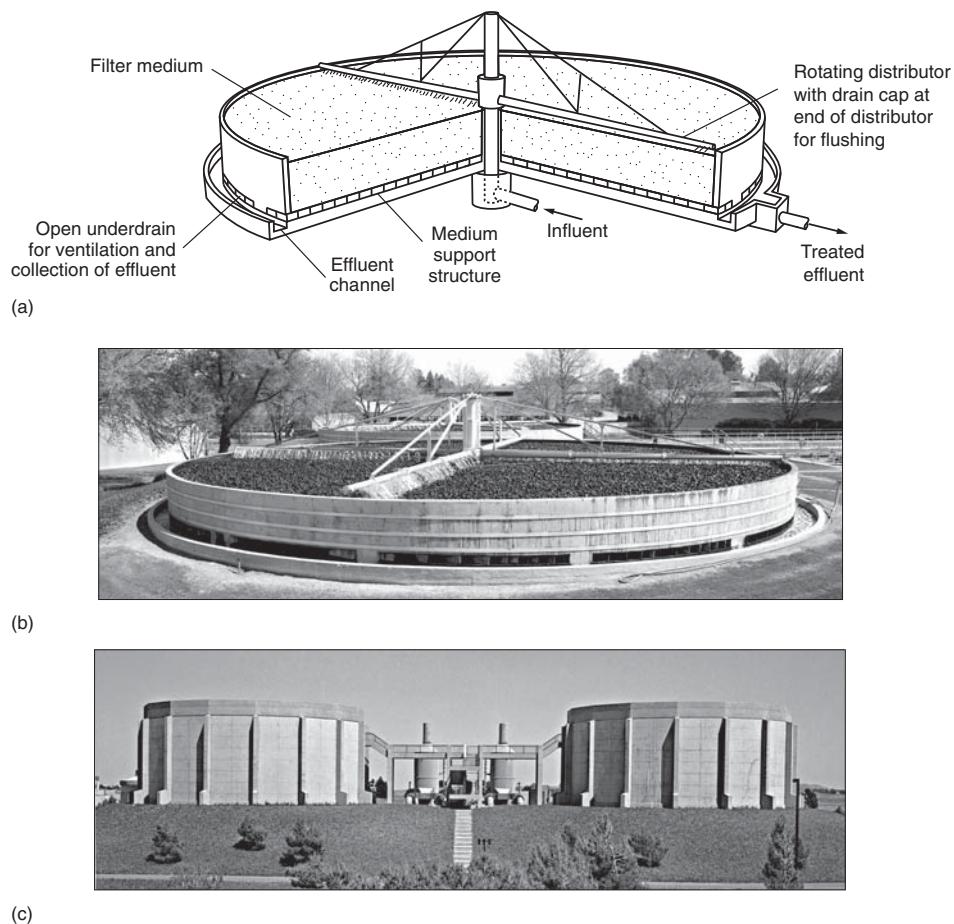
General Process Description

As noted previously, the trickling filter is a nonsubmerged fixed film biological reactor using rock or plastic packing over which wastewater is distributed continuously. The physical features and functional description of the trickling filter process are introduced in this section, and considered further subsequently.

Physical Features. The three principal features of trickling filters, as illustrated on Fig. 9-2(a), are (1) the filter media, (2) the wastewater distribution system, and (3) the underdrain system.

Figure 9–2

Typical trickling filters:
 (a) cutaway view of a rock
 trickling filter (b) view of
 conventional rock-filled trickling
 filter, and (c) view of tower
 trickling filters with plastic
 filter media.



Filter Media. The ideal filter packing is a material that has a high specific surface area ($\text{m}^2 \text{ exposed surface}/\text{m}^3 \text{ bulk volume}$), is low in cost, has a high durability, has a high enough porosity to avoid clogging, and allows good air ventilation by natural draft or low pressure blowers. The media serves as a structure for the attachment and growth of a biofilm. Treatment occurs as the liquid flows over the attached biofilm. Many conventional trickling filters using rock as the packing material [see Fig. 9–2(b)] have been converted to plastic packing to increase treatment capacity. Virtually all new trickling filters in the United States are now constructed with plastic packing in a tower arrangement [see Fig. 9–2(c)]. Primary clarification is used before application of wastewater to the trickling filter media to prevent clogging, although in some cases primary clarification has been omitted where fine screens are used.

Distribution System. The distributor is used to apply the wastewater to be treated uniformly over the surface of the filter media. The flow-driven rotary distributor for trickling filtration has become a standard for the process because it is reliable and easy to maintain. A distributor consists of two or more arms that are mounted on a pivot in the center of the filter and revolve in a horizontal plane [see Fig. 9–2(b)]. The arms are hollow and contain nozzles through which the wastewater is discharged over the filter bed. The distributor assembly may be driven either by the dynamic reaction of the wastewater discharging from the nozzles or by an electric motor.

Underdrain System. The functions of the underdrain system are twofold: (1) to collect wastewater that has passed through the filter, along with any biofilm that has sloughed off

from the media, and (2) to provide an open area for the movement of air, either by natural draft or by forced air, needed for treatment of the applied wastewater.

Functional Description. Functionally, trickling filter media is covered with a very visible *biofilm* as a result of the BOD removal. In the literature, the biofilm is often identified as a *slime layer* (also a *zoogaeal film*), especially in those cases where an especially thick slimy biofilm layer develops. Soluble substrates and dissolved oxygen diffuse into the biofilm to support biomass growth. During the time when the wastewater flow is not applied or the *resting period*, oxygen continues to diffuse into the biofilm. Colloidal and particulate substances are removed by attachment and trapping in the biofilm. Effluent solids that become detached from the biofilm support are removed in a secondary clarifier, but clarifier underflow solids are not recycled to the trickling filter as they are in the activated sludge process. Recirculation of the trickling filter effluent to the trickling filter influent is done to maintain wetting of the media at low flows, and because it also enhances the supply of oxygen and treatment performance.

Biofilm Development. Depending on operating conditions, the thickness of the biofilm can reach 10 mm. In the outer portions of the biofilm (0.1 to 0.2 mm), the organic material is degraded by aerobic microorganisms. As the microorganisms grow and the biofilm thickness increases, oxygen is consumed before it can penetrate the full depth, and an anaerobic environment is established near the surface of the packing. As the biofilm thickness increases, the substrate in the wastewater is used before it can penetrate the inner depths of the biofilm, where the biofilm then enters an endogenous respiration state and lose their ability to cling to the packing surface.

Sloughing. When the microorganisms can no longer cling to the media, the applied liquid washes the biofilm off the packing, and a new biofilm layer starts to grow. The phenomenon of losing a portion of a biofilm layer is called *sloughing* and is primarily a function of the organic and hydraulic loading on the filter. The hydraulic loading affects the shear velocities, and the organic loading affects the rate of metabolism and rate of biofilm growth. Methods to control the hydraulic loading and sloughing are discussed subsequently.

Process Microbiology. The biological community in the biofilm is very diverse and includes aerobic and facultative bacteria, protozoan, fungi, and algae. Higher life forms, such as worms, insect larvae, and snails, are also present. Facultative bacteria are the predominating organisms in trickling filters, and decompose the organic material in the wastewater along with aerobic and anaerobic bacteria. *Achromobacter*, *Flavobacterium*, *Pseudomonas*, and *Alcaligenes* are among the bacterial species commonly associated with the trickling filter. The fungi present are also responsible for waste stabilization, but their role is usually important only under low-pH conditions or with certain industrial wastes. At times, their growth can be so rapid that the filter clogs and ventilation becomes restricted. Among the fungi species that have been identified are *Fusarium*, *Mucor*, *Pencillium*, *Geotrichum*, *Sporatichum*, and various yeasts (Hawkes, 1963; Higgins and Burns, 1975). For low-loaded trickling filters where little organic substrate remains nitrifying bacteria will be established on the media. At higher organic substrate concentrations the heterotrophic bacteria outcompete the nitrifying bacteria for growth on the media surface area due to their faster growth rates and higher biomass yields.

Filamentous Bacteria. When adverse dissolved oxygen conditions prevail within the biofilm due to high organic loading, the filamentous bacteria forms *Sphaerotilus natans* and *Beggiatoa* will be found. The *Beggiatoa* are favored due to their ability to oxidize hydrogen sulfide and other reduced organic substances that can be produced within anaerobic layers in the depth of the biofilm.

Presence of Higher Forms. Higher life forms, such as protozoa, worms, snails, and insects feed on the biological film. The protozoa in the filter are predominantly of the ciliate group, including *Vorticella*, *Opercularia*, and *Epistylis* (Hawkes, 1963; Higgins and Burns, 1975). Their function is to feed on the biological films and free bacteria to decrease effluent turbidity and help keep the film in a higher growth state. Snails are especially troublesome in trickling filters used mainly for nitrification, where they have been known to consume enough of the nitrifying bacteria to significantly reduce treatment efficiency (Timpany and Harrison, 1989).

Presence of Algae. Algae can grow only in the upper reaches of the filter where sunlight is available. *Phormidium*, *Chlorella*, and *Ulothrix* are among the algae species commonly found in trickling filters (Hawkes, 1963; Higgins and Burns, 1975). Generally, algae do not take a direct part in waste degradation, but during the daylight hours they add oxygen to the percolating wastewater. From an operational standpoint, the algae may be troublesome because they can cause clogging of the media surface, which produces odors.

Trickling Filter Classification and Applications

Trickling filters are often classified according to the applied organic loading rates expressed as kg BOD applied/m³ bulk media volume·d (in US Customary units as lb BOD/1000 ft³·d). Trickling filter applications and loadings, based on historical terminology developed originally for rock filter designs, are summarized in Table 9–1. As shown in Table 9–1, trickling

Table 9–1

Trickling filter process applications and representative design parameter when treating primary effluent^a

Design parameter	Unit	Low rate BOD removal	High rate BOD removal	High rate BOD removal	BOD removal and nitrification	Partial BOD removal
BOD removal efficiency	%	80–90	80–90	70–90	85–90	40–70
Type of packing		Rock	Rock	Plastic	Plastic (P)/rock (R)	Plastic
Ventilation	Type	Natural	Forced air	Forced air	Forced air	Forced air
Organic loading	kg BOD/m ³ ·d (lb BOD/10 ³ ft ³ ·d)	0.08–0.3 (5–20)	0.6–1.6 (40–100)	0.6–2.4 (50–150)	0.08–0.4 (5–25)	1.6–3.5 (100–220)
Hydraulic loading	m ³ /m ² ·d (gal/ft ² ·d)	1–4 (25–100)	4–40 (100–1000)	15–75 (350–1850)	5–16 (125–400)	40–100 (1000–2500)
Recirculation ratio	Q _r /Q	0–1	1–2	1–2	1–2	0–2
Depth	m (ft)	1–2.5 ^b (3–8)	1–2.5 ^b (3–8)	3–12 (8–40)	P, 3–12 (8–40) R, 1–2.5 (3–8)	0.9–6 (3–20)
Effluent quality	BOD, mg/L	<30	<30	<30	<20	>30
	NH ₄ -N, mg/L	<5	>5	>5	<3	

^a Adapted in part from Tchobanoglou et al. (2003) and Daigger and Boltz (2011).

^b Depth is limited due the weight of the rock.

