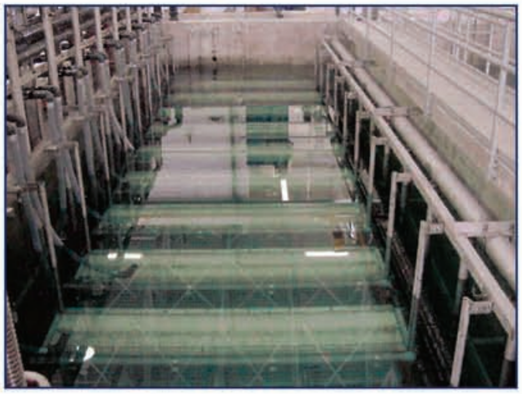
Municipal Nutrient Removal Technologies Reference Document

Volume 1 – Technical Report



U.S. Environmental Protection Agency
Office of Wastewater Management, Municipal Support Division
Municipal Technology Branch

EPA 832-R-08-006 • September 2008

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EPA 832-R-08-006, September 2008

This document was prepared by the following staff from the Ann Arbor, Michigan, and Fairfax, Virginia, offices of Tetra Tech, Inc., under U.S. Environmental Protection Agency (EPA) Contract EP-C-05-046.

Shin Joh Kang, Ph.D., P.E., Principal Author
Kevin Olmstead, Ph.D., P.E., Author
Krista Takacs, P.E., Author
James Collins, Author and Project Manager

Acknowledgements

EPA Office of Wastewater Management staff and the authors would like to acknowledge the efforts of the following EPA-state project workgroup members:

Dan Murray — EPA, ORD
David Pincumbe — EPA Region 1
Dave Ragsdale — EPA Region 10
Denny Rowland — CT, DEP
Gary Johnson — CT, DEP (formerly)
Dr. Ta-Shan Yu — MD, MDE

The authors appreciate the review and feedback on an early draft of the document that was conducted by the EPA Regions and also by several state permitting and water quality agencies, as coordinated by the Association of State and Interstate Water Pollution Control Agencies.

In addition, a formal technical review of the draft document was conducted by professionals with experience in wastewater treatment in accordance with EPA Peer Review Guidelines. While every effort was made to accommodate all of the Peer Review comments, the results and conclusions do not indicate consensus and may not represent the views of all the reviewers. The technical reviewers of this document included the following:

Ms. Jeanette Brown, Executive Director, Water Pollution Control Authority, Stamford, CT

Sudhir Murthy, Ph.D., P.E., Manager for Process Development, D.C. Water and Sewer Authority (DCWASA), Washington, DC

J.B. Neethling, Ph.D., PE, BECC, Senior Vice president, HDR, Inc., Folsom, CA

Amit Pramanik, Ph.D., BCEEM, Senior Program Director, Water Environment Research Foundation, Alexandria, VA

Clifford W. Randall, Ph.D., Professor, Virginia Polytechnic Institute and State University, Blacksburg, VA

H. David Stensel, Ph.D., PE, Professor, University of Washington, Seattle, WA

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Preface

The U.S. Environmental Protection Agency (EPA) is charged by Congress with protecting the nation's land, air, and water resources. Under a mandate of environmental laws, the Agency strives to formulate and implement actions leading to a balance between human activities and the ability of ecosystems to support and sustain life. To meet this mandate, the Office of Wastewater Management (OWM) provides information and technical support to solve environmental problems today and to build a knowledge base necessary to protect public health and the environment well into the future. This publication was prepared under contract to EPA, by Tetra Tech, Inc. The document provides current state of development as of the publication date; however, it is expected that this document will be revised periodically to reflect advances in this rapidly evolving area. Except as noted, information, interviews, and data development were conducted by the contractor. While there are many proven, cost-effective nutrient removal technologies and numerous new technologies or modifications of existing technologies available to detailed study, the case studies in this document were selected on the basis of specific criteria. The criteria included the ability to provide as least one year of full-scale operating and performance data, capability of providing detailed capital and operation and maintenance cost breakdowns, and the ability to provide the data within the time frame established for completing the document. It is anticipated that as the document is updated, additional case studies on new technologies could be included. Some of the information, especially related to emerging technologies, was provided by the manufacturer or vendor of the equipment or technology and could not be verified or supported by a full-scale case study. In some cases, cost data were based on estimated savings without actual field data. When evaluating technologies, estimated costs, and stated performance, efforts should be made to collect current and more up-to-date information.

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How to use this document

EPA is providing this reference document to make information available on recent advances in nutrient removal technology and practices. The goal of this document is straightforward – to provide information that will assist local decision makers and regional and state regulators plan cost-effective nutrient removal projects for municipal wastewater treatment facilities. Volume 1 provides key information on technologies, case studies, and capital and O&M costs for retrofitting or expanding existing facilities.

Chapter 1 provides a brief history of nutrient removal in the United States which sets the stage for the detailed presentation of the current status of nutrient removal practices and costs.

Chapter 2 includes detailed information on commonly used biological and physiochemical nutrient removal technologies and preliminary information on emerging phosphorus removal and side stream nitrogen removal processes. It presents detailed technical and cost information about both biological and physiochemical treatment technologies. The technical information includes detailed process descriptions and operating factors for more than 40 different treatment alternatives for removing nitrogen, phosphorus, or both from municipal wastewater streams. The information also includes data on process performance and reliability that were developed from full-scale operating data obtained voluntarily from 30 wastewater treatment plants throughout the United States and in Canada. This extensive analysis allows decision makers to evaluate full-scale performance data obtained from specific facilities.

Chapter 3 provides a synthesis of the information generated from the nine fully scale case studies. The case studies facilities represented a variety of technologies in both cold- and warm-weather locations were the subject of in-depth discussion of the factors involved in successful process design and operation, as well as a detailed process cost analysis. The full case study reports are provided in Volume 2 of this document.

Chapter 4 contains information on general cost estimates for many of the available nutrient removal technologies for both retrofits and expansion of existing facilities. The accuracy of the cost estimate will vary depending on the level of detail provided in the evaluation. If the cost estimate is based on cost curves or costs from similar facilities or technologies with very little consideration of local conditions, the cost estimate might be accurate to within only approximately 50 percent. If more detailed studies such as soil borings, preliminary engineering design drawings, and draft specifications are prepared, the estimate will be more accurate.

Chapter 5 presents a set of considerations and an approach for planning for process upgrades that includes projecting future loads, assessing existing capabilities, preparing a mass balance that includes all return and recycle flows and loads, developing the needed expansion and upgrade that should incorporate flexibility into the operation of the plant to account for future uncertainties, evaluating feasible alternatives, and selecting the recommended plan. Chapter 5 also presents a list of technologies capable of meeting the selected target effluent range for nitrogen, phosphorus, or both and technology selection factors to be considered in identifying and evaluating feasible technologies on the basis of design and operational and cost factors.

Volume 2, Appendix A contains detailed case study reports for each of the nine facilities evaluated as part of this project. The objective of the case studies was to present the data from selected technologies for a one-year period; to identify the factors that contribute to the reliability of nitrogen and phosphorus removal; to identify the factors that contribute to the costs of various removal technologies; and to evaluate the reliability of nutrient removal through a simple, yet sound statistical method by which performance data could be presented and compared.

Volume 2, Appendix B provides the technical and statistical basis of using coefficient of variation to describe performance reliability.