Note: kg/m³·d × 62.4280 = lb/10³ ft³·d

m³/m²·d × 24.5424 = gal/ft²·d

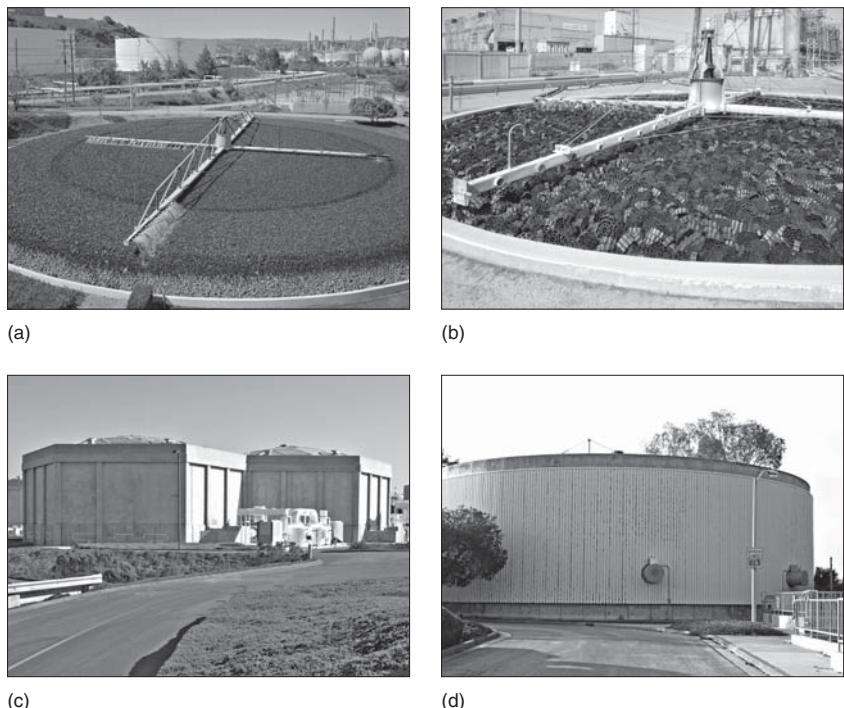
filter applications and the corresponding loading rates will vary depending on whether the treatment objective is (a) BOD removal with natural draft aeration, (b) BOD removal with forced draft aeration, (c) BOD removal and nitrification with forced draft aeration, and (d) partial BOD removal, or *roughing* treatment with forced draft aeration. Each type of loading is considered briefly in the following discussion.

BOD Removal with Natural Draft Aeration. Historically, rock trickling filters have been designed and operated with natural draft aeration. Because of low air movement during parts of the day when the temperature of the wastewater and air are within 1.7°C (3°F) of each other, the organic loading rates were low. These filters are often identified in the literature as *low rate* filters. Although relatively simple, a low rate filter is a highly dependable device that produces an effluent of consistent quality with an influent of varying strength. Low-rate filters with rock as the filter medium are usually operated without recirculation, although some recirculation has been used to keep the filters moist during low flow periods. Circular filters are the most common shape but rectangular and polygonal shapes have been used. In some cases, filters have been constructed without a surrounding enclosure [see Fig. 9-3(a), see also Fig. 7-15 in Chap. 7]. Often rock is replaced with plastic packing material [see Fig. 9-3(b)].

At lower organic loading rates, in the range of $0.07\text{--}0.25 \text{ kg BOD/m}^3\cdot\text{d}$, secondary treatment is achieved with 85 to 90 percent BOD removal. In most low-rate filters, only the top 0.6 to 1.2 m (2 to 4 ft) of the filter packing will have appreciable biofilm. As a result, the lower portions of the filter may be populated by autotrophic nitrifying bacteria, which oxidize ammonia nitrogen to nitrite and nitrate forms. Heterotrophic bacteria, with higher yield coefficients and faster growth rates, are more competitive than nitrifying bacteria for space on the fixed film packing. Thus, significant nitrification only occurs after the BOD concentration is reduced appreciably. Based on soluble BOD, Harremoes (1982) concluded that a concentration of less than 20 mg/L is needed to initiate nitrification. Low rate filters are seldom used these days, primarily due to odor issues and space constraints.

Figure 9-3

Typical examples of trickling filters: (a) conventional shallow-depth, rock trickling filter without containment structure (see also Fig. 7-15 in Chap. 7), (b) conventional shallow-depth filter in which the original rock media (see Table 9-2) has been replaced with random pack plastic media (see Table 9-2), (c) multisided tower trickling filter employing plastic media (air pollution control equipment shown in foreground), and (d) tower trickling filter with fans for forced air aeration located around periphery of the filter.



BOD Removal with Forced Draft Aeration. Significantly higher organic loading rates (see Table 9–1) can be achieved with forced draft aeration. Although either rock or plastic packing can be used, the current trend is to use a plastic packing material in tall circular towers [see Figs. 9–3(c) and (d)]. The use of plastic packing allows for the construction of deeper filters with less land area required. Recirculation of the trickling filter effluent permits higher organic loadings, provides higher dosing rates on the filter to improve the liquid distribution and better control of the biofilm thickness, provides more oxygen in the influent wastewater flow, and returns viable organisms [see Figs. 9–4(a) and (b)]. It should be noted that other recirculation patterns have been used (Tchobanoglous et al., 2003).

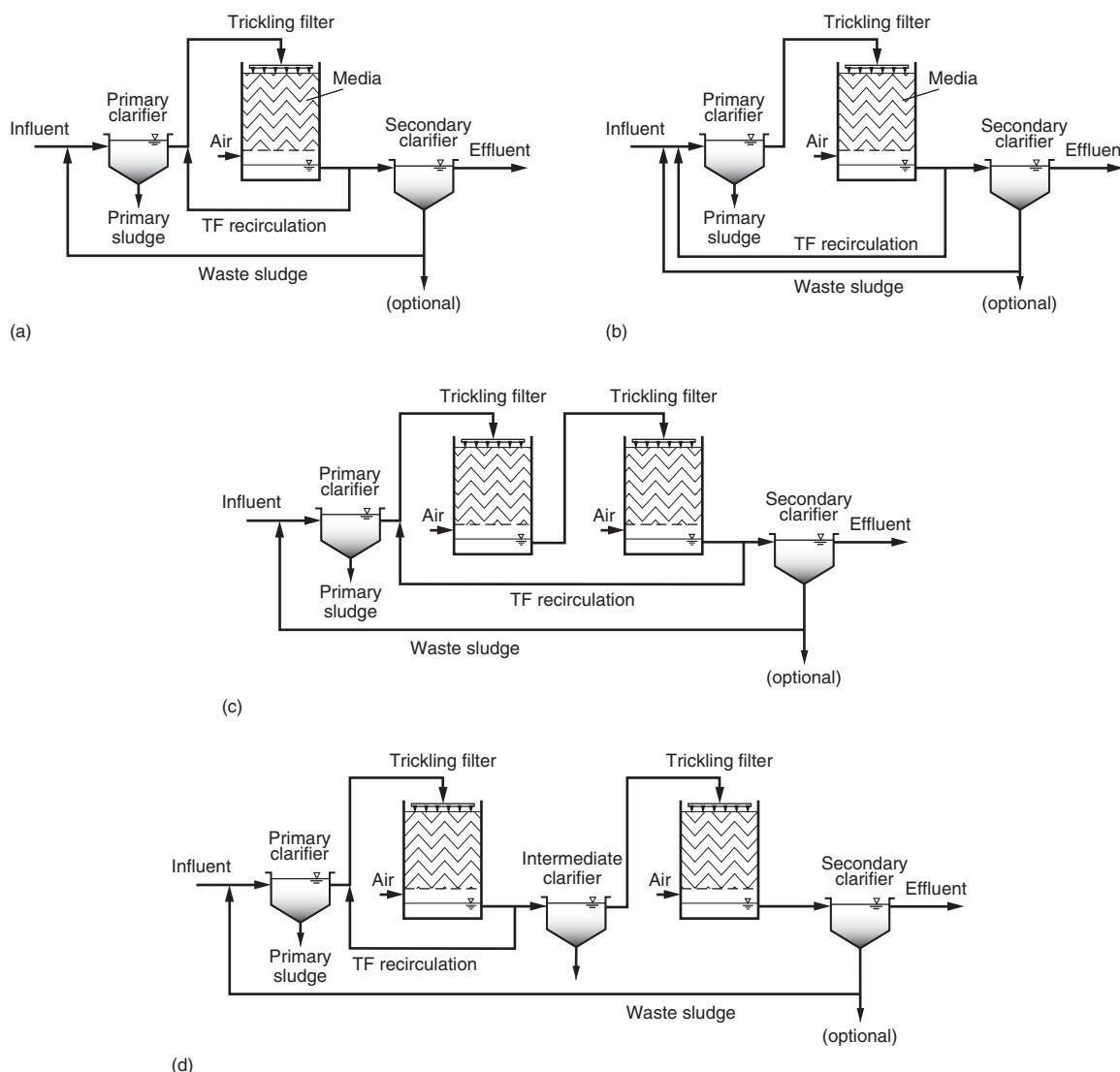


Figure 9–4

Typical trickling filter process schemes: (a) single trickling filter with waste sludge to the primary clarifier, (b) single trickling filter with recirculation and waste sludge to the primary clarifier, (c) two-stage trickling filter with waste sludge to the primary clarifier, and (d) two-stage trickling filter with intermediate clarifier. Note that many other flow schemes have been used (Tchobanoglous et al., 2003).

Recirculation also helps prevent ponding in the filter and to reduce the nuisance from odors and flies.

BOD Removal and Nitrification with Forced Draft Aeration. Both BOD removal and nitrification can be accomplished in rock or plastic packing trickling filters operated at reduced organic loading rates (Stenquist et al., 1974, Parker and Richards, 1986). As the BOD loading rate is decreased to what has been historically referred to as a *low-rate* trickling filter, as discussed above, nitrification also occurs in the same trickling filter or in the second stage of a system with two trickling filters in series [see Figs. 9-4(c) and (d)]. Plastic or rock media may be used in these trickling filters.

The hydraulic loading decreases with lower organic loadings. At very low hydraulic loading rates problems can occur due to improper media wetting and filter fly growth, unless the design adequately addresses filter effluent recirculation flows. Recirculation ratios (Q_R/Q) used range from 0.5 to 4.0. Vertical-flow media require an average total hydraulic loading exceeding $1.8 \text{ m}^3/\text{m}^2\cdot\text{h}$ to maximize wetting and the BOD removal efficiency, and total hydraulic loadings in the range 0.4 to $1.1 \text{ m}^3/\text{m}^2\cdot\text{h}$ have been used in shallow towers with cross-flow media.

Partial BOD Removal with Forced Draft Aeration. At organic loading rates typically greater than $1.6 \text{ kg BOD/m}^3\cdot\text{d}$, trickling filters have been used to achieve partial BOD removal in the range of 50 to 70 percent. These highly loaded filters have been identified in the literature as *roughing* (or *high-rate*) filters. Most roughing filters are designed using plastic media (WEF, 2011). A roughing filter can be an attractive alternative for the pretreatment of modest to high strength industrial wastewaters. One of the advantages of roughing filters is the low energy requirement for BOD removal of higher strength wastewaters as compared to activated sludge aeration. Because the energy required is only for pumping the influent wastewater and recirculation flows, the amount of BOD removal per unit of energy input can increase as the wastewater strength increases until more recirculation is needed. The energy requirement for a roughing application may range from 2 to 5 kg BOD applied/kWh versus 1.2 to 2.4 kg BOD/kWh for activated sludge treatment.

Advantages and Disadvantages of Trickling Filters

Trickling filters, as noted above, have been used as aerobic attached growth processes for BOD removal only, combined BOD removal and nitrification, and for tertiary nitrification after secondary treatment by suspended growth or attached growth processes. The advantages and disadvantages of the use of trickling filters are as follows.

Advantages. The principal advantages claimed for trickling filters over the activated sludge process are as follows:

1. Less energy required
2. Simpler operation with no issues of mixed liquor inventory control and sludge wasting
3. No problems of bulking sludge in secondary clarifiers
4. Better sludge thickening properties
5. Less equipment maintenance needs
6. Better recovery from shock toxic loads

Disadvantages. Many disadvantages often cited for trickling filters, such as a poorer effluent quality in terms of BOD and TSS concentrations, greater sensitivity to lower temperatures, odor production, and uncontrolled solids sloughing events, are related more to the specific process and final clarifier designs than to the actual process capabilities

(WEF, 2011). In general, the actual limitations of the processes are the difficulty in accomplishing biological nitrogen and phosphorus removal compared to single-sludge biological nutrient removal suspended growth designs, and the production of an effluent with a higher suspended solids concentration than activated sludge treatment.

Physical Facilities for Trickling Filters

Factors that must be considered in the design of trickling filters include (1) type and physical characteristics of filter packing to be used, (2) type and dosing characteristics of the distribution system, and (3) configuration of the underdrain system. The dosing rate, the provision of adequate airflow (i.e., ventilation) by either natural or forced air and settling tank design are considered under design considerations.

Filter Packing Material. Typical trickling filter packing materials, including rock, random pack plastic media, and cross- and vertical-flow plastic bundles, are shown on Fig. 9–5. Redwood slates have also been used in the past, but are seldom used today. The physical characteristics of commonly used trickling filter packings, including those shown on Fig. 9–5, are reported in Table 9–2. Since the 1960s, plastic packing material, either cross-flow or vertical-flow, has become the packing of choice in the United States.

Rock. Where locally available and cost effective, rock media may be used. The most suitable material is rounded river rock or crushed stone, graded to a uniform size so that 95 percent is within the range of 75 to 100 mm (3 to 4 in.). The specification of size uniformity is a way of ensuring adequate pore space for wastewater flow and air circulation.

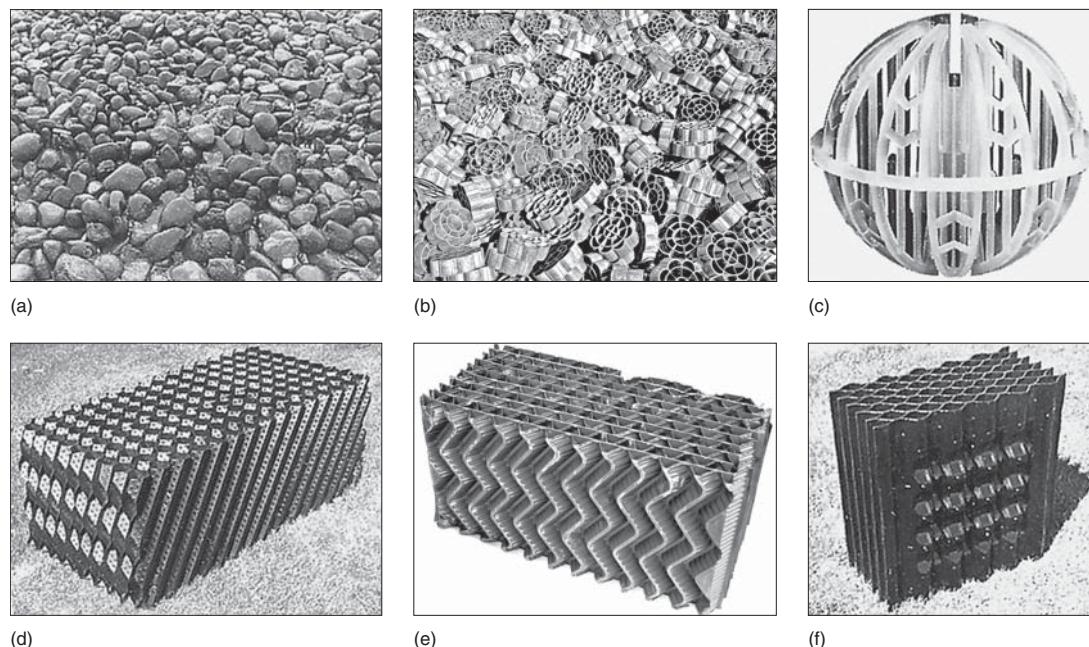


Figure 9–5

Typical packing material for trickling filters: (a) rock, (b) and (c) random pack plastic media, (d) and (e) plastic cross-flow bundle, and (f) plastic vertical-flow bundle. Media characteristics are summarized in Table 9–2.

13

Processing and Treatment of Sludges

13-1	SLUDGE SOURCES, CHARACTERISTICS, AND QUANTITIES	1453
	<i>Sources</i>	1453
	<i>Characteristics</i>	1454
	<i>Quantities</i>	1456
13-2	REGULATIONS FOR THE REUSE AND DISPOSITION OF SLUDGE IN THE UNITED STATES	1461
	<i>Land Application</i>	1461
	<i>Surface Disposition</i>	1462
	<i>Pathogen and Vector Attraction Reduction</i>	1462
	<i>Incineration</i>	1463
13-3	SLUDGE PROCESSING FLOW DIAGRAMS	1466
13-4	SLUDGE AND SCUM PUMPING	1467
	<i>Pumps</i>	1467
	<i>Headloss Determination</i>	1475
	<i>Sludge Piping</i>	1480
13-5	PRELIMINARY SLUDGE PROCESSING OPERATIONS	1481
	<i>Grinding</i>	1481
	<i>Screening</i>	1482
	<i>Degritting</i>	1482
	<i>Blending</i>	1483
	<i>Storage</i>	1484
13-6	THICKENING	1486
	<i>Application</i>	1486
	<i>Description and Design of Thickeners</i>	1487
13-7	INTRODUCTION TO SLUDGE STABILIZATION	1497
13-8	ALKALINE STABILIZATION	1498
	<i>Chemical Reactions in Lime Stabilization</i>	1498
	<i>Heat Generation</i>	1499
	<i>Application of Alkaline Stabilization Processes</i>	1500
13-9	ANAEROBIC DIGESTION	1502
	<i>Process Fundamentals</i>	1503
	<i>Description of Mesophilic Anaerobic Digestion Processes</i>	1504
	<i>Process Design for Mesophilic Anaerobic Digestion</i>	1506
	<i>Selection of Tank Design and Mixing System</i>	1512
	<i>Methods for Enhancing Sludge Loading and Digester Performance</i>	1520
	<i>Gas Production, Collection, and Use</i>	1520
	<i>Digester Heating</i>	1525
	<i>Advanced Anaerobic Digestion</i>	1530

<i>Sludge Pretreatment for Anaerobic Digestion</i>	1533
<i>Co-digestion with Other Organic Waste Material</i>	1538

13-10 AEROBIC DIGESTION	1541
<i>Process Description</i>	1542
<i>Conventional Air Aerobic Digestion</i>	1544
<i>Dual Digestion</i>	1549
<i>Autothermal Thermophilic Aerobic Digestion (ATAD)</i>	1549
<i>Improved ATAD Systems</i>	1553
<i>High-Purity Oxygen Digestion</i>	1553
PROBLEMS AND DISCUSSION TOPICS	1554
REFERENCES	1556

WORKING TERMINOLOGY

Term	Definition
Acid-gas digestion	A modified process of anaerobic digestion where the acid phase hydrolysis is separated from the gas producing phase for increased volatile solids reduction.
Aerobic digestion	Biological stabilization process operated in the presence of oxygen in which the biodegradable matter in primary and secondary sludge is oxidized to carbon dioxide and other end products.
Anaerobic digestion	Biological stabilization process operated in the absence of oxygen in which the biodegradable matter in primary and secondary sludge is converted to methane, carbon dioxide, and other end products.
Autothermal thermophilic aerobic digestion (ATAD)	An aerobic digestion process in which the microbes generate enough heat to maintain temperatures in the thermophilic range. When maintained for enough time to meet 40CFR 503 requirements, the process results in biosolids that are relatively pathogen free and meet Class A standards.
Biosolids	Sludge from wastewater treatment processes that has been stabilized to meet the criteria in the U.S. EPA's 40 CFR 503 regulations and, therefore, can be used beneficially.
Class A biosolids	Biosolids that contain less than 1000 most probable number (MPN)/g of fecal coliforms and less than 3 MPN/4g of <i>Salmonella</i> bacteria and meet one of six stabilization alternatives given in 40 CFR 503. The material also must meet the pollutant limits and vector attraction reduction requirements set forth in 40 CFR 503.
Class B biosolids	Biosolids that contain less than 2 million colony-forming units (CFU) or most probable number (MPN) of fecal coliforms per gram of dry biosolids. The material also must meet the pollutant limits and vector attraction reduction requirements set forth in 40 CFR 503.
Digestion	The process of biologically degrading organic matter in sludge, thereby reducing the concentrations of volatile solids and pathogens.
Disposition	Disposition is a term used to reflect disposal of biosolids or sludge for either beneficial or non-beneficial use due to the value of the material.
Dissolved air flotation	A clarification process in which small air bubbles become attached to flocculated material, float to the surface, and are removed by skimming. Heavier solids which settle are removed by mechanical scrapers.
Dual digestion	A two-stage digestion process wherein the first stage is aerobic thermophilic digestion and the second stage is mesophilic anaerobic digestion. High-purity oxygen has also been used for the first stage.
Grit	Sand, gravel, cinders, other heavy inorganic materials and also organic matter such as eggshells, bone chips, seeds, and coffee grounds.

Term	Definition
Humus	Sludge removed from trickling filters.
Mesophilic anaerobic digestion	Anaerobic digestion that occurs in a temperature range of 30 to 38°C (85 to 100°F).
Methanogenesis	The metabolic conversion of organic acids or hydrogen and carbon dioxide to methane.
Screenings	The material removed from a screening device.
Scum	Buoyant materials (e.g., grease, food waste, paper, and foam) often found floating on the surface of primary and secondary clarifiers and thickeners.
Sidestream	A portion of the wastewater flow that has been diverted from the main treatment process flow for specialized treatment.
Solids	A term often used as a replacement for sludges that have not been stabilized by physical, chemical or biological treatment. The term solids is not used as a substitute for sludge in this chapter. The mass of dry material in sludge is referred to as the solids content.
Sludge	Any material (i.e., sludge) produced during primary, secondary, or advanced wastewater treatment that has not undergone any process to reduce pathogens or vector attraction.
Stabilization	A treatment process designed to reduce the number of pathogens in sludge and to reduce the attraction of vectors as defined in the requirements of 40 CFR 503.
Thermal hydrolysis	A thermal conditioning process, utilizing high-pressure steam for pretreating dewatered sludge prior to anaerobic digestion, that hydrolyzes and reduces the viscosity of the sludge.
Thermophilic anaerobic digestion	Anaerobic digestion that occurs in a temperature range of 50 to 57°C (122 to 135°F).

The constituents removed and/or produced in wastewater treatment plants include screenings, grit, scum, sludge, and biosolids. The sludge and biosolids (formerly collectively called sludge) resulting from wastewater treatment processes are usually in the form of a liquid or semisolid liquid, which typically contains from 0.25 to 12 percent solids by weight, depending on the operations and processes used. In the United States, the term *biosolids*, as defined by the Water Environment Federation (WEF 2010a), refers to any sludge that has been stabilized to meet the criteria in the U.S. Environmental Protection Agency's 40 CFR 503 regulations and, therefore, can be used beneficially. The term *sludge* is only used before beneficial use criteria (discussed in Sec. 14–2) have been achieved. The term *sludge* is generally used in conjunction with a process descriptor, such as *primary sludge*, *enhanced primary sludge*, *waste activated sludge*, and *secondary sludge*. Although the terms *solids* and *has been used as a substitute for sludge*, to avoid confusion only the terms *sludge*, as defined above, and *biosolids* are used in this chapter and book.

Of the constituents removed by treatment, sludge is by far the largest in volume, and its processing, reuse, and disposition present perhaps the most complex problem in the field of wastewater treatment. For this reason, two chapters have been devoted to this subject. The disposition of grit and screenings is discussed in Chap. 5. The problems of dealing with sludge are complex because (1) sludge is composed largely of the substances responsible for the offensive character of untreated wastewater; (2) the portion of sludge produced from biological treatment requiring disposition is composed of the organic matter contained in the wastewater but in another form, and it, too, will decompose and become offensive; and (3) only a small part of sludge is solid matter.

The purpose of this chapter is to describe the principal processes and methods used for sludge processing and treatment as identified in Table 13–1. Resource recovery

Table 13-1
Sludge handling and processing methods

Handling or processing method	Function	See Sec.
Pumping	Transport of sludge and biosolids	13-4
Preliminary operation		13-5
Grinding	Particle size reduction	13-5
Screening	Removal of fibrous material	13-5
Degritting	Grit removal	13-5
Blending	Homogenization of sludge	13-5
Storage	Flow equalization	
Thickening		
Gravity thickening	Volume reduction	13-6
Flotation thickening	Volume reduction	13-6
Centrifugation	Volume reduction	13-6
Gravity belt thickening	Volume reduction	13-6
Rotary drum thickening	Volume reduction	13-6
Stabilization		
Alkaline stabilization	Stabilization	13-8
Anaerobic digestion	Stabilization, mass reduction, resource recovery	13-9
Aerobic digestion	Stabilization, mass reduction	13-10
Composting	Stabilization, product recovery	14-5
Heat drying	Stabilization, volume reduction, resource recovery	14-3
Conditioning	Improve dewatering	14-1
Dewatering		
Centrifuge	Volume reduction	14-2
Belt filter press	Volume reduction	14-2
Rotary press	Volume reduction	14-2
Screw press	Volume reduction	14-2
Filter press	Volume reduction	14-2
Advanced dewatering	Volume reduction and stabilization	14-2
Drying beds	Volume reduction	14-2
Reed beds	Storage and volume reduction	14-2
Lagoons	Storage and volume reduction	14-2
Advanced Thermal Oxidation	Volume and mass reduction, resource recovery	14-4
Application of biosolids to land	Beneficial use and disposition	14-10
Conveyance and storage	Transport and storage of sludge and biosolids	14-6

methods, also identified in Table 13-1, and the beneficial use of the biosolids is discussed in Chapter 14. To understand the various sludge handling and processing methods, the first two sections of this chapter are devoted to a discussion of the sources, characteristics, and quantities of sludge; the current regulatory environment; and a presentation of representative sludge-treatment process flow diagrams. Because the pumping of sludge is a fundamental

part of wastewater treatment plant design, a separate discussion (Sec. 13–4) is devoted to sludge and scum pumping. The preliminary processing of sludge is discussed in Secs. 13–5 and 13–6. Stabilization of sludge is introduced in Sec. 13–7 and is divided into three subsequent sections for more detailed discussion: alkaline stabilization, anaerobic digestion, and aerobic digestion (see Secs. 13–8 through 13–10). Composting, also used for sludge stabilization after dewatering, is considered in Chap. 14.