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Acronyms/Abbreviations

Al^3	Aluminum sulfate (or Alum)
ANAMMOX	anaerobic ammonia oxidation
ASCE	American Society of Civil Engineers
A/O	anoxic/oxic
A2/O	anaerobic/anaerobic/oxic
AS	activated sludge
AT 3	Aeration Tank 3 process
BABE	bio-augmentation batch enhanced
BAF	biological aerated filter
BAR	bioaugmentation reaeration
BASIN	biofilm activated sludge innovative nitrifiation
BNR	biological nutrient removal
BOD	biochemical oxygen demand
BOD-to-TKN	biochemical oxygen demand-to-total Kjeldahl nitrogen ratio
BOD-to-TP	biochemical oxygen demand-to-total phosphorus ratio
BPR	biological phosphorus removal
CANON	completely autotrophic nitrogen removal over nitrite
CAS	cyclic activated sludge
CBOD	carbonaceous biochemical oxygen demand
COD	chemical oxygen demand
COV	coefficient of variation
CP	central plant
CWSRF	Clean Water State Revolving Fund
DAF	dissolved-air flotation unit
DO	dissolved oxygen
DON	dissolved organic nitrogen
EBPR	enhanced biological phosphorus removal
ENR	*Engineering News-Record*

EPA	Environmental Protection Agency
$FeCl_3$	ferric chloride
FFS	fixed-film systems
GAO	glycogen accumulating organism
GPD	gallons per day
HDWK	headworks
HCO_3^-	bicarbonate
H_2CO_3	carbonic acid
HRT	hydraulic retention time
IFAS	integrated fixed-film activated sludge
I&C	instrumentation and control
kWh/year	kilowatt-hours per year
MAUREEN	mainstream autotrophic recycled enhanced N-removal
MBBR	moving-bed biofilm reactor
MBR	membrane bioreactor
MG	million gallons
MGD	million gallons per day
mg/L	milligrams per liter (equivalent to parts per million)
LE	modified Ludzack-Ettinger
MLSS	mixed liquor suspended solids
MW	molecular weight
N	nitrogen
NH_4	ammonium
NH4-N	ammonia nitrogen
NL	no limit
NPDES	National Pollutant Discharge Elimination System
OLAND	oxygen limited aerobic nitrification-denitrification
ORP	oxidation-reduction potential
O&M	operation and maintenance
PAO	phosphate accumulating organisms

PID	phased isolation ditch
PLC	programmable logic controller
PO_4^{3-}	phosphate
POTWs	publicly owned treatment works
RAS	return-activated sludge
rbCOD	readily biodegradable carbonaceous oxygen demand
rDON	refractory dissolved organic nitrogen
SCADA	supervisory control and data acquisition
SBR	sequencing batch reactor
SE	secondary effluent
SND	simultaneous nitrification and denitrification
SF	subsurface flow
SHARON	single reactor high-activity ammonia removal over nitrate
SRT	solids retention time
TMDL	total maximum daily load
TKN	total Kjeldahl nitrogen
TN	total nitrogen
TP	total phosphorus
TRPA	Tahoe Regional Planning Agency
TSS	total suspended solids
UCT	University of Cape Town
UV	ultraviolet
VFA	volatile fatty acids
VIP	Virginia Initiative process
VSS	volatile suspended solids
WAS	waste activated sludge
WEF	Water Environment Federation
WERF	Water Environment Research Foundation
WQBEL	water quality-based effluent limit
WWTP	wastewater treatment plant

Executive Summary

One of the major concerns regarding constituents in municipal wastewater treatment plant discharges is the concentration of nutrient compounds, in particular nitrogen and phosphorus. Nutrients stimulate the growth of microorganisms (including algae) and other aquatic vegetation in receiving waters, leading to decreased oxygen levels. Excess nutrients are a significant water quality concern in many of the nation's waters and a leading cause of impairment of designated uses. Wastewater treatment plants that employ conventional biological treatment processes designed to meet secondary treatment effluent standards typically do not remove total nitrogen (TN) or total phosphorus (TP) to an extent sufficient to protect certain receiving waters. Wastewater treatment facilities are increasingly being required to address this issue by implementing treatment processes that reduce effluent nutrient concentrations to levels that regulators deem sufficiently protective of the environment. Such implementation usually involves retrofitting the plant to enhance the biological treatment processes or to include chemical treatment to effect phosphate precipitation. The challenge for those facilities, however, is to determine which treatment alternatives will best meet their needs, both technically and financially, and to make the decision that is most sustainable.

Purpose of this Document

This reference document includes technical information developed to assist municipal decisionmakers and regional and state regulators in planning for nutrient removal from municipal wastewater streams. Consequently, it is not intended to be a design manual for use by engineers in generating design parameter values or drawings. It presents detailed technical and cost information about both biological and physiochemical treatment technologies. The technical information includes detailed process descriptions and operating factors for more than 40 different treatment alternatives for removing nitrogen, phosphorus, or both from municipal wastewater streams. The information also includes data on process performance and reliability that were developed from full-scale operating data obtained voluntarily from 30 wastewater treatment plants throughout the United States and in Canada. This extensive analysis allows decisionmakers to evaluate full-scale performance data obtained from specific facilities. Nine of the facilities were the subject of in-depth case studies that further examined the factors involved in successful process design and operation, as well as process cost analysis. Case study summaries are provided in Volume II, Appendix A.

Cost information for various technologies was also developed from literature sources, as well as from the facilities contacted for the case studies for this document. Capital and operation and maintenance (O&M) cost estimates in 2007 dollars were determined for the nine case study facilities. In addition, capital, O&M, and life-cycle costs were estimated for 12 retrofit and 20 expansion alternatives using CAPDETWorks software (Hydromantis Corporation, Ontario, Canada).

Content of Document

The document has five chapters. Chapter 1 provides a brief introduction and history of nutrient removal at wastewater treatment plants. Chapter 2 provides an overview of more than 40 alternative technologies that are available for providing nitrogen and phosphorus removal in municipal wastewater treatment. Nitrogen removal technologies are based on biological nitrification-denitrification because that is the generally preferred method for removing nitrogen. Both chemical and biological methods for phosphorus removal are described. The technologies discussed range in complexity from one-point chemical addition for phosphorus precipitation to a 5-stage Bardenpho system for combined biological phosphorus and nitrogen removal. The descriptions include process configurations, factors important in design and operation, and observed ranges of effluent concentrations. Chapter 3 of the document summarizes important findings of the case studies and associated technologies. Information on capital operations, maintenance, and life-cycle costs is provided in Chapter 4. Finally, Chapter 5 presents information about upgrading existing facilities for those who are evaluating the use of nutrient removal technologies.

Technologies, Performance, and Reliability

Chapter 2 provides operating results from 30 full-scale treatment facilities. Full-scale operating data were obtained from these facilities on a voluntary basis and analyzed for the reliability of nitrogen and phosphorus removal, as applicable under the facility's National Pollutant Discharge Elimination System (NPDES) permit or, in some cases, where facilities voluntarily achieved removal results above and beyond the NPDES requirements. Most of the facilities are throughout the United States; one is in Canada. It should be emphasized that the performance data for these facilities reflect differences in operating philosophy, permit limitations, temperature, influent conditions, flow conditions, and the relative plant load compared to design. Thus, the documented performance does not necessarily represent optimum operation of the technologies presented.

Common Statistical Base

Performance data from the facilities are presented in Tables 2-1, 2-5, and 2-8 for nitrogen removal, phosphorus removal, and combined nitrogen plus phosphorus removal, respectively. In general, performance was affected by both the selected technology and the permit limit for each substance. The data presented include the range of monthly average effluent concentrations observed at the facility, as well as concentrations corresponding to the statistically derived values for the annual average, maximum month, maximum week, and maximum day, as data were available. These concentrations were derived by plotting the monthly average effluent concentrations for a year in ascending order on probability paper.

The annual average corresponded to the average, or the 50 percent probability, while the *monthly maximum* corresponded to the 92 percent probability. The slopes of these *reliability curves* corresponded to the reliability, or *variability* as defined statistically: lower slopes meant that the process as implemented at that facility would produce more consistent effluent parameter values compared to processes implemented elsewhere with higher slopes. These slopes are related to the coefficient of variation (COV) for the data set, which is defined as one standard deviation divided by the mean. To attain the same target effluent concentrations (e.g., meet the same NPDES limits), treatment processes with a low (e.g., 20 percent) COV are considered more *reliable*, whereas those with a high (e.g., 60 percent) COV are considered less *reliable, because they are more variable*. Appendix B, in Volume II of this document, presents detailed information about this technique. COV alone might not be sufficient to evaluate the performance of treatment processes.

Performance of Technologies

Technologies are available to reliably attain an annual average of 0.1 milligram per liter (mg/L) or less for TP and 3 mg/L or less for TN. Reliability curves were developed for the 9 case studies and for 21 other facilities for which suitable data were acquired from plants that removed TN, TP, and both TN and TP. The reliability of the plants that were required to remove ammonia nitrogen is also included.