13-1 SLUDGE SOURCES, CHARACTERISTICS, AND QUANTITIES

To design sludge processing, treatment, and disposition facilities properly, the sources, characteristics, and quantities of the sludge to be handled must be known. The method of primary and secondary treatment of wastewater has a significant impact on quantity and quality of the sludge produced. For example, using membrane bioreactors in secondary treatment produces sludge that is difficult to dewater and digest anaerobically, as compared to using a conventional waste activated sludge process. Stringent regulations for producing high quality effluent have an impact on the process used for secondary treatment, which in turn impact the quantity and quality of biosolids produced from sludge. For example, using biological nutrient removal (BNR) systems to meet stringent nutrient effluent quality produces lesser amount of sludge, but a sludge that is more difficult to process downstream by dewatering or digestion. The purpose of this section is to present background data and information on these topics that will serve as a basis for the material to be presented in the subsequent sections of this chapter.

Sources

The sources of sludge in a treatment plant vary according to the type of plant and its method of operation. The principal sources of sludge and the types generated are reported in Table 13–2. For example, in a complete mix activated sludge process, if the wasting of sludge is accomplished from the mixed liquor line or aeration chamber, the activated sludge

Table 13-2

Sources of sludge from conventional wastewater treatment plants^a

Unit operation or process	Types of sludge	Remarks
Preaeration	Grit and scum	In some plants, scum removal facilities are not provided in preaeration tanks. If the preaeration tanks are not preceded by grit removal facilities, grit deposition may occur in preaeration tanks.
Primary sedimentation	Primary and scum	Quantities of sludge and scum depend upon the nature of the collection system and whether industrial wastes are discharged to the system.
Biological treatment	Secondary and scum	Suspended solids are produced by the biological conversion of BOD. Some form of thickening may be required to concentrate the waste sludge stream from biological treatment.

^aThe coarse material removed by screening and grit during preliminary treatment are considered in Chap. 5.

^aThe coarse material removed by screening and grit during preliminary treatment are considered in Chap. 5.

settling tank is not a source of sludge. On the other hand, if wasting is accomplished from the activated sludge return line, the activated sludge settling tank constitutes a source of sludge. Processes used for thickening, digesting, conditioning, and dewatering of sludge produced from primary and secondary settling tanks also constitute sources.

Characteristics

To treat and reuse the sludge produced from wastewater treatment plants in the most effective manner, it is important to know the characteristics of the sludge that will be processed. The characteristics vary depending on the origin of the sludge, the amount of aging that has taken place, and the type of processing to which the sludge has been subjected (see Table 13-3).

General Composition. Typical data on the chemical composition of sludges are reported in Table 13-4. Many of the chemical constituents, including nutrients, are important in considering the ultimate disposition of the processed sludge and the liquid removed during processing. The measurement of pH, alkalinity, and organic acid content

Table 13-3

Characteristics of sludge and biosolids produced during wastewater treatment

Type	Description
Scum/grease	Scum consists of the floatable materials skimmed from the surface of primary and secondary settling tanks. Scum may contain grease, vegetable and mineral oils, animal fats, waxes, soaps, food wastes, vegetable and fruit skins, hair, paper and cotton, cigarette tips, plastic materials, condoms, grit particles, and similar materials. The specific gravity of scum is less than 1.0 and usually around 0.95.
Primary sludge	Sludge from primary settling tanks is usually gray and slimy and, in most cases, has an extremely offensive odor. Primary sludge can be readily digested under suitable conditions of operation.
Sludge from chemical precipitation	Sludge from chemical precipitation with metal salts is usually dark in color, though its surface may be red if it contains much iron. Lime sludge is grayish brown. The odor of chemical sludge may be objectionable, but is not as bad as primary sludge. While chemical sludge is somewhat slimy, the hydrate of iron or aluminum in it makes it gelatinous. If the sludge is left in the tank, it undergoes decomposition similar to primary sludge, but at a slower rate. Substantial quantities of gas may be given off and the sludge density increased by long residence times in storage.
Activated sludge	Activated sludge generally has a brown flocculant appearance. If the color is dark, the sludge may be approaching a septic condition. If the color is lighter than usual, there may have been underaeration with a tendency for the sludge to settle slowly. Sludge in good condition has an inoffensive "earthy" odor. The sludge tends to become septic rapidly and then has a disagreeable odor of putrefaction. Activated sludge digests well aerobically, but not anaerobically.
Trickling filter sludge	Humus sludge from trickling filters is brownish, flocculant, and relatively inoffensive when fresh. It generally undergoes decomposition more slowly than other undigested sludges. When trickling filter sludge contains many worms, it may become inoffensive quickly. Trickling filter sludge digests readily.
Aerobically digested biosolids	Aerobically digested biosolids are brown to dark brown and have a flocculant appearance. The odor of aerobically digested sludge is not offensive; it is often characterized as musty. Well digested aerobic sludge dewatered easily on drying beds.
Anaerobically digested biosolids	Anaerobically digested biosolids are dark brown to black and contain an exceptionally large quantity of gas. When thoroughly digested, they are not offensive, the odor being relatively faint and like that of hot tar, burnt rubber, or sealing wax.

^aThe characteristics of the coarse material removed by screening and grit during preliminary treatment are considered in Chap. 5.

Table 13-4**Typical chemical composition of untreated primary and activated sludge^a**

Item	Untreated primary sludge		Untreated activated sludge	
	Range	Typical	Range	Typical
Total dry solids (TS),%	1–6	3	0.4–1.2	0.8
Volatile solids (% of TS)	60–85	75	60–85	70
Grease and fats (% of TS)	5–8	6	5–12	8
Protein (% of TS)	20–30	25	32–41	36
Nitrogen (N, % of TS)	1.5–4	2.5	2.4–5	3.8
Phosphorus (P_2O_5 , % of TS)	0.8–2.8	1.6	2.8–11	5.5
Potash (K_2O , % of TS)	0–1	0.4	0.5–0.7	0.6
Cellulose (% of TS)	8–15	10	—	—
Iron (not as sulfide)	2–4	2.5	—	—
Silica (SiO_2 , % of TS)	15–20	—	—	—
pH	5–8	6	6.5–8	7.1
Alkalinity (mg/L as $CaCO_3$)	500–1500	600	580–1100	790
Organic acids (mg/L as HAc)	200–2000	500	1100–1700	1350
Energy content, kJ/kg VSS	23,000–29,000	25,000	19,000–23,000	20,000

^a Adapted, in part, from U.S. EPA (1979).

Note: kJ/kg × 0.4303 = Btu/lb.

is important in process control of anaerobic digestion. The content of heavy metals, pesticides, and hydrocarbons has to be determined when incineration and land application methods are contemplated. The thermal content of sludge is important where a thermal reduction process such as incineration or gasification is considered.

Specific Constituents. Biosolids characteristics that affect their suitability for application to land and for beneficial use include organic content (usually measured as volatile solids), nutrients, pathogens, metals, and toxic organics. The fertilizer value of the biosolids, which should be evaluated where they are to be used as a soil conditioner, is based primarily on the content of nitrogen, phosphorus, and potassium (potash). Typical nutrient values of wastewater biosolids as compared to commercial fertilizers are reported in Table 13-5. In most land application systems, biosolids provide sufficient nutrients for

Table 13-5**Comparison of nutrient levels in commercial fertilizers and wastewater biosolids**

Product	Nutrients, %		
	Nitrogen	Phosphorus	Potassium
Fertilizers for typical agricultural use ^a	5	10	10
Typical values for stabilized wastewater biosolids (based on TS) ^b	3.3	2.3	0.3

^a The concentrations of nutrients may vary widely depending upon the soil and crop needs.

^b The concentrations of nutrients may vary depending on wastewater nutrient removal requirements.

Table 13–6
Typical metal content in wastewater solids^a

Metal	Range of dry solids, mg/kg ^b
Arsenic ^c	1.18–49.2
Cadmium ^c	0.21–11.8
Chromium ^c	6.74–1160
Cobalt	0.87–290
Copper ^c	115–2580
Iron	1575–299,000
Lead ^c	5.81–450
Manganese	34.8–14,900
Mercury ^c	0.17–8.3
Molybdenum ^c	2.51–132
Nickel	7.44–526
Selenium ^c	1.1–24.7
Tin	7.5–522
Zinc ^c	216–8550

^a US EPA (2009).

^b Because of the wide range of values there is no typical value.

^c Metals currently regulated under 40 CFR 503.

plant growth. In some applications, the phosphorus and potassium content may be low and require augmentation.

Trace elements are those inorganic chemical elements that, in very small quantities, can be essential or detrimental to plants and animals. The term “heavy metals” is used to denote several of the trace elements present in sludge and biosolids. Concentrations of heavy metals may vary widely, as indicated in Table 13–6. Due to successful implementation of pretreatment programs, quality of sludge and biosolids in terms regulated heavy metals improved significantly. For the application of biosolids to land, concentrations of heavy metals may limit the application rate and the useful life of the application site (see Sec. 14–10).

Quantities

Data on the quantities of sludge produced from various processes and operations are presented in Table 13–7. Although the data in Table 13–7 are useful as presented, it should be noted that the quantity of sludge produced would vary widely. Corresponding data on expected sludge concentrations from various processes are given in Table 13–8.

Quantity Variations. The quantity of semi-solid and solid material entering the wastewater treatment plant daily may be expected to fluctuate over a wide range. To ensure capacity capable of handling these variations, the following items must be considered in the design of sludge processing and disposition facilities (1) the average and maximum rates of sludge production, and (2) the potential storage capacity of the treatment units within the plant. The variation in daily quantity of sludge that may be expected in large cities is shown

Table 13-7

Typical data for the physical characteristics and quantities of sludge produced from various wastewater treatment operations and processes

Treatment operation or process	Specific gravity of solids	Specific Gravity of Sludge	Dry solids lb/10 ³ gal		Dry solids kg/10 ³ m ³	
			Range	Typical	Range	Typical
Primary sedimentation	1.4	1.02	0.9–1.4	1.25	110–170	150
Activated sludge	1.25	1.05	0.6–0.8	0.7	70–100	80
Trickling filter	1.45	1.025	0.5–0.8	0.6	60–100	70
Extended aeration	1.3	1.015	0.7–1.0	0.8 ^a	80–120	100 ^a
Aerated lagoon	1.3	1.01	0.7–1.0	0.8 ^a	80–120	100 ^a
Filtration	1.2	1.005	0.1–0.2	0.15	12–24	20
Algae removal	1.2	1.005	0.1–0.2	0.15	12–24	20
Chemical addition to primary tanks for phosphorous removal						
Low lime (350–500 mg/L)	1.9	1.04	2.0–3.3	2.5 ^b	240–400	300 ^b
High lime (800–1600 mg/L)	2.2	1.05	5.0–11.0	6.6 ^b	600–1300	800 ^b
Suspended growth nitrification	—	—	—	—	—	— ^c
Suspended growth denitrification	1.2	1.005	0.1–0.25	0.15	12–30	18
Roughing filters	1.28	1.02	—	— ^d	—	— ^d

^a Assuming no primary treatment.

^b Solids in addition to that normally removed by primary sedimentation.

^c Negligible.

^d Included in biosolids production from secondary treatment processes.

on Fig. 13–1. The curve is characteristic of large cities having a number of large collection lines laid on flat slopes; even greater variations may be expected at small plants.

A limited quantity of sludge may be stored temporarily in the sedimentation and aeration tanks. Where digestion tanks with varying levels are used, their large storage capacity provides a substantial dampening effect on peak digested sludge loads. In sludge treatment systems where digestion is used, the design is usually based on maximum monthly loadings to provide a minimum of 15 d residence time during these maximum month loadings. Thus, based on average daily loadings, digesters have some sludge storage capacity. Where digestion is not used, the sludge treatment processes should be designed based on the inherent storage capacity available in the sludge handling system. For example, the mechanical dewatering system following gravity thickening could be based on the maximum 1 or 3 d sludge production. Certain components of the sludge processing system, such as sludge pumping and thickening, are sized to handle the maximum-day conditions.

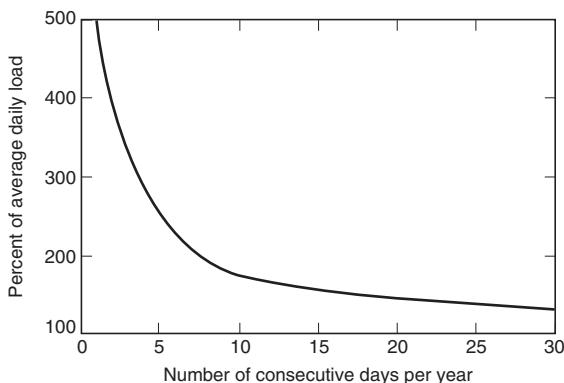
Volume-Mass Relationships. The volume of sludge depends mainly on its water content and only slightly on the character of the solid matter. A 10 percent sludge, for example, contains 90 percent water by weight. If the solid matter is composed of fixed

Table 13-8**Expected solids concentrations from various treatment operations and processes**

Operation or process application	Solids concentration, % dry solids	
	Range	Typical
Primary settling tank		
Primary sludge	1–6	3
Primary sludge to a cyclone degritter	0.5–3	1.5
Primary sludge and waste activated sludge	1–4	2
Primary sludge and trickling filter humus	4–10	5
Primary sludge with iron addition for phosphorus removal	0.5–3	2
Primary sludge with low lime addition for phosphorus removal	2–8	4
Primary sludge with high lime addition for phosphorus removal	4–16	10
Scum	3–10	5
Secondary settling tank		
Waste activated sludge with primary settling	0.5–1.5	0.8
Waste activated sludge without primary settling	0.8–2.5	1.3
High purity oxygen activated sludge with primary settling	1.3–3	2
High purity oxygen activated sludge without primary settling	1.4–4	2.5
Trickling filter humus	1–3	1.5
Rotating biological contactor waste sludge	1–3	1.5
Gravity thickener		
Primary sludge only	3–10	5
Primary sludge and waste activated sludge	2–6	3.5
Primary sludge and trickling filter humus	3–9	5
Dissolved air flotation thickener		
Waste activated sludge with polymer addition	4–6	5
Waste activated sludge without polymer addition	3–5	4
Centrifuge thickener (waste activated sludge only)	4–8	5
Gravity belt thickener (waste activated sludge with polymer addition)	3–6	5
Anaerobic digester		
Primary sludge	2–5	4
Primary sludge and waste activated sludge	1.5–4	2.5
Primary sludge and trickling filter humus	2–4	3
Aerobic digester		
Primary sludge only	2.5–7	3.5
Primary sludge and waste activated sludge	1.5–4	2.5
Waste activated sludge only	0.8–2.5	1.3

Figure 13-1

Peak sludge load as a function of the average daily load.



(mineral) and volatile (organic) solids, the specific gravity of all of the solid matter can be computed using Eq. (13-1).

$$\frac{W_s}{S_s \rho_w} = \frac{W_f}{S_f \rho_w} + \frac{W_v}{S_v \rho_w} \quad (13-1)$$

where W_s = weight of solids

S_s = specific gravity of solids

ρ_w = density of water

W_f = weight of fixed solids (mineral matter)

S_f = specific gravity of fixed solids

W_v = weight of volatile solids

S_v = specific gravity of volatile solids

Therefore, if one-third of the solid matter in a sludge containing 90 percent water is composed of fixed mineral solids with a specific gravity of 2.5, and two-thirds is composed of volatile solids with a specific gravity of 1.0, then the specific gravity of all solids S_s would be equal to 1.25, as follows:

$$\frac{1}{S_s} = \frac{0.33}{2.5} + \frac{0.67}{1.0} = 0.82$$

$$S_s = \frac{1}{0.82} = 1.25$$

If the specific gravity of the water is taken to be 1.0, the specific gravity of the sludge S_{sl} is 1.02, as follows:

$$\frac{1}{S_{sl}} = \frac{0.1}{1.25} + \frac{0.9}{1.0} = 0.98$$

$$S_{sl} = \frac{1}{0.98} = 1.02$$

The volume of sludge may be computed with the following expression:

$$V = \frac{M_s}{\rho_w S_s P_s} \quad (13-2)$$

where V = volume, m^3

M_s = mass of dry solids, kg

ρ_w = specific weight of water, 10^3 kg/m 3

S_{sl} = specific gravity of the sludge

P_s = percent solids expressed as a decimal

For approximate calculations for a given solids content, it is simple to remember that the volume varies inversely with the percent of solid matter contained in the sludge as given by

$$\frac{V_1}{V_2} = \frac{P_2}{P_1} \quad (\text{approximate})$$

Where V_1, V_2 = sludge volumes

P_1, P_2 = percent of solid matter

The application of these volume and weight relationships is illustrated in Example 13–1.

EXAMPLE 13–1 Volume of Untreated and Digested Dewatered Sludge Determine the liquid volume before and after digestion and dewatering and the percent reduction for 500 kg (dry basis) of primary sludge with the following characteristics:

	Primary	Digested and dewatered
Solids, %	5	20
Volatile matter, %	80	60 (destroyed)
Specific gravity of fixed solids	2.5	2.5
Specific gravity of volatile solids	≈1.0	≈1.0

Solution

1. Compute the average specific gravity of all the solids in the primary sludge using Eq. (13–1).

$$\frac{1}{S_s} = \frac{0.2}{2.5} + \frac{0.8}{1.0} = 0.88$$

$$S_s = \frac{1}{0.88} = 1.14 \quad (\text{primary solids})$$

2. Compute the specific gravity of the primary sludge.

$$\frac{1}{S_{sl}} = \frac{0.05}{1.14} + \frac{0.95}{1} = 0.99$$

$$S_{sl} = \frac{1}{0.99} = 1.01$$

3. Compute the volume of the primary sludge using Eq. (13–2).

$$V = \frac{500 \text{ kg}}{(10^3 \text{ kg/m}^3)(1.01)(0.05)} \\ = 9.9 \text{ m}^3$$

4. Compute the percentage of volatile matter after digestion total volatile solids after digestion.

$$\begin{aligned} \text{Volatile matter, \%} &= \frac{\text{total VS after digestion}}{\text{total TS after digestion}} \times 100 \\ &= \frac{(\text{VS Primary})M_s(1 - \text{VSR})}{M_s - M_s(\text{VS Primary})(\text{VSR})} \times 100 \\ &= \frac{(0.8)(500 \text{ kg})(1 - 0.6)}{500 \text{ kg} - 500 \text{ kg}(0.8)(0.6)} \times 100 = 61.5\% \end{aligned}$$

5. Compute the average specific gravity of all the solids in the digested sludge using Eq. (13–1).

$$\frac{1}{S_s} = \frac{0.385}{2.5} + \frac{0.615}{1.0} = 0.769$$

$$S_s = \frac{1}{0.769} = 1.30 \text{ (digested solids)}$$

6. Compute the specific gravity of the digested sludge (S_{ds}).

$$\frac{1}{S_{ds}} = \frac{0.20}{1.3} + \frac{0.80}{1} = 0.95$$

$$S_{ds} = \frac{1}{0.95} = 1.05$$

7. Compute the volume of digested sludge using Eq. (13–2).

$$V = \frac{500 \text{ kg} - 500\text{kg}(0.8)(0.6)}{(10^3 \text{ kg/m}^3)(1.05)(0.20)} \\ = 1.2 \text{ m}^3$$

8. Determine the percentage reduction in the sludge volume after digestion.

$$\text{Reduction} = \frac{(9.9 - 1.2) \text{ m}^3}{9.9 \text{ m}^3} \times 100 = 87.8\%$$

13-2 REGULATIONS FOR THE REUSE AND DISPOSITION OF SLUDGE IN THE UNITED STATES

In selecting the appropriate methods of sludge processing, reuse, and disposition, consideration must be given to the appropriate regulations. In the United States, regulations (40 CFR Part 503) were promulgated in 1993 by the U.S. Environmental Protection Agency (U.S. EPA) that established pollutant numerical limits and management practices for the reuse and disposition of sludge generated from the processing of municipal wastewater and septage (Federal Register, 1993). The regulations were designed to protect public health and the environment from any reasonably anticipated adverse effects of pollutants contained in the biosolids.

The regulations addressed by 40 CFR Part 503 cover specifically (1) land application of biosolids, (2) surface disposition of biosolids, (3) pathogen and vector reduction in treated biosolids, and (4) incineration. Each of these subjects is discussed below. The regulations directly affect selection of many of the processes used for sludge treatment, especially for sludge stabilization, i.e., alkaline stabilization, anaerobic digestion, aerobic digestion, and composting. In some cases, to achieve compliance, appropriate treatment requirements or methods are stipulated by the regulations. Additional discussion regarding regulations for applying biosolids on land is provided in Sec. 14–8.

Land Application

Land application relates to biosolids reuse and includes all forms of applying bulk or bagged biosolids to land for beneficial uses at agronomic rates, i.e., rates designed to

Emission Guidelines and New Source Performance Standards. The new rule requires facilities to meet the maximum achievable control technology (MACT) limits. The MACT standards for existing units were based on the best performing 12 percent of the existing units, while MACT standards for new or “modified” units are based on the “best controlled similar unit.” MACT standards have been set for nine pollutants. These pollutants are: cadmium (Cd), lead (Pb), mercury (Hg), particulate matter (PM), carbon monoxide (CO), hydrogen chloride (HCl), sulfur dioxide (SO₂), nitrogen oxides (NO_x), and dioxins and furans (PCDD/PCDF).

The new SSI rules and emission guidelines contain standards for existing and new multiple-hearth furnaces (MHFs) and fluidized bed incinerators (FBIs). This rule stipulates that all SSIs will require Title V operating permits, annual operator training, annual stack testing and/or continuous emissions monitoring systems, recordkeeping requirements, and establishment of operating limits. For new or modified SSIs, the proposed MACT limits are set as a composite of the best emissions performance from the best SSIs tested. For new SSIs, owners or operators are required to conduct a siting analysis prior to construction. This analysis would include site specific analysis of air pollution control alternatives to minimize the environmental and health impacts to the maximum extent practicable. Details regarding air pollution controls are discussed in Sec. 14–6 in Chap. 14.