Nitrogen Removal

The nitrogen removal processes evaluated for this document all employ biological nitrification of ammonia nitrogen and organic nitrogen under aerobic conditions. Most of the systems also employ biological denitrification under anoxic conditions. Table 2-1 presents effluent TN concentrations reported by 19 facilities. The performances were grouped into three categories: (1) high effluent nitrogen (annual average concentrations above 5 mg/L TN); (2) medium effluent nitrogen (annual average concentrations between 3 and 5 mg/L); and (3) low effluent nitrogen (annual average concentrations below 3 mg/L TN). Reliability curves are presented in Chapter 2, Figures 2-31 and 2-33, for low and medium effluent TN, respectively.

The following seven systems reported annual average concentrations above 5 mg/L:

- Johannesburg process, Hagerstown, Maryland, full-year data, 7.86 mg/L, COV 21 percent
- Virginia initiative process (VIP), literature report, 6.12 mg/L, no COV available
- Step-feed activated-sludge (AS) process, Cumberland, Maryland, full-year data, 6.7 mg/L, COV 27 percent
- Anerobic/anox/oxic (A^2O) process, literature report, 7.3 to 9.0 mg/L, no location or COV available

- Schreiber process, literature report, 8 mg/L, no location or COV available
- Blue Plains process, as done at the Blue Plains facility in Washington, D.C., 7.5 mg/L, no COV available
- Step-feed AS, Fairfax County, Virginia, case study, 5.25 mg/L, 12 percent COV

Six processes were found to generate annual average effluent TN between 3 and 5 mg/L:

- Cyclic on-off (operational air adjustment), Ridgefield, Connecticut, full-year data, 4.59 mg/L, COV 25 percent
- Sequencing batch reactor, Thomaston, Connecticut, full-year data, 4.59 mg/L, 50 percent COV
- Modified Ludzack-Ettinger (MLE), Westminster, Maryland, full-year data, 4.35 mg/L, 23 percent COV
- Westbank process, Kelowna, British Columbia (Canada), case study, 4.38 mg/L, 12 percent COV
- Phased isolation ditches (PIDs), Jewett City, Connecticut, full-year data, 4.2 mg/L, 42 percent COV
- Biological anoxic filters, Cheshire, Connecticut, two-thirds-year data, 3.6 mg/L, 62 percent COV

Seven processes were found to produce annual average effluent TN levels below 3 mg/L:

- Integrated fixed-film activated sludge (IFAS), literature, 2.8 mg/L (low observed value), no location or COV available
- Concentric ring oxidation ditch, Hammonton, New Jersey, full-year data, 3 mg/L, 32 percent COV
- Step-feed AS, Piscataway, Maryland, full-year data, 2.58 mg/L, 57 percent COV
- 5-stage Bardenpho, Clearwater, Florida–Marshall Street facility, case study, 2.32 mg/L, 16 percent COV
- 5-stage Bardenpho, Clearwater, Florida–Northeast facility, full-year data, 2.04 mg/L, 42 percent COV
- Denitrification filter, Central Johnston County, North Carolina, case study, 2.14 mg/L, 16 percent COV
- Denitrification filter, Lee County, Florida, case study, 1.71 mg/L, 28 percent COV

- 3-sludge process including denitrifying AS, Western Branch, Maryland, case study, 1.6 mg/L, 36 percent COV

Phosphorus Removal

Table 2-5 in Chapter 2 presents effluent TP concentration reported by 17 facilities. Most systems employed chemical treatment to effect part or all of the TP removal, except as noted. The performances were grouped into two categories—low effluent phosphorus (annual average concentrations between 0.1 and 0.5 mg/L TP) and very low effluent phosphorus (annual average concentrations below 0.1 mg/L TP). The corresponding groups of reliability curves are presented in Chapter 2, Figures 2-35 and 2-36, for very low and low effluent TP, respectively.

The following six systems reported producing an annual average effluent TP between 0.1 and 0.5 mg/L:

- VIP, literature report, 0.4 mg/L, no COV available
- University of Cape Town (UCT) process with filter, Penticton, British Columbia, literature report, 0.3 mg/L, no COV available
- Anoxic/oxic (A/O) process (no chemical and no filter), Genesee County, Michigan, full-year data, 0.24 mg/L, 50 percent COV
- The *Westbank* process with fermenter and filter, Kelowna, British Columbia, case study, 0.139 mg/L, 12 percent COV
- A^2O with volatile fatty acid (VFA), chemical, tertiary clarifier, and filter, Durham, Oregon, full-year data, 0.132 mg/L, 33 percent COV
- Modified UCT with fermenter and filter, Kalispell, Montana, case study, 0.12 mg/L, 19 percent COV

Eleven systems reported producing an annual average effluent TP at or below 0.1 mg/L:

- A/O with filter, Clark County, Nevada, case study, 0.1 mg/L, 30 percent COV
- PhoStrip with filter, Truckee Meadows, Nevada, literature report, < 0.1 mg/L, no COV available
- Oxidation ditch with denitrification filter with alum, Lee County, Florida, case study, 0.098 mg/L, 47 percent COV
- Chemical addition with flocculating clarifier, Chelsea, Michigan, full-year data, 0.09 mg/L, 14 percent COV

- Step-feed AS with fermenter and filter, Fairfax County, Virginia, case study, 0.09 mg/L, 21 percent COV
- Membrane bioreactor, Hyrum, Utah, full-year data, 0.07 mg/L, 107 percent COV
- Chemical addition with tertiary clarifier and filter, McMinnville, Oregon, seasonal (6 months) data, 0.058 mg/L, 63 percent COV
- 5-Stage Bardenpho with chemical and filter, Pinery, Colorado, 0.031 mg/L, 34 percent COV
- Membrane bioreactor, Lone Tree Creek, Colorado, 0.027 mg/L, 27 percent COV
- Enhanced biological phosphorus removal with chemical addition and filter, Breckenridge, Colorado, literature report, 0.01 mg/L, no COV available
- Chemical addition with tertiary filter and infiltration basin, Brighton, Michigan, full year data, 0.01 mg/L, 0 percent COV

Combined Nitrogen and Phosphorus Removal

Table 2-8 in the document presents effluent TN and TP concentrations reported by 12 facilities. Most systems employed chemical treatment to effect part or all of the TP removal, except as noted. The facilities were divided into three groups: those with TN greater than 5 mg/L; those with TN less than 5 mg/L and TP between 0.1 mg/L and 0.5 mg/L; and those with TN less that 5 mg/L and TP less than 0.1 mg/L. The reliability curves for nitrogen removal for these facilities are included in Chapter 2, Figures 2-31 and 2-33; the reliability curves for phosphorus removal for these facilities are included in Figures 2-35 and 2-36.

The following five systems reported an annual average TN over 5 mg/L and variable TP:

- UCT, literature report, 8.9 to 10 mg/L TN, TP and location unspecified, COV not available
- IFAS, literature report, Broomfield, Colorado, 5.6 to 11.3 mg/L TN, 0.2 to 1.7 mg/L TP, COV not available
- VIP, literature report, 3 to 10 mg/L TN, 0.19 to 5.75 mg/L TP, location unspecified, COV not available
- VIP with VFA addition, literature report, 5 to 10 mg/L TN, 0.6 to 0.8 mg/L TP, COV not available
- Modified UCT with VFA addition, McDowell Creek, North Carolina, literature report, 5 to 6 mg/L TN, 0.1 to 2.7 mg/L TP, COV not available

Five systems reported annual average TN levels less than 5 mg/L and annual average TP levels between 0.1 and 0.5 mg/L:

- Biodenipho/PID, North Cary, North Carolina, case study, 3.55 mg/L TN, 0.31 mg/L TP, 26 percent COV (TN), 87 percent COV (TP)
- Three-stage AS system with denitrifying sludge and filter, Western Branch, Maryland, case study, 1.63 mg/L TN, 0.43 mg/L TP, 36 percent COV (TN), 62 percent COV (TP)
- 5-Stage Bardenpho with chemical addition and filter, Clearwater, Florida, Marshall Street facility, case study, 2.32 mg/L TN, 0.11 mg/L TP, 16 percent COV (TN), 64 percent COV (TP)
- 5-Stage Bardenpho with chemical addition and filter, Clearwater, Florida, Northeast facility, full-year data, 2.04 mg/L TN, 0.20 mg/L TP, 42 percent COV (TN), 82 percent COV (TP)
- Denitrification filter with chemical addition, Central Johnston County, North Carolina, case study, 2.14 mg/L TN, 0.26 mg/L TP, 16 percent COV (TN), 60 percent COV (TP)

Two systems reported an annual average TN less than 5 mg/L and an annual average TP less than 0.1 mg/L:

- Step-feed AS with fermenter and filter, Piscataway, Maryland, full-year data, 2.59 mg/L TN, 0.09 mg/L TP, 57 percent COV (TN), 89 percent COV (TP)
- Denitrification filter with chemical addition, Lee County, Florida, Fiesta Village facility, case study, 1.38 mg/L TN, 0.098 mg/L TP, 40 percent COV (TN), 47 percent COV (TP)

Performance Factors for Design and Operation

Key factors found to affect the performance of nutrient removal processes are discussed in Chapters 2 and 3.

Nitrogen Removal

For nitrogen removal, key factors are presented in Section 2.2.2. They include an adequate supply of carbon from internal or external sources, the number of anoxic zones, favorable temperature, sufficient alkalinity, the sludge age and maintenance of a deep sludge blanket in the secondary clarifier, and proper management of the recycle flows.

An adequate supply of carbon is needed to meet one of the following: chemical oxygen demand (COD)-to-total Kjeldahl nitrogen (TKN) ratio, readily biodegradable COD-to-TKN ratio, biochemical oxygen demand (BOD)-to-TKN ratio, or VFAs. A fermenter is a good source of internal carbon at a facility. If an insufficient amount of carbon is present inside the facility, however, external carbon sources are needed. Typically, methanol or another locally available material is selected.

The number of anoxic zone is a function of the target concentration and the existing facility. For facilities with a target concentration of 3 mg/L or less, typically two anoxic zones are required for a single-sludge system. Having a number of swing zones provides significant protection against changing wastewater characteristics and other conditions. In addition, the size of the anoxic zone depends on the carbon source: smaller zones are designed with a readily biodegradable carbon like VFAs, whereas larger zones are designed with a less-biodegradable carbon source in the wastewater. A separate denitrifying filter, however, is not affected by these factors.

Alkalinity is an essential requirement for nitrification, and its stoichiometry is well established. Denitrification in the same sludge system enables recovery of approximately 30 percent of the alkalinity in accordance with the stoichiometry. Supplementary alkalinity, in the form of caustic or lime, can be used in soft-water regions.

The sludge age is an operating parameter that varies from region to region, reflecting the temperature and changing characteristics of wastewater during the year. It varied from 10 to 50 days in the case studies.

Operationally, many facilities reported a strategy by which a significant amount of denitrification was accomplished by maintaining 3 to 4 feet of sludge blanket in the secondary clarifier.

Recycle flows contributed to a significant amount of nutrient and affected the performance of the technologies. Ammonia nitrogen from anaerobic digestion and dewatering are significant, and flow equalization reduces shock effects on the biological processes.

Phosphorus Removal

For phosphorus removal, the key factors are presented in Section 2.3. They are based on a system that consists of multiple processes conducted in series to reach an extremely low target concentration. This proven practice consists of a biological process, followed by a chemical process, and eventually by a physical process in which solids are separated from the effluent water. The key factors included, for biological removal, an adequate supply of VFAs in the wastewater (and the use of a fermenter to generate additional VFAs where needed), the size of the anaerobic and aerobic zones, the number of swing zones, the sludge age, the control of secondary release, and the depth of the sludge blanket in the secondary clarifier. For chemical removal, the key factors included the number of chemical application points, the dosage, the need for a tertiary clarifier, and the type of filters for final polishing. Management of recycle flows is another key factor for reliable operations.

The recommended carbon supply is expressed in many ways, which include a VFA-to-TP ratio of 4 or higher, a BOD-to-TP ratio of greater than 20, and a readily biodegradable COD-to-TP ratio of 15 or greater. If insufficient VFAs are present in the wastewater year-round, a fermenter is a necessity for reliable performance. The typical design includes a sludge age of 4 days or longer and a hydraulic retention time of longer than 6 hours, depending on the temperature and location. The sizes of the anaerobic and aerobic zones, along with the swing zone, are important factors for ensuring reliable performance. Because of varying wastewater characteristics during the year, the swing zone provides added assurance of reliable performance.

Maintaining 2 to 4 feet of sludge blanket in the secondary clarifier was reported to be beneficial to many facilities, including those described in the Kelowna, Lee County, and Central Johnston County case studies. Clark County, on the other hand, reported use of a no-blanket policy to prevent the secondary release of phosphorus.

Tertiary clarifiers were effective in reducing the load to the tertiary filter, where the target concentration was low. Fairfax County; Clark County; and Brighton, Michigan, have shown added flexibility and reliability.

Special filters were effective in reaching a phosphorus concentration below 0.1 mg/L. They included a membrane filter at Hyrum, Utah, and at Lone Tree Creek, Colorado, a Dynasand filter at Breckenbridge, Colorado, and a Trident filter at Pinery, Colorado. Land application of tertiary clarifier effluent through 6 feet of soil layer in Brighton produced an effluent concentration of 0.01 mg/L at all times, making it the most reliable of all the technologies.

The management of recycle flows is necessary to avoid high phosphorus concentrations from building up in the wastewater remaining in the facility (phosphorus that is not disposed of with the biosolids). Therefore, opportunities for sludge to be anaerobic for long periods should be minimized. This is done by avoiding the use of long-term sludge storage as well as avoiding the use of an anaerobic digester or thickener. The use of lime or ferric chloride to chemically remove phosphorus from the recycle stream might be needed in such instances.

Combined Nitrogen and Phosphorus Removal

For combined removal of nitrogen and phosphorus, the key factors presented above apply. In addition, a careful approach is necessary to balance the needs for each process in a single-sludge system (see Section 2.2 for details). These needs include optimal allocation of carbon sources, the size of the anoxic zone, control of dissolved oxygen and nitrate nitrogen, and the number and placement of chemical application points.

The VFA from the same sources can be allocated between the anaerobic zone and the anoxic zone, depending on the needs and the control philosophy. Both the step-feed AS and the Westbank processes use this principle with a varying split formula to produce reliable performance. Under this approach, the size of anoxic zone can be reduced in comparison with the normal-mode AS.

Good control of dissolved oxygen and nitrate nitrogen is achieved with online sensors and automation of control functions, as is the case in Clark County and Clearwater.

For two-sludge and three-sludge systems, the controls are separated, and thus less effort is required to balance the needs for nitrogen removal versus phosphorus removal. These systems include the denitrification filters at Lee County and Central Johnston County and the three-sludge system in Western Branch. A simpler control and good performance are advantages at these facilities; the disadvantages include the new facilities—with their larger footprint and high cost for carbon—required.

Wet-Weather Flow

The reliability of technologies also depends in part on how the facility manages wet-weather flows. The key factors are the size and location of the equalization basin, selection of treatment processes, and operational flexibility available.

The size of the equalization basin depends on the sewer system in place. The peaking factors and the ability to bypass upstream treatment process are site-specific constraints. The reliability of the overall technologies is ensured with a large-enough basin, as is the case in North Cary, which has 58 percent of the average daily flow rate; Fairfax County, which has 11.5 percent of the average daily flow rate in the raw wastewater and 20 percent in the secondary effluent; and Kelowna, with 7.5 percent of the daily average flow rate.

Step-feed AS offers a distinct advantage in maintaining a higher sludge inventory in the aeration basin while maintaining a low solids loading rate on the secondary clarifiers, as shown in Fairfax County and Kelowna.

To address extreme storm conditions, many facilities have developed storm modes of operation, under which aeration is suspended in a section of the aeration basin during the high-flow period. The size and duration depend strictly on the size of the storm and the layout of the facility.

The PID at North Cary has a storm mode of operation, which allows quick adjustment of the cycle time and thus affords protection of the sludge inventory from washouts.

Emerging Technologies

Chapter 2 also includes presents emerging technologies that show promise for effective nutrient removal but have not yet had extensive full-scale operation so that their performance and reliability could be assessed. These include the Co-Mag process, which entails ballasted flocculation with magnetic separation of the ballast; the Blue PRO process, which involves ferric chloride-aided adsorption of phosphorus on ferric oxide; and the Trident HS process, which employs metal precipitation of phosphorus, two-stage clarification, and filtration.