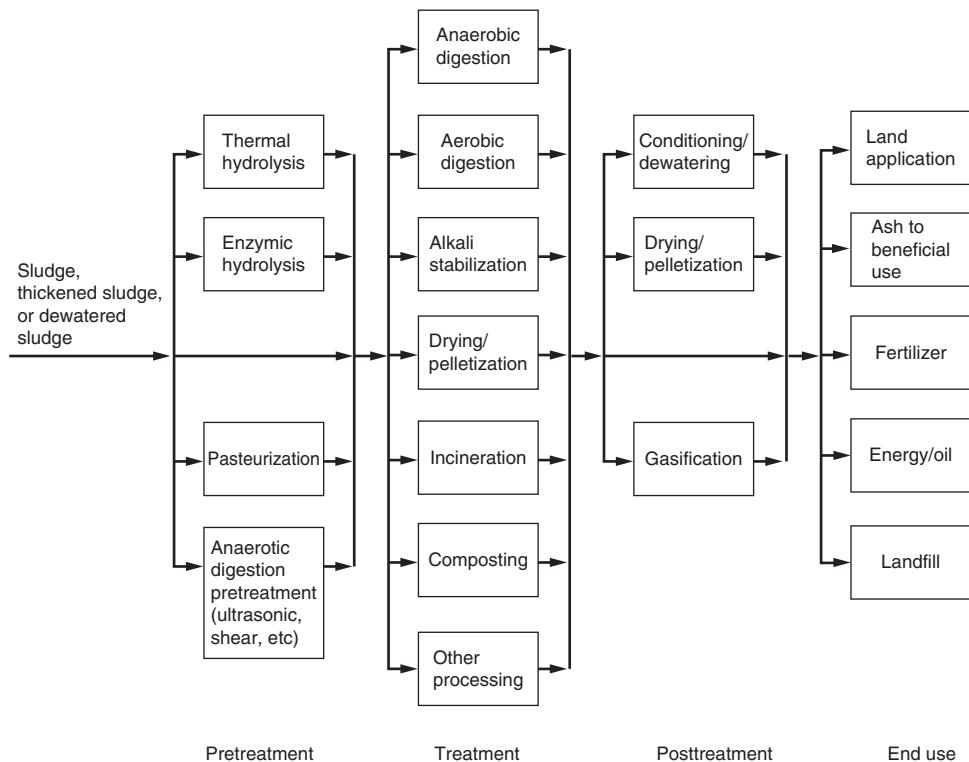
Clean Water Act. Under the 503 regulations, the requirements for biosolids incineration are still applicable. There is some overlap between the 503 regulations and the Clean Air Act Section 129 MACT based EGs and NSPSs, but they differ significantly due to the different approaches used to develop each set of rules. The 503 regulations are based on a risk based approach and aim to avoid adverse impacts. Limits are based partially on maximum allowable concentrations of pollutants within the feed biosolids coupled with stack monitoring of total hydrocarbons (or carbon monoxide) and operational standards to ensure good combustion and emissions performance. However, the MACT rules are technology based and set the limits relative to the best performing incinerator units within their class (ie. MHF or FBI). These limits are expressed as maximum concentrations of pollutants in the incinerator flue gases. The methods of measurement and media in which the concentrations are measured are entirely different and are not directly comparable to the 503 regulations. In practical application, the MACT emission requirements are much more stringent than previously required under Part 503 and they will generally dictate the required emission performance levels for both new and existing incinerators. For the time being, both sets of rules apply with overlapping requirements leading to duplicate sampling by operators of both the feed biosolids and the flue gases.

13–3 SLUDGE PROCESSING FLOW DIAGRAMS

A generalized flow diagram incorporating the unit operations and processes to be discussed in this chapter and chapter 14 is presented on Fig. 13–2. As shown, an almost infinite number of combinations are possible. In practice, the most commonly used process flow diagram for sludge processing involves biological treatment. Typical flow diagrams incorporating biological processing are presented on Fig. 13–3. Thickeners may be used depending upon the source of sludge and the method of sludge stabilization, dewatering, and disposition. Following biological digestion, any of the several methods shown may be used to dewater the sludge; the choice depends on economic evaluation, beneficial use requirements, and local conditions. In instances where biological stabilization is not used, dewatered sludge undergoes thermal decomposition, in either multiple-hearth or fluidized-bed incinerators. Furthermore, unstabilized dewatered cake can be dried, alkaline stabilized, or hauled to a landfill.

Figure 13-2

Generalized sludge processing flow diagram.



13-4 SLUDGE AND SCUM PUMPING

Sludge produced in wastewater treatment plants must be conveyed from point to point in the plant in conditions ranging from a watery sludge or scum to a thick sludge. Sludge may also be pumped off-site for long distances for treatment and disposition. For each type of sludge and pumping application, a different type of pump may be needed (see Table 13-13).

Pumps

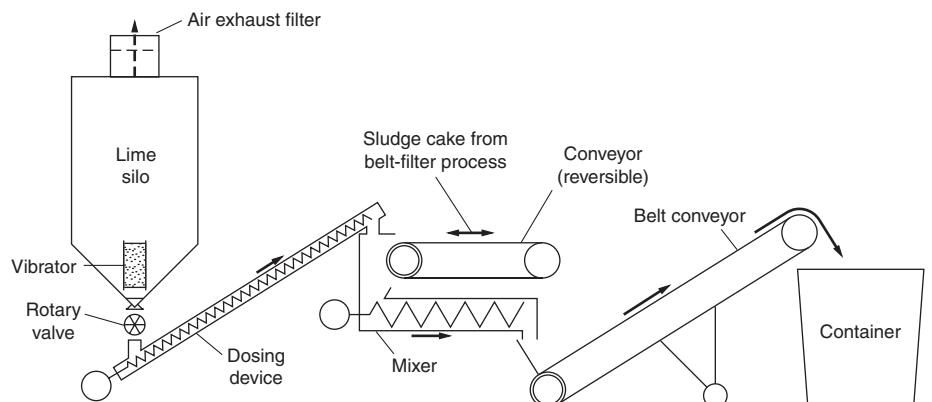
Pumps used most frequently to convey sludge include the plunger, progressive cavity, hose, solids handling centrifugal (screw centrifugals and traditional “non-clog designs), recessed impeller, diaphragm, high-pressure piston diaphragm, and rotary lobe types. Other types of pumps such as hydraulic piston slurry pumps have also been used to pump sludge. Chopper pumps are used extensively for pumping scum containing rags, plastics, and other fibrous materials that require shredding. The advantages and disadvantages of each type of pump are summarized in Table 13-14.

Plunger Pumps. Plunger pumps [see Fig. 13-4(a)] have been used frequently for sludge applications, especially primary sludges, and have proved to be quite satisfactory. The advantages of plunger pumps are as follows:

1. Pulsating action of simplex and also duplex pumps tends to concentrate the sludge in the hoppers ahead of the pumps and resuspend solids in pipelines when pumping at low velocities.
2. They are suitable for suction lifts up to 3 m (10 ft) and are self-priming.

Figure 13-17

Typical lime posttreatment system. (From Roediger Pittsburgh.)



of other chemicals, a higher chemical dose, and supplemental drying. These processes alter the characteristics of the feed material and, depending on the process, may increase product stability, decrease odor potential, and provide product enhancement. To utilize these technologies, dewatered sludge is required.

Pasteurization may be accomplished by the exothermic reaction of quicklime with water to achieve a process temperature of 70°C and maintain it for more than 30 min. Other sources of energy can be used to aid in increasing the temperature generated from the exothermic chemical reaction. For example, the pasteurization system marketed by RDP Company uses electricity to generate heat for raising the temperature to the required degree. N-Viro International Corporation markets an advanced alkaline stabilization system combined with drying. To meet Class A biosolids criteria, the pasteurization reaction must be carried out under carefully controlled and monitored mixing and temperature conditions to ensure uniform treatment and inactivation of pathogens by the heat generated during the reaction. The process produces a soil-like material that is not subject to liquefaction under mechanical stress. Several other process variations of advanced alkaline stabilization are available, some of which are proprietary. Additional information may be found in WEF (2010a) and WEF (2012).

13-9 ANAEROBIC DIGESTION

Anaerobic digestion is among the oldest processes used for the stabilization of sludge. As described in Chap. 10, anaerobic digestion involves the decomposition of organic matter and reduction of inorganic matter (principally sulfate) in the absence of molecular oxygen. The major applications of anaerobic digestion are in the stabilization of concentrated sludges produced from the treatment of municipal and industrial wastewater. Great progress has been made in the fundamental understanding and control of the process, the sizing of tanks, and the design and application of equipment. Because of the emphasis on energy conservation and recovery and the desire to obtain beneficial use of wastewater biosolids, anaerobic digestion continues to be the dominant process for stabilizing sludge. Furthermore, anaerobic digestion of municipal wastewater sludge can, in many cases, produce sufficient digester gas to meet most of the energy needs for plant operation. An aerial view of a large digester installation is shown on Fig. 13-18.

In this section, a brief review is provided of process fundamentals followed by discussions of mesophilic anaerobic digestion, the most common basic process used; thermophilic digestion; and phased digestion. Phased digestion covers many of the new developments in anaerobic digestion.

Figure 13-18

Aerial view of several large anaerobic digesters at Boston, MA.



Process Fundamentals

As described in Chap. 7, the three types of chemical and biochemical reactions that occur in anaerobic digestion are hydrolysis; fermentation, also called acidogenesis (the formation of soluble organic compounds and short-chain organic acids); and methanogenesis (the bacterial conversion of organic acids into methane and carbon dioxide). Important environmental factors in the anaerobic digestion process are (1) solids retention time, (2) hydraulic retention time, (3) temperature, (4) alkalinity, (5) pH, (6) the presence of inhibitory substances, i.e., toxic materials, and (7) the bioavailability of nutrients and trace metals. The first three factors are important in process selection and are discussed in this section. Alkalinity is a function of feed solids and is important in controlling the digestion process. The effects of pH and inhibitory substances are discussed in Chaps. 7 and 10. The presence of nutrients and trace metals necessary for biological growth is described in Sec. 10-2 in Chap. 10.

Solids and Hydraulic Retention Times. Anaerobic digester sizing is based on providing sufficient residence time in well-mixed reactors to allow significant destruction of volatile suspended solids (VSS) to occur. Sizing criteria that have been used are (1) solids retention time SRT, the average time the solids are held in the digestion process, and (2) the hydraulic retention time τ , the average time the liquid is held in the digestion process. For soluble substrates, the SRT can be determined by dividing the mass of solids in the reactor (M) by the mass of solids removed daily (M/d). The hydraulic retention time τ is equal to the volume of liquid in the reactor (m^3) divided by the quantity of biosolids removed (m^3/d). For digestion systems without recycle, $SRT = \tau$.

The three reactions (hydrolysis, fermentation, and methanogenesis) are directly related to SRT (or τ). An increase or decrease in SRT results in an increase or decrease in the extent of each reaction. There is a minimum SRT for each reaction. If the SRT is less than the minimum SRT, bacteria cannot grow rapidly enough and the digestion process will fail eventually (WEF, 2010a).

Temperature. As discussed in Sec. 7-5, temperature not only influences the metabolic activities of the microbial population but also has a profound effect on such factors as gas transfer rates and the settling characteristics of biological sludges. In anaerobic digestion, temperature is important in determining the rate of digestion, particularly the rates of hydrolysis and methane formation. The minimum SRT required to achieve a given

amount of VSS destruction is based on the design operating temperature. Most anaerobic digestion systems are designed to operate in the mesophilic temperature range, between 30 and 38°C (85 and 100°F). Other systems are designed for operation in the thermophilic temperature range of 50 to 57°C (122 to 135°F). Newly developed systems, as discussed in a latter part of this section, use a combination of mesophilic and thermophilic digestion in separate stages.

While selection of the design operating temperatures is important, maintaining a stable operating temperature is more important because the bacteria, especially the methane formers, are sensitive to temperature changes. Generally, temperature changes greater than 1°C/d affect process performance, and thus changes less than 0.5°C/d are recommended (WEF, 2010a).

Alkalinity. Calcium, magnesium, and ammonium bicarbonates are examples of buffering substances found in a digester. The digestion process produces ammonium bicarbonate from the breakdown of protein in the raw sludge feed; the others are found in the feed sludge. The concentration of alkalinity in a digester is, to a great extent, proportional to the solids feed concentration. A well-established digester has a total alkalinity of 2000 to 5000 mg/L.

The principal consumer of alkalinity in a digester is carbon dioxide, and not volatile fatty acids as is commonly believed (Speece, 2001). Carbon dioxide is produced in the fermentation and methanogenesis phases of the digestion process (see Sec. 7–12 in Chap. 7). Due to the partial pressure of gas in a digester, the carbon dioxide solubilizes and forms carbonic acid, which consumes alkalinity. The carbon dioxide concentration in the digester gas is, therefore, reflective of the alkalinity requirements. Volatile fatty acids are intermediate products from the acid phase of digestion and consume alkalinity. Volatile acids in digesters range from 50 to 300 mg/L. The ratio of volatile acids to the alkalinity is a parameter that is used to monitor the health of the digestion process and should be monitored closely. The volatile acids to alkalinity ratio for well-established digesters should fall between 0.05 to 0.25 with a 0.1 value indicating a good buffering capacity. Supplemental alkalinity can be supplied by the addition of sodium bicarbonate, lime, or sodium carbonate.