Case Studies Involving Nine Municipalities

Chapter 3 provides a synthesis of the information generated from the nine case studies that were developed for this document. Most of the case study facilities are throughout the United States; one plant is in Canada. Facilities in both cold and warm weather locations were included in the study. The facilities varied with respect to whether phosphorus removal, nitrogen removal, or both were required under the NPDES permit. All the facilities exhibited outstanding performance. The objective of the case studies was four-fold:

- To present the data from selected technologies for a one-year period
- To identify the factors that contribute to the reliability of nitrogen and phosphorus removal
- To identify the factors that contribute to the costs of various removal technologies
- To evaluate the reliability of nutrient removal through a simple, yet sound statistical method by which performance data could be presented and compared

The performance of the technologies in use at the case study facilities is summarized below.

For biological phosphorus removal, efficient and reliable performance was shown at four facilities, two of which had dedicated fermenters. The annual average effluent concentration was 0.12 mg/L in Kalispell (modified UCT process with fermenter) and 0.14 mg/L in Kelowna (3-stage Westbank process with fermenter), with corresponding COVs of only 19 percent and 21 percent, respectively. North Cary (PID process) had an average concentration of 0.38 mg/L with a COV of 64 percent, and Central Johnston County (MLE with long anoxic detention time and high internal recirculation) had an average concentration of 0.26 mg/L with a COV of 62 percent. Key factors included favorable

Case Study Locations

Kelowna, British Columbia (Canada)
Clearwater, Florida (Marshall Street Plant)
Lee County, Florida
Upper Marlboro, Maryland (Western Branch Plant)
Kalispell, Montana
Clark County, Nevada
Central Johnston County, North Carolina
North Cary, North Carolina
Fairfax County, Virginia (Noman Cole Plant)

wastewater characteristics, an on-site fermenter, and a good design that included flexible features like swing zones, multiple points of carbon feed, safeguards against secondary release and minimal return of recycle flows from sludge handling, and good controls developed and practiced based on key process parameters. Operational factors included sludge blankets (also referred to as denitrification blankets) in the secondary clarifiers, maintained at Central Johnston County and Kelowna; optimization of fermenter parameters throughout the year; and control strategies based on nitrate concentration, blanket monitoring, dissolved oxygen, and oxidation-reduction potential measurements at some facilities.

Three facilities practiced a combination of biological phosphorus removal and chemical addition. The average annual effluent concentrations were lowered down to 0.09 mg/L with COVs of 30 percent in Clark County (A/O process) and 21 percent in Fairfax County (step-feed AS process with fermenter). Clearwater (5-stage Bardenpho process) achieved an average concentration of 0.132 mg/L with a COV of 40 percent. Key factors for increased removal included the added flexibility of having a tertiary clarifier, multiple feed points for chemical addition, a good filter system, and prevention of secondary release.

Two facilities—Lee County and Western Branch—relied on adding chemicals to their AS processes for phosphorus removal. The average annual concentrations for the two facilities were 0.1 mg/L and 0.47 mg/L, with COVs of 56 and 62 percent, respectively.

For ammonia nitrogen removal, two facilities were studied. The Kalispell and Clark County facilities removed ammonia with the highest efficiency and reliability—0.07 mg/L and 0.12 mg/L as an annual average with COVs of 0 percent in Kalispell and 14 percent in Clark County. Contributing factors included adequate sludge age and a sufficient supply of oxygen.

Six facilities were required to remove TN, and they all met their respective permit limits efficiently and reliably. The average annual effluent concentrations were 2.32 mg/L with a COV of 16 percent in Clearwater (5-Stage Bardenpho), 3.7 mg/L with a COV of 14 percent in North Cary (PID), and 4.38 mg/L with a COV of only 12 percent at Kelowna (Westbank process). Denitrification filters were installed to remove nitrogen at Central Johnston County and Lee County; their performance was good and reliable. The average concentrations were 2.14 mg/L and 1.57 mg/L with COVs of 16 and 34 percent, respectively.

To achieve low effluent nitrogen concentrations, Western Branch added a denitrifying AS process with a methanol feed system to its existing two-stage AS-nitrifying sludge system. This created a three-sludge system, the performance of which was good and reliable. The average concentration was 1.7 mg/L with a COV of 36 percent. The key factors for this high reliability included favorable wastewater characteristics, an adequate supply of carbon, a flexible design with swing zones, separate control of mixing and aeration, multiple carbon feed points, minimization of recycle loads, and proper design of the sludge-handling processes. Operating factors included a denitrification blanket at some locations; proper sludge age; online monitoring; and automation of process control based on nitrate, dissolved oxygen, and oxidation-reduction potential at some locations.

Costs

Chapter 4 presents comprehensive cost information for nutrient removal technologies at municipal facilities. The information is based on the literature reviewed, the data collected specifically for the case studies prepared for this document, and the use of the CAPDETWorks software.

The literature from which cost information was obtained includes studies surveying facilities in Maryland, Connecticut and Colorado. The Maryland studies include the costs for capital and O&M (except labor). The Connecticut study had capital and O&M costs (including labor costs) for one facility and capital and O&M costs (excluding labor costs) for the other facility. The Colorado study includes only capital costs. These literature data were assumed to be accurate. The U.S. Environmental Protection Agency (EPA) does not assume responsibility for the reported data.

Additional cost information was obtained in conjunction with developing the case studies prepared for this report. The costs reported from these sources included capital costs updated to 2007 dollars using the *Engineering News-Record* Index, and O&M costs for energy and chemicals but not for labor or maintenance. These cost data were provided and verified by the study facilities and were assumed to be accurate. EPA does not assume responsibility for the reported data.

In addition, cost estimates were developed for all cost items using CAPDETWorks software for 12 retrofit and 20 expansion alternatives. The cost estimates are based on unit costs updated to 2007 using published cost index values. CAPDETWorks was run for three flow rates: 1 million gallons per day (MGD), 5 MGD, and 10 MGD. These rates were selected to represent the majority of facilities in the country. On the basis of information from EPA's Clean Watersheds Needs Survey database, nearly 80 percent of existing facilities are less than 1 MGD, and about 97 percent of those facilities are 10 MGD or less. The CAPDETWorks cost database was assumed to be accurate. Capital costs were automatically updated to 2007 dollars. EPA does not assume responsibility for the data.

Capital, O&M, and Life-Cycle Costs for Retrofits

Table ES-1 presents the incremental costs of retrofit technologies obtained from the literature, case studies, and projections using CapdetWorks arranged by the amount of treatment achieved by the various systems. The sources of the data are shown on the table, and the flow rate in each system is noted. CAPDETWorks results are presented below for 1- and 10-MGD flow; more complete cost curves are presented in Chapter 4. For all technologies, costs vary substantially, which reflects the selected technologies, chemical use, power use, and for CAPDETWorks estimates, projected labor requirements. The size of the system is a significant cost factor: Higher unit costs are associated with smaller facilities compared to larger facilities because of economies of scale.

Table ES-1. Cost estimates for retrofit technologies

Target concentration (annual average)	Initial concentration (annual average)	Technologies (conversion or add-on indicated by footnote)	Location	Flow rate (MGD)	Capital $/gpd capacity	O&M $/MG treated	Life-cycle $/MG treated
Total N target only							
TN, 5.1 mg/L	9.6 mg/L TN	Cyclic on/off aeration[a]	Ridgefield, CT	1	$0.20	$111	---
TN, 5.0 mg/L	7 mg/L TN	Denitrification filter[b]	Cheshire, CT	3.5	$1.65	$136	---
TN, 3 mg/L or less	8 mg/L TN	MLE→4-stage Bardenpho[a]	Seneca, MD	20	$0.21	$63	---
	8 mg/L TN	MLE→4-stage Bardenpho[a]	Freedom, MD	3.5	$0.99	---	---
	8 mg/L TN	MLE→4-stage Bardenpho[a]	Cumberland, MD	15	$1.10	$122	---
	15 mg/L TN	Lagoon→4-stage Bardenpho[a]	Hurlock, MD	1.5	$4.12	---	---
	8 mg/L TN	Denitrification filter[b]	Baltimore, MD	180	$1.39	---	---
	8 mg/L TN	Denitrification filter[b]	Cox Creek, MD	15	$1.74	$104	---
	6.5 mg/L TN	5-stage Bardenpho + denitrification filter[b]	Frederick, MD	7	$1.41	---	---
	40 mg/L TN[d]	Phased oxidation ditch[a]	CW	10	$0.47	$44	$157
	40 mg/L TN[d]	MLE retrofit[a]	CW	10	$0.71	$82	$164
	40 mg/L TN[d]	Step-feed retrofit[a]	CW	10	$0.65	$91	$245
	40 mg/L TN[d]	Denitrification filter[b]	CW	10	$0.71	$156	$324
TN, 1 mg/L	42 mg/L TN[e]	5-stage Bardenpho with MBR and reverse osmosis[b]	Las Virgenes, Calabasas, CA	16	$5.20	---	---

Table ES-1. Cost estimates for retrofit technologies *(continued)*

Target concentration (annual average)	Initial concentration (annual average)	Technologies (conversion or add-on indicated by footnote)	Location	Flow rate (MGD)	Capital $/gpd capacity	O&M $/MG treated	Life-cycle $/MG treated
Total P Target Only							
TP, 0.5 mg/L	5 mg/L TP	Fermenter retrofit, no filter[b]	CW	10	$0.18	$7	$50
	5 mg/L TP	1-point chemical addition, no filter[b]	CW	10	$0.03	$91	$98
	5 mg/L TP	Fermenter + sand filter retrofit[b]	CW	10	$0.44	$25	$130
TP, 0.1 mg/L	5 mg/L TP	Fermenter + sand filter + 1-point chemical addition[b]	CW	10	$0.47	$106	$218
	5 mg/L TP	2-point chemical addition + filter[b]	CW	10	$0.29	$215	$283
Ammonia-N + TP limits							
Ammonia-N+TP, 1.5 mg/L & 1 mg/L	37 mg/L Ammonia-N, 10 mg/L TP	Cyclic on/off aeration[a] for ammonia-N	Broomfield, CO	8	$1.00	---	---
Ammonia-N+TP, 1.5 mg/L & 1 mg/L	37 mg/L Ammonia-N, 10 mg/L TP	IFAS[a] for ammonia-N	Broomfield, CO	8	$0.85	---	---
Ammonia-N+TP, 1.5 mg/L & 1 mg/L	37 mg/L Ammonia-N, 10 mg/L TP	MBBR[b] for nitrification/ denitrification	Broomfield, CO	8	$1.70	---	---
Ammonia-N+TP, 1.4 mg/L & 1 mg/L	24 mg/L Ammonia-N, 4 mg/L TP	Modified UCT with fermenter and sand filter[c]	Kalispell, MT, case study	3	$3.03	$108	---
Ammonia-N+TP, 1 mg/L & 0.18 mg/L	18.9 mg/L Ammonia-N, 6.4 mg/L TP	Step-feed AS with dual-media and deep-bed filter + 1-point chemical addition[c]	Fairfax, VA, case study	67	$1.07	$106	---
Ammonia-N+TP, 0.6 mg/L & 0.2 mg/L	27 mg/L Ammonia-N, 5.8 mg/L TP	A/O with VFA and dual media filters and chemical addition[a]	Clark Co., NV, case study	100	$2.01	$183	---
TN+TP limits							
TN+TP, 6 mg/L & 0.25 mg/L	28.8 mg/L TN, 6 mg/L TP	3-stage Westbank with fermenters[a]	Kelowna, BC, case study	10.5	$3.25	$77	---

Table ES-1. Cost estimates for retrofit technologies *(continued)*

Target concentration (annual average)	Initial concentration (annual average)	Technologies (conversion or add-on indicated by footnote)	Location	Flow rate (MGD)	Capital $/gpd capacity	O&M $/MG treated	Life-cycle $/MG treated
TN+TP, 3.9 mg/L & 2 mg/L	56 mg/L TN, 7.7 mg/L TP	Oxidation ditch with sand filter[a]	North Cary, NC, case study	12	$2.84	$60	---
TN+TP, 3.7 mg/L & 1 mg/L	31.2 mg/L TN, 5.8 mg/L TP	Plug flow AS with denitrification filter[c]	Central Johnston Co., NC case study	7	$0.58	$221	---
TN+TP, 3 mg/L & 1 mg/L	24 mg/L TN, 3.7 mg/L TP	3-stage activated sludge with chemical addition[b]	Western Branch, MD, case study	30	$1.73	$165	---
TN+TP, 3 mg/L & 1 mg/L	28 mg/L TN, 5 mg/L TP	5-stage Bardenpho with sand filter + chemical addition[a]	Clearwater, FL (Marshall Street) case study	10	$2.95	$242	---
TN+TP, 3 mg/L & 0.5 mg/L	33.2 mg/L TN, 3.8 mg/L TP	Denitrification filter + chemical addition[b]	Lee Co., FL (Fiesta Village) case study	5	$2.79	$265	---
TN+TP, 3 mg/L & 0.1 mg/L	40 mg/L TN, 5 mg/L TP	PID retrofit with 1-point chemical addition, clarifier, and filter[a]	CW	10	$0.89	$199	$411
	40 mg/L TN, 5 mg/L TP	5-stage Bardenpho retrofit with chemical addition[a]	CW	10	$1.30	$256	$566
	40 mg/L TN, 5 mg/L TP	Nitrification/ chemical addition/ denitrification filter retrofit[b]	CW	10	$0.75	$448	$626

Notes:
Case = case study as described in Chapter 3; O&M does not include labor.
CW = cost from CAPDETWorks
A/O with VFA = anoxic/oxic enriched with volatile fatty acids
AS = activated sludge
CT = Connecticut Study. CT-1 plant included labor in the O&M cost; CT-2 did not.
IFAS = integrated fixed-film activated sludge
MD = Maryland Study. Incremental cost for retrofitting from 8 mg/L TN to 3 mg/L TN does not include labor in O&M.
MLE = modified Ludzack-Ettinger process
MBBR = moving-bed biofilm reactor
MBR = membrane bioreactor
PID = phased isolation ditch
Other = other literature sources
TN = total nitrogen as N
TP = total phosphorus as P

UCT = University of Cape Town process
For combined TN and TP technologies (TN+TP), first target number is for TN, second for TP
[a] Conversion
[b] Add-on
[c] Combination conversion and add-on
[d] Secondary effluent (as influent for add on processes)
[e] 5-stage Bardenpho with MBR and reverse osmosis removes both TN and TP but the study reported only on TN

Life-cycle costs were calculated for the CAPDETWorks results by first annualizing the capital cost at 20 years at 6 percent interest. The annualized capital cost was then added to the annual O&M cost to obtain a total annual cost. This cost was then divided by the annual flow to get the life-cycle cost per million gallons (MG) treated.

For nitrogen removal technologies, the capital costs ranged between $0.99 and $1.74 per gallon per day (gpd) capacity. The low cost represents converting AS facilities from meeting 8 mg/L to meeting 3 mg/L, while the high cost represents adding new denitrification filters.

For nitrogen removal, the O&M cost reported in the Maryland study ranged from $63 to $122 and did not include labor in the O&M cost. The Connecticut plant at $111 per MG treated included labor in the O&M cost, but the Connecticut plant at $136 per MG treated did not include labor in the O&M cost.

For chemical phosphorus removal, the capital costs ranged between $0.03 and $0.29 per gpd capacity. The low cost was for chemical feed equipment and storage tanks for one chemical addition point, while the higher cost was for two chemical addition points. For biological phosphorus removal, the costs ranged between $0.44 and $0.47 per gpd capacity. The costs were from conversion of the AS process and the addition of new fermenters.

For chemical phosphorus removal, the O&M cost range was $91 to $215 per MG treated. This cost included the cost of chemicals, power, labor, and the handling of the additional sludge caused by chemical addition. The low cost represents single-point chemical addition without filtration. The high cost represents two-point chemical feed with tertiary filtration to achieve a lower target limit, 0.1 mg/L.

The rest of the costs presented in Table ES-1 are for combined processes that remove phosphorus and a nitrogen species. The following rules were applied to rationally allocate costs between nitrogen and phosphorus removal during preparation of the case studies described in Chapter 3:

- Electrical power was proportional to oxygen demand (e.g., nitrification versus BOD removal).
- The external carbon source (e.g., methanol) was entirely for nitrogen removal.
- Fermenters were assumed to be entirely for phosphorus removal.