Description of Mesophilic Anaerobic Digestion Processes

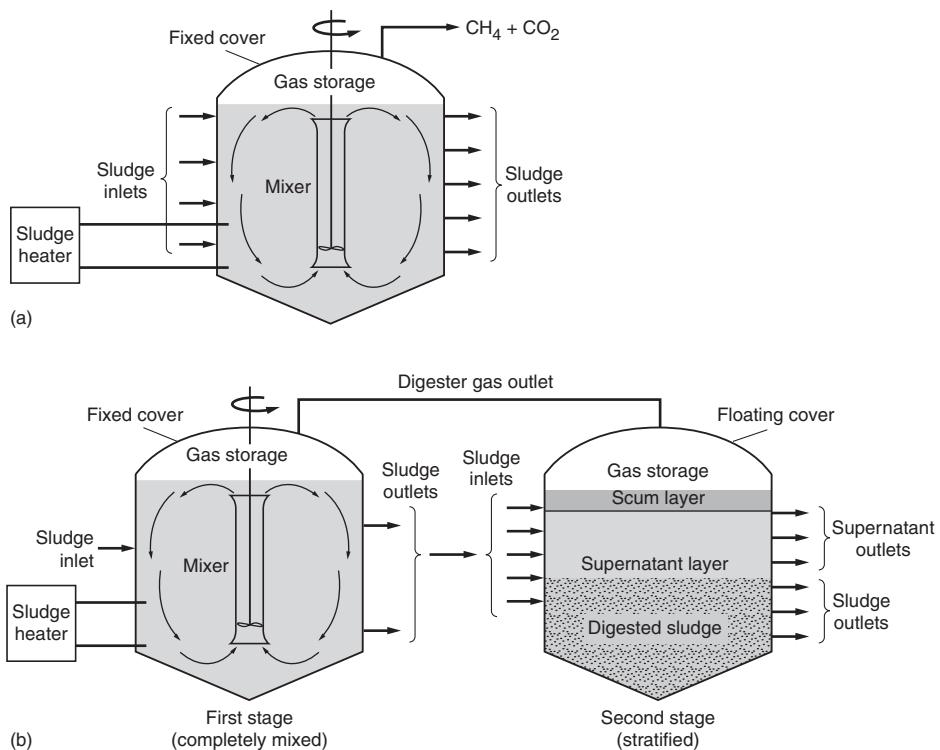
The operation and physical facilities for mesophilic anaerobic digestion in single-stage high-rate, two-stage, and separate digesters for primary sludge and waste activated sludge are described in this section. Standard-rate, sometimes called low-rate, digestion is seldom used for digester design (because of the large tank volume required and the lack of adequate mixing) and is not covered in this text. For information about standard-rate digestion, the reader is referred to the third edition of this text (Metcalf & Eddy, 1991) and WEF (1998). The processes described below normally operate in the mesophilic range; high-rate digesters also operate in the thermophilic range. Thermophilic digestion is discussed at the end of the section.

Single-Stage High-Rate Digestion. Heating, auxiliary mixing, uniform feeding, and thickening of the feed stream characterize the single-stage high-rate digestion process. The sludge is mixed by one of many systems such as gas recirculation, pumping, or draft-tube mixers (separation of scum and supernatant does not take place), and sludge is heated to achieve optimum digestion rates [see Fig. 13–19(a)].

Uniform feeding is very important, and sludge should be pumped to the digester continuously or on a 30-min to 2-h time cycle to help maintain constant conditions in

Figure 13-19

Schematic diagram of typical anaerobic digesters (a) high-rate and (b) two-stage.



the reactor. In digesters fed on a daily cycle of 8 or 24 h, it is important to withdraw digested sludge from the digester before adding the feed sludge, because the pathogen kill is significantly greater when compared to using the feed sludge to displace the waste sludge (Speece, 2001). Because there is no supernatant separation in the high-rate digester, and the total solids are reduced by 45 to 50 percent and given off as gas, the digested sludge is about half as concentrated as the untreated sludge feed. Digestion tanks may have fixed roofs or floating covers (see subsequent discussion of digester types). Any or all of the floating covers may be of the gas holder type, which provides excess gas storage capacity. Alternatively, gas may be stored in a separate low-pressure gas holder or compressed and stored under pressure.

Two-Stage Digestion. Two-stage digestion, which was frequently used in the past, is seldom used in modern digester design. In two-stage digestion, a high-rate digester is coupled in series with a second tank [see Fig. 13-19(b)]. The first tank is used for digestion and is heated and equipped with mixing facilities. The second tank is usually unheated and used principally for storage. The tanks may be identical, in which case either one may be the primary. Tanks may have fixed roofs or floating covers, the same as single-stage digestion. In other cases, the second tank may be an open tank or a sludge lagoon. In the case of an open second digester some methane would escape if digestion continued leading to increased carbon footprint of the processes. Two-stage digestion of the type described above is seldom used, mainly because of the expense of building a large tank that is not fully utilized and because the second tank was of negligible benefit, operationally.

Because anaerobically digested biosolids may not settle well, the supernatant withdrawn from the second-stage tank may contain high concentrations of suspended solids. Reasons for poor settling characteristics include incomplete digestion in the primary

digester (which generates gases in the secondary digester and causes floating solids) and fine-sized solids that have poor settling characteristics. Supernatant returned to the liquid processing system could cause upset conditions and might require separate treatment. Where two-stage digestion is used, return flows from the second tank must be accounted for in the solids mass balance. Less than 10 percent of the gas generated comes from the second stage.

In some installations, the second stage is a heated and mixed reactor to achieve further stabilization prior to dewatering or other subsequent processing. Additional discussion is provided later in this section on two-phase mesophilic digestion that provides more effective utilization of tank capacity.

Separate Sludge Digestion. Most wastewater treatment plants employing anaerobic digestion use a single digester for the digestion of a mixture of primary and biological sludge. The solid-liquid separation of digested primary sludge, however, is downgraded by even small additions of biological sludge, particularly activated sludge. The rate of reaction under anaerobic conditions is also slowed slightly. In separate sludge digestion, the digestion of primary and biological sludges is accomplished in separate tanks. Reasons cited for separate digestion include (1) the excellent dewatering characteristics of the digested primary sludge are maintained, (2) the digestion process is specifically tailored to the sludge being treated, and (3) optimum process control conditions can be maintained. Design criteria and performance data for the separate anaerobic digestion of biological sludges, however, are very limited. In some cases, especially where biological phosphorus removal is practiced, biological sludge is digested aerobically instead of anaerobically to prevent resolubilization of the phosphorus under anaerobic conditions. Separate sludge digestion is not currently a common practice at most plants.

Process Design for Mesophilic Anaerobic Digestion

Ideally, the design of anaerobic sludge digestion processes should be based on an understanding of the fundamental principles of biochemistry and microbiology discussed in Chap. 7 in Sec. 7–12. Because these principles have not been appreciated fully in the past, a number of empirical methods have also been used in the design of digesters. The purpose of the following discussion is to illustrate the various methods that have been used to design single-stage, high-rate digesters in terms of size. These methods are based on (1) solids retention time, (2) the use of volumetric loading factors, (3) volatile solids destruction, (4) observed volume reduction, and (5) loading factors based on population.

Solids Retention Time. Digester design based on SRT involves application of the principles discussed in Chaps. 7 and 10. To review briefly, the respiration and oxidation end products of anaerobic digestion are methane gas and carbon dioxide. The quantity of methane gas can be calculated using Eq. (13–12):

$$V_{\text{CH}_4} = (0.35)[(S_o - S)(Q)(1 \text{ kg}/10^3 \text{ g}) - 1.42P_x] \quad (13-12)$$

where V_{CH_4} = volume of methane produced at standard conditions (0°C and 1 atm), m^3/d

0.35 = theoretical conversion factor for the amount of methane produced, m^3 , from the conversion of 1 kg of bCOD at 0°C (conversion factor at $35^\circ\text{C} = 0.40$, see Example 7–10 in Chap. 7)

Q = flowrate, m^3/d

S_o = bCOD in influent, g/m^3

S = bCOD in effluent, g/m^3

P_x = net mass of cell tissue produced per day, kg/d

Table 13-27

Suggested solids retention times for use in the design of complete-mix anaerobic digesters^a

	Operating temperature, °C	SRT (minimum)	SRT _{des}
	18	11	28
	24	8	20
	30	6	14
	35	4	10
	40	4	10

^a From McCarty (1964) and (1968).

Note: 1.8 (°C) + 32 = °F.

The theoretical conversion factor for the amount of methane produced from the conversion of 1 g of bCOD is derived in Sec. 7–12 in Chap. 7. For a complete-mix high-rate digester without recycle, the mass of biological solids synthesized daily, P_x , can be estimated using Eq. (13–13).

$$P_x = \frac{YQ(S_o - S)(1 \text{ kg}/10^3 \text{ g})}{1 + b(\text{SRT})} \quad (13-13)$$

where Y = yield coefficient, g VSS/g bCOD

b = endogenous coefficient, d⁻¹ (typical values range from 0.02 to 0.04)

SRT = solids retention time, d

other terms as defined previously

For a complete-mix digester, the SRT is the same as the hydraulic retention time τ .

Typical anaerobic reaction values for Y and b are given in Table 10–13 in Chap. 10 and range from 0.05 to 0.10 and 0.01 to 0.04, respectively. Typical values for SRT at various temperatures are reported in Table 13–27. In practice for high-rate digestion, however, values for SRTs range from 15 to 20 d. Grady, Daigger, and Lim (1999) observed that (1) a lower SRT limit of 10 days at a temperature of 35°C is sufficient to ensure an adequate safety factor against a washout of the methanogenic population, and (2) incremental changes in volatile solids destruction are relatively small for SRT values above 15 d at 35°C. In selecting the design SRT for anaerobic digestion, peak hydraulic loading must be considered. The peak loading can be estimated by combining poor thickener performance with the maximum sustained plant loading expected during seven continuous days during the design period (U.S. EPA, 1979). The application of Eqs. (13–12) and (13–13) in the process design of a high-rate digester is illustrated in Example 13–5.

EXAMPLE 13-5

Estimating Single-stage, High-rate Digester Volume and Performance

Estimate the size of digester required for primary sludge from a primary clarifier designed for 38,000 m³/d (10 Mgal/d) of wastewater. Check the volumetric loading and the amount of gas produced. The influent wastewater BOD and TSS concentrations are 400 and 300 mg/L, respectively. The primary clarifier achieves 35 percent BOD removal and 50 percent TSS removal. Assume that the primary sludge contains about 95 percent moisture and has a specific gravity of 1.02. Other pertinent design assumptions are as follows:

1. The hydraulic regime of the reactor is complete-mix.
2. $\tau = \text{SRT} = 15 \text{ d}$ at 35°C (see Table 13–27).

3. Efficiency of waste utilization (solids conversion) $E = 0.70$.
4. The sludge contains adequate nitrogen and phosphorus for biological growth.
5. $Y = 0.08 \text{ kg VSS/kg bCOD utilized}$ and $b = 0.03 \text{ d}^{-1}$.
6. Constants are for a temperature of 35°C .
7. Digester gas is 65 percent methane.