- The tertiary filter was assumed to be entirely for phosphorus removal.
- The denitrification filter (capital and O&M) was assumed to be 95 percent for nitrogen removal and 5 percent for phosphorus removal via filtration.
- The coagulation chemicals were assumed to be entirely for phosphorus removal.
- Other costs were allocated in proportion to the hydraulic retention times for nitrogen and phosphorus removal.

Although the focus of this document is the removal of both nitrogen and phosphorus, some of the study facilities have limits for only phosphorus and ammonia nitrogen. The capital cost estimates for these facilities ranged from $0.85 per gpd capacity (Broomfield, Colorado, for IFAS installation) to $3.03 per gpd capacity (Kalispell). O&M costs were available for only three case study facilities—Kalispell, Clark County, and Fairfax County. They ranged from $106 to $183 per MG treated; the lower numbers resulted from use of a fermenter, rather than chemical addition, to obtain TP removal.

For combined nitrogen and phosphorus removal technologies, capital costs ranged between $0.58 and $3.25 per gpd capacity. The high-cost facilities typically included a fermenter, additional piping for the anoxic zone, swing zones, and filters. The factors that most affected O&M costs were the use of an external carbon source, power for the equipment installed, the use of chemical coagulants and polymers, and increased sludge production.

The combined nitrogen and phosphorus removal O&M costs ranged from $60 to $265 per MG treated for the case study facilities. Those costs did not include labor estimates. The lower costs were associated with systems that use fermenters to aid in biological phosphorus removal (with no chemical addition), while the higher costs were associated with chemical addition for phosphorus or nitrogen removal.

The O&M cost estimates generated using CAPDETWorks ranged from $199 to $448 per MG treated. The CAPDETWorks estimates included labor costs. The low costs reflect one-point chemical feed, while the high cost represents methanol feed, chemicals for phosphorus removal, and filtration. The life-cycle cost range was from $411 to more than $1,000 per MG treated. The wide range of life-cycle costs reflects the selected unit operations, chemical use, power use, and projected labor requirements. Details of the costs are provided in Chapter 4.

For nitrogen removal, nitrification/denitrification attained 5–8 mg/L without extra chemicals, using a single anoxic zone; the capital cost ranged from $0.63 to $2.17 per gpd capacity. The low cost was for a PID, and the high cost was for a 4-stage Bardenpho. The range of O&M costs was from $122 to $453 per MG treated, representing all cost items, including chemicals, labor, sludge handling, and power.

Capital, O&M, and Life-Cycle Costs for Expansion

Table ES-2 presents the CAPDETWorks-generated cost estimates for 20 expansion technologies for a 10-MGD average flow for the indicated target nutrient concentrations. The term *expansion*, as used here, is defined as a parallel train and no increase in design flow. Chapter 4 presents a full set of cost curves, including values for 1 and 5 MGD. For all technologies, costs vary; higher unit costs are associated with smaller facilities compared to larger facilities because of economies of scale. In general, costs are higher for the expansion processes compared to corresponding retrofit processes because there is no opportunity to use the existing facilities. Chapter 4 also includes bar graphs that show the breakdown of O&M for the expansion technologies into labor, energy, material, and chemical/sludge costs. The graphs show that the percentage of the O&M cost devoted to labor was higher for 1-MGD plants (labor at 40 to 50 percent of total O&M) than for 10-MGD facilities (labor at 15 to 25 percent of total O&M). The percentages devoted to energy and chemical costs remained proportional to the flow.

For chemical phosphorus removal, the cost ranged from $0.03 to $0.29 for chemical feed equipment and storage tanks. The O&M cost range was $91 to $215 per MG treated, based on one or two chemical feed points.

For biological phosphorus removal to reach 1 mg/L without extra chemicals, the cost was estimated at $1.21 per gpd capacity. With a fermenter and tertiary filter, the cost went up to $1.52 per gpd capacity, and the O&M cost went up from $280 to $308 per MG treated.

For combined nitrogen and phosphorus removal, the capital cost range was $1.36 to $2.48 per gpd capacity. The lower costs were for target TN concentrations of 5 mg/L and target TP concentrations of 1 mg/L, while the high costs were for target concentrations of 3 mg/L TN and 0.1 mg/L TP. The higher costs also reflect the use of two chemical addition points and methanol feed. The O&M costs ranged from $259 to $477. The low cost represents high target TN and TP, while the high cost represents low target TN and TP limits, as shown in Table ES-2.

To achieve a TP level of 0.1 mg/L or below, two-point chemical addition followed by a clarifier and high-performance filters might be needed. High-performance filters that can remove phosphorus to this level include the Dynasand D2, DensaDeg, membrane filters, and Trident. Emerging technologies include CoMag and Blue PRO. Cost information for these processes should be obtained from the manufacturers of the technologies.

Table ES-2. Cost estimates for expansion technologies for 10 MGD

Nutrient/ target concentration	Initial concentration	Technology	Capital $/gpd capacity	O&M $/MG treated	Life-cycle $/MG treated
TN, 5 mg/L	40 mg/L TN	PID	$0.63	$122	$273
	40 mg/L TN	MLE	$1.61	$309	$695
	40 mg/L TN	4-stage Bardenpho	$2.17	$453	$971
	40 mg/L TN	Denitrification filter	$0.71	$156	$324
TP, 1 mg/L	5 mg/L TP	A/O, no additional equipment	$1.21	$280	$568
TP, 0.5 mg/L	5 mg/L TP	1-point chemical addition	$0.03	$91	$98
	5 mg/L TP	A/O with fermenter	$1.26	$290	$590
	5 mg/L TP	A/O with fermenter and sand filter	$1.52	$308	$670
TP, 0.1 mg/L	5 mg/L TP	2-point chemical addition with filter	$0.29	$215	$283
	5 mg/L TP	A/O with fermenter, filter, and chemical addition	$1.55	$389	$758
TN+TP, 5 mg/L & 1 mg/L	40 mg/L TN, 5 mg/L TP	Step-feed	$1.36	$299	$625
	40 mg/L TN, 5 mg/L TP	SBR	$1.94	$302	$766
	40 mg/L TN, 5 mg/L TP	3-stage processes (e.g., UCT, VIP)	$2.05	$436	$925
TN+TP, 5 mg/L & 0.5 mg/L	40 mg/L TN, 5 mg/L TP	5-stage Bardenpho, no filter	$2.19	$452	$975
	40 mg/L TN, 5 mg/L TP	Modified UCT with fermenter and filter	$2.33	$456	$1014
	40 mg/L TN, 5 mg/L TP	5-stage Bardenpho with filter	$2.45	$455	$1040
TN+TP, 5 mg/L & 0.1 mg/L	40 mg/L TN, 5 mg/L TP	PID with chemical addition, clarifier, and filter	$0.83	$259	$456
	40 mg/L TN, 5 mg/L TP	SBR with chemical addition and filter	$1.87	$387	$834
TN+TP, 3 mg/L & 0.1 mg/L	40 mg/L TN, 5 mg/L TP	Nitrification with 1-point chemical addition and denitrificiation filter	$0.75	$448	$626
	40 mg/L TN, 5 mg/L TP	5-stage Bardenpho with chemical addition and filter	$2.48	$477	$1070

Notes:
A/O = anoxic/oxic
PID = phased isolation ditch
SBR = sequencing batch reactor
TN = total nitrogen as N
TP = total phosphorus as P
UCT = University of Cape Town process
VIP = Virginia Initiative process
For combined TN and TP technologies (TN+TP), first target number is for TN, second for TP.

Technology Selection and Permit Limits

Chapter 5 of the document describes how to evaluate options for nutrient removal retrofits. The technologies are summarized in Tables 5-1 through 5-4. The first consideration is to determine what the candidate processes should be, which depends on the target concentration of the nutrient and the existing facility.

The review of data sources performed in preparing this document identified six technologies that meet low (less than 3 mg/L) TN concentration limits with good reliability. These are the 4-stage Bardenpho process with a conventional filter; the step-feed AS process with a conventional filter; concentric (phased) oxidation ditches; and two types of denitrification filters, one downflow and the other upflow.