Solution

1. Determine the daily sludge mass and volume using Eq. (13-2).

$$\text{Sludge mass} = \frac{(38,000 \text{ m}^3/\text{d})(300 \text{ g/m}^3)(0.5)}{(10^3 \text{ g}/1 \text{ kg})} = 5700 \text{ kg/d}$$

$$\text{Sludge volume} = \frac{(5700 \text{ kg/d})}{1.02(10^3 \text{ kg/m}^3)(0.05)} = 111.8 \text{ m}^3/\text{d}$$

2. Determine the bCOD loading.

$$\text{bCOD loading} = (0.35)(400 \text{ g/m}^3)(38,000 \text{ m}^3/\text{d})(1 \text{ kg}/10^3 \text{ g}) = 5320 \text{ kg/d}$$

3. Compute the digester volume.

$$\tau = \frac{V}{Q}$$

$$V = Q\tau = (111.8 \text{ m}^3/\text{d})(15\text{d}) = 1677 \text{ m}^3$$

4. Compute the volumetric loading.

$$\frac{(\text{kg bCOD/d})}{\text{m}^3} = \frac{(5320 \text{ kg/d})}{1677 \text{ m}^3} = 3.17 \text{ kg/m}^3 \cdot \text{d}$$

5. Compute the quantity of volatile solids produced per day using Eq. (13-13).

$$P_x = \frac{YQ(S_o - S)(10^3 \text{ g/kg})^{-1}}{1 + b(\text{SRT})}$$

$$S_o = 5320 \text{ kg/d}$$

$$S = 5320(1 - 0.70) = 1596 \text{ kg/d}$$

$$S_o - S = 5320 - 1596 = 3724 \text{ kg/d}$$

$$P_x = \frac{(0.08)[(5320 - 1596)\text{kg/d}]}{1 + (0.03\text{d}^{-1})(15\text{d})} = 205.5$$

6. Compute the volume of methane produced per day at 35°C using Eq. (13-12) (conversion factor at $35^\circ\text{C} = 0.40$).

$$V_{\text{CH}_4} = (0.40)[(S_o - S)(Q)(10^3 \text{ g/kg})^{-1} - 1.42 P_x]$$

$$\begin{aligned} V_{\text{CH}_4} &= (0.4 \text{ m}^3/\text{kg})[(5320 - 1596)\text{kg/d} - 1.42(205.5 \text{ kg/d})] \\ &= 1373 \text{ m}^3\text{d} \end{aligned}$$

7. Estimate the total gas production.

$$\text{Total gas volume} = \frac{1373}{0.65} = 2112 \text{ m}^3/\text{d}$$

Table 13-28

Typical design criteria for sizing mesophilic high-rate complete-mix anaerobic sludge digesters^a

Parameter	U.S. customary units		SI units	
	Units	Value	Units	Value
Volume criteria				
Primary sludge	ft ³ /capita	1.3–2.0	m ³ /capita	0.03–0.06
Primary sludge + trickling filter humus sludge	ft ³ /capita	2.6–3.3	m ³ /capita	0.07–0.09
Primary sludge + activated sludge	ft ³ /capita	2.6–4.0	m ³ /capita	0.07–0.11
Solids loading rate ^b	lb VSS/ 10 ³ ft ³ ·d	100–300	kg VSS/m ³ ·d	1.6–4.8
Solids retention time ^b	d	15–20	d	15–20

^a Adapted, in part, from U.S. EPA (1979).

^b Based on combined primary and secondary sludges digestion without any pretreatment methods.

Loading Factors. One of the most common methods used to size digesters is to determine the required volume based on a loading factor. Although a number of different factors have been proposed, the two most favored are based on (1) the mass of volatile solids added per day per unit volume of digester capacity and (2) the mass of volatile solids added to the digester each day per mass of volatile solids in the digester. Of the two, the first method is preferred. Loading criteria are based generally on sustained loading conditions (see Chap. 3), typically peak 2-wk or peak mo sludge production with provisions for avoiding excessive loadings during shorter periods. Typical design criteria for sizing mesophilic high-rate anaerobic digesters are given in Table 13-28. The upper limit of volatile solids loading rates is typically determined by the rate of accumulation of toxic materials, particularly ammonia, or washout of methane formers (WEF, 2010a).

Excessively low volatile solids loading rates can result in designs that are costly to build and are troublesome to operate. In a survey conducted by Speece (1988) of 30 digester installations in the United States, one of the most significant observations was the relatively low solids content in the sludge feed to the digesters. The average TSS in the sludge feed was 4.7 ± 1.6 percent and the average volatile solids content was 70 percent. The average VSS value in the digesters was a dilute 1.6 percent. Dilute sludge feed causes low volatile solids loading leading to starving conditions within the digester, resulting in the following adverse effects in digester operation: (1) reduced τ , (2) reduced VS destruction, (3) reduced methane generation, (4) reduced alkalinity, (5) increased volumes of digested biosolids and supernatant, (6) increased heating requirements, (7) increased dewatering capacity, and (8) increased hauling cost for liquid biosolids. As a cautionary note, a potential problem with ammonia toxicity could occur if the waste activated sludge is thickened too much. Thus, in planning the design and operation of anaerobic digesters, consideration should be given to optimizing volatile solids loading to effectively utilize digester capacity. The effect of solids concentration and hydraulic detention time on volatile solids loading is reported in Table 13-29.

Estimating Volatile Solids Destruction. The degree of stabilization obtained is often measured by the percent reduction in volatile solids. The reduction in volatile solids can be related either to the SRT or to the detention time based on the untreated sludge feed.

Table 13-29**Effect of sludge concentration and hydraulic detention time on volatile solids loading factors^a**

Sludge concentration, %	Volatile solids loading factor							
	lb/ft ³ ·d				kg/m ³ ·d			
	10 d ^b	12 d	15 d	20 d	10 d	12 d	15 d	20 d
2	0.09	0.07	0.06	0.04	1.4	1.2	0.95	0.70
3	0.13	0.11	0.09	0.07	2.1	1.8	1.4	1.1
4	0.18	0.15	0.12	0.09	2.9	2.4	1.9	1.4
5	0.22	0.19	0.15	0.11	3.6	3.0	2.4	1.8
6	0.27	0.22	0.18	0.13	4.3	3.6	2.9	2.1
7	0.31	0.26	0.21	0.16	5.0	4.2	3.3	2.5
8	0.36	0.30	0.24	0.18	5.7	4.8	3.8	2.9

^aBased on 70 percent volatile content of sludge, and a sludge specific gravity of 1.02 (concentration effects neglected).

^bHydraulic detention time, d.

The amount of volatile solids destroyed in a high-rate complete-mix digester can be roughly estimated by the following empirical equation (Liptak, 1974):

$$V_d = 13.7 \ln(SRT_{des}) + 18.9 \quad (13-14)$$

where V_d = volatile solids destruction, %

SRT_{des} = time of digestion, d (range 15 to 20 d)

The equation does not account for variation in the sludge feed to digestion and the digestion mixing and other operating conditions and should be used to obtain a rough estimate only, and it appears that the equation overestimates volatile solids destruction. Typical volatile solids destruction ranges as a function of SRT are provided in Table 13-30. Because the untreated sludge feed can be measured easily, this method is also used commonly. In plant operation, calculation of volatile solids reduction should be made routinely as a matter of record whenever sludge is drawn to processing equipment or drying beds. Alkalinity and volatile acids content should also be checked daily as a measure of the stability of the digestion process.

In calculating the volatile solids reduction, the ash content of the sludge is assumed to be conservative; that is, the number of pounds of ash going into the digester is equal to that being removed. Digester VSR can be calculated based on two different methods. The first method is the mass balance method, which is shown below.

$$R_{VSS} = \frac{M_{VS \text{ in feed}} - M_{VS \text{ in digested sludge}} - M_{VS \text{ in supernatant}}}{M_{VS \text{ in feed}}} \times 100 \quad (13-15)$$

Table 13-30**Estimated volatile solids destruction in high-rate complete-mix mesophilic anaerobic digestion**

Digestion time, d	Volatile solids destruction, %
30	50–65
20	50–60
15	45–50

where R_{VSS} = volatile solids destruction, %
 $M_{VS \text{ in feed}}$ = mass flowrate of volatiles in digester feed, kg/d
 $M_{VS \text{ in digested sludge}}$ = mass flowrate of volatiles out of digester, kg/d
 $M_{VS \text{ in supernatant}}$ = mass flowrate of volatiles in digester decant stream, kg/d

It should be noted that in modern high rate digesters, there is no decant so the $M_{VS \text{ in supernatant}}$ number goes to zero. Digester VSR can also be calculated with the simplified Van Kleeck formula given below.

$$R_{VSS} = \frac{W_{VS \text{ in feed}} - W_{VS \text{ in digested sludge}}}{W_{VS \text{ in feed}} - (W_{VS \text{ in digested sludge}})(W_{VS \text{ in feed}})} \times 100 \quad (13-16)$$

where $W_{VS \text{ in feed}}$ = Weight fraction of digested sludge volatile content per total dry solids
 $W_{VS \text{ in digested sludge}}$ = Weight fraction of volatiles out of digester per total dry solids

It should be noted that the Van Kleeck formula assumes that there is no supernatant withdrawal or accumulation of grit inside the digester so in practice the results may not be 100 percent accurate. A typical example calculation of volatile solids reduction is presented in Example 13-6.

EXAMPLE 13-6 Determination of Volatile Solids Reduction From the following analysis of untreated and digested biosolids, determine the total volatile solids reduction achieved during digestion. It is assumed that (1) the weight of fixed solids in the digested biosolids equals the weight of fixed solids in the untreated sludge and (2) the volatile solids are the only constituents of the untreated sludge lost during digestion.

	Volatile solids, %	Fixed solids, %
Untreated sludge	68	32
Digested sludge	50	50

Solution

1. Determine the weight of the digested solids. Because the quantity of fixed solids remains the same, the weight of the digested solids based on 1.0 kg of dry untreated sludge, as computed below, is 0.64 kg.

$$\text{Fixed solids in untreated sludge} = \frac{0.32 \text{ kg}}{(0.32 + 0.68) \text{ kg}} 100 = 32\%$$

Let X equal the weight of volatile solids after digestion. Then

$$\text{Fixed solids after digestion} = \frac{0.32 \text{ kg}}{(0.32 + X) \text{ kg}} 100 = 50\%$$

$$\text{Weight of volatile solids after digestion, } X \text{ kg} = \frac{0.32 \text{ kg}}{0.5} - 0.32 = 0.32 \text{ kg}$$

$$\text{Weight of digested solids} = 0.32 \text{ kg} + 0.32 \text{ kg} = 0.64 \text{ kg}$$

2. Determine the percent reduction in total and volatile suspended solids.
 a. Percent reduction of total suspended solids

$$R_{TSS} = \frac{(1.0 - 0.64) \text{ kg}}{1.0 \text{ kg}} 100 = 36\%$$

- b. Percent reduction in volatile suspended solids using both methods

Using the mass balance method [Eq. (13–15)]

$$R_{VSS} = \frac{(0.68 - 0.32) \text{ kg}}{0.68 \text{ kg}} 100 = 52.9\%$$

Using the Van Kleeck method [Eq. (13–16)]

$$R_{VSS} = \frac{0.68 - 0.5}{0.68 - 0.5(0.68)} 100 = 52.9\%$$

Population Basis. Digestion tanks are also designed on a volumetric basis by allowing a certain number of cubic meters per capita (cubic feet per capita). Detention times range from 10 to 20 d for high-rate digesters (U.S. EPA, 1979). These detention times are recommended for design based on total tank volume, plus additional storage volume if sludge is dried on beds and weekly sludge withdrawals are curtailed because of inclement weather.

Typical design criteria for heated anaerobic digesters based on population are shown in Table 13–28. The criteria are applied only where analyses and volumes of sludge to be digested are not available. The capacities shown in Table 13–28 should be increased 60 percent in a municipality where the use of food-waste grinders is universal and should be increased on a population-equivalent basis to allow for the effect of industrial wastes.

Selection of Tank Design and Mixing System

Most anaerobic digestion tanks are either cylindrical, conventional German design, or egg-shaped (see Fig. 13–20). The most common shape used in the United States is a shallow, vertical cylinder with a floating cover [see Fig. 13–20(a)] or fixed cover [see Fig. 13–20(b)]. Rectangular tanks were used in the past, but they experienced great difficulty in mixing the tank contents uniformly. German designers have worked on optimizing the shape of digesters, and two basic types have emerged: the conventional German digester and the egg-shaped digester. The conventional German digester [see Fig. 13–20(c)] is a deep cylindrical vessel with steeply sloped top and bottom cones (Stukenberg et al., 1992). The egg-shaped digester, shown on Fig. 13–20(d), is similar in appearance to an upright egg, and the design is sometimes modified to a sphere-cone shape. Egg-shaped tanks have been used extensively in Europe, especially in Germany, and are growing in popularity in the United States. Essentially all of the modern digester designs in the United States are of either the cylindrical or egg-shaped type. Cylindrical and egg-shaped digesters and the mixing systems used for each type of tank are discussed in the following paragraphs. Advantages and disadvantages of each type of digester are summarized in Table 13–31.

Proper mixing is one of the most important considerations in achieving optimum process performance. Various systems for mixing the contents of the digesters have been used; the most common types involve the use of (1) gas injection, (2) mechanical stirring, and (3) mechanical pumping. Some digester installations use a combination of gas mixing and recirculation by pumping. The advantages and disadvantages of the various mixing systems are summarized in Table 13–32; typical design parameters are shown in Table 13–33.

Cylindrical Tanks. Cylindrical sludge digesters are seldom less than 6 m (20 ft) or more than 38 m (125 ft) in diameter. The water depth should not be less than 7.5 m (25 ft) at the sidewall because of the difficulty in mixing shallow tanks, and the depth may be as