The technologies identified for achieving mid-level TN removal concentrations (between 3 and 8 mg/L) are the A^2/O, biological aerated filter, the MLE process, sequencing batch reactors, cyclic AS, biological aerated filters, IFAS, moving bed biofilm reactor, 3-stage Westbank, 4-stage Bardenpho and post-aeration anoxic with methanol (known as the Blue Plains process). The nitrogen removal technologies mentioned in the preceding paragraph were also identified as being able to reliably meet TN concentrations of 8 mg/L or higher without the use of filters.

The technologies that meet low (less than 0.1 mg/L) TP concentration limits with good reliability are membrane filters, proprietary high-performance filters (Trident, Dual Sand D2, Advanced Filtration System, and land application through infiltration bed. The emerging technologies include the priopRietary Blue PRO and CoMag process.

The additional technologies identified for meeting mid-level TP removal concentrations (between 0.1 and 0.5 mg/L) are chemical precipitation with conventional filter, 3-stage Westbank with conventional filter, chemical precipitation with tertiary clarifier and conventional filter, modified UCT process with conventional filter, PIDs with conventional filter, 5-stage Bardenpho with chemical and conventional filter, and step-feed AS with conventional filter. The conventional filters include sand filters, deep bed anthracite filters, dual-media filters, and other traditional filters.

The technologies that meet TP concentrations between 1.0 and 0.5 mg/L are chemical precipitation, A/O with conventional filter preferred, 5-stage Bardenpho, the proprietary PhoStrip process with conventional filter preferred, and sequencing batch reactors.

The technologies that meet concentrations of 3 mg/L or less of TN and 0.1 mg/L or less of TP are step-feed AS plus a tertiary clarifier with chemical addition and a high-performance filter, denitrification filter with chemical addition and a high-performance filter, membrane bioreactors with chemical addition, and the use of land application associated with any of the three technologies listed above.

Additional technologies that meet TN concentrations of less than 3 mg/L and achieve TP concentrations between 0.1 and 0.5 mg/L were identified. These are step-feed AS with a conventional filter, 5-stage Bardenpho with a conventional filter, PID with a conventional filter, denitrification filters plus chemical phosphorus removal and conventional filtration, 4-stage Bardenpho process plus chemical phosphorus removal with conventional filtration and denitrification, and AS plus chemical phosphorus removal with conventional filtration.

The technologies identified as meeting TN concentrations between 3 and 8 mg/L and meeting TP concentrations between 0.5 and 1 mg/L are sequencing batch reactors with conventional filtration, the modified UCT process, the A/O process, the PhoStrip II process, 3-stage Westbank, step-feed AS, PIDs, and 5-stage Bardenpho, as well as the following technologies with chemical phosphorus removal added: A^2/O, biological aerated filter, the MLE process, cyclic AS, biological aerated filters, IFAS, moving bed biofilm reactor, 4-stage Bardenpho, and post-aeration anoxic with methanol (known as the Blue Plains process). Sequencing batch reactors and the 3-stage Westbank process can also achieve these limits if accompanied by chemical phosphorus removal.

Factors in Upgrading Existing Facilities, Decision Matrix, and Sensitivity Analysis

Factors to be considered in upgrading existing facilities include wastewater characteristics, site constraints, existing solids-handling facilities, wet-weather flows, automation and sensors, and sustainability. These factors are listed in Chapter 5, Tables 5-5 through 5-7. Among other factors, the wastewater characteristics determine the need for fermenters for VFA production for use in biological nutrient removal. Fermenters can produce enough carbon to supplement the wastewater for both biological phosphorus removal and denitrification. An external carbon supply should be avoided for long-term sustainability, where feasible.

Site constraints could determine whether it is feasible to add new processes or whether there is only enough space to modify the existing facilities. Thus, such constraints are a key factor in making decisions regarding upgrades.

Existing on-site sludge-handling facilities have a significant impact on selecting technologies and eventually sizing the selected technologies. One needs to determine whether a side-stream treatment is needed or whether there is a need to oversize the main treatment processes to handle recycle loadings from solids thickening, digestion, and dewatering. For biological phosphorus removal processes, aerobic thickening and digestion are recommended. For facilities with anaerobic digestion, proper recycle loads should be incorporated into process sizing and selection.

Wet-weather flows are a significant factor in selecting technologies, as well as in incorporating an equalization basin. The size of the basin and its location are usually based on the site-specific flow conditions.

Automation and the use of online sensors are significant factors in managing energy, labor, and chemicals. Cost savings from reduced energy use is a key justification for the investment, as well as a reliable process control measure. Online sensors monitor dissolved oxygen, ammonia nitrogen and nitrate nitrogen, ortho-phosphate, oxidation reduction potential, and some specific parameters of local interest. Supervisory control and data acquisition (SCADA) systems are usually recommended for monitoring and, in some cases, for process control and optimization.

Regulators and municipalities evaluating nutrient removal technologies should consider sustainability as a key criterion in selecting a technology. One of the great challenges in the wastewater sector is to provide reasonable cost treatment that will enable reuse of the effluent, as well as recovery of the nutrients, and thereby reduce the overall demand on water resources. The costs of nutrient removal are affected significantly by the selection of a technology. Each technology requires a certain level of energy consumption, requires a particular amount of treatment chemicals, and has implications for the potential reuse of effluent and biosolids.

The process evaluation tables (decision matrices) in Chapter 5 show specific criteria that project proponents can develop on the basis of stakeholder input. These criteria and the weighting factors are based on the site-specific conditions of the facility and the priority of the project. Typical criteria include site requirements and the room for future plant expansion, costs (capital and life-cycle costs), the efficiency and sustainability of treatment, and flexibility for accommodating future changes in wastewater characteristics. Sustainability factors include the energy (electrical) consumption, the external carbon source (e.g., methanol) requirement, the potential for methane gas recovery and power generation, the sludge reuse potential, odors, and public perceptions. It is recommended that a sensitivity analysis be conducted in anticipation of increased costs associated with rising energy costs, increased biosolids management costs, uncertainties in waste characteristics, uncertainties in regulations, and other costs, assuming increases of 50 percent or 100 percent. Alternatives should be compared with respect to these factors in making the final technology selection.

CHAPTER 1: Introduction and History

1.1 Overview

Wastewater treatment plants (WWTPs) that use conventional biological treatment processes designed to meet secondary treatment effluent levels do not remove nitrogen or phosphorus to any substantial extent. Retrofitting the conventional secondary biological treatment processes is an increasingly popular way to achieve nitrogen and phosphorus reductions. This approach typically relies on modifications to biological treatment processes so that the bacteria in these systems also convert nitrate nitrogen to inert nitrogen gas and trap phosphorus in the biosolids that are removed from the effluent.

This document presents information about available technologies that can be used to remove nitrogen and phosphorus from municipal wastewater. Descriptions of the technologies are presented, along with data that show the cost and reliability of the performance that was achieved for specific applications of the technology. One of the purposes of the resources presented in this document is to help regulators develop appropriate discharge permit limits with a full understanding of available technologies, the reliability of the technologies, and the ability of plants that are retrofitted with such technologies to meet their permit limits in a sustainable way. Another key objective of the document is for this information to be available for assimilation by the regulated community so that plant operators and other decisionmakers, including rate payers, can be better informed about this topic.

Information about many of the technologies described in this document was obtained by developing a series of nine case studies. Additional information about these and other technologies was gathered from the literature, including publications of the Water Environment Federation, Water Environment Research Foundation, other organizations, recent reports from Maryland and Connecticut, and data from about 20 municipalities that participated voluntarily by providing data during the case study development process. One of the unique aspects of this document is that full-scale plant performance data were collected, and this information is expressed on the same statistical basis for easy comparison. The data are accompanied by a discussion of the key factors that contribute to the performance reliability of various process systems. These factors include wastewater characteristics; process parameters; environmental parameters such as temperature, chemistry, and biology; and operating parameters. Cost information is presented, as are factors that should be considered when selecting a retrofit or new upgrade process technology for removing nitrogen, phosphorus, or both.

1.2 Background of Nitrogen and Phosphorus Removal in the United States

The Federal Water Pollution Control Act of 1965 began a series of environmental legislative reforms that initiated a consistent approach to pollution control based on water quality and beneficial use goals (USEPA 1993). During the late 1960s and throughout the 1970s, much effort was devoted to reducing phosphorus in wastewater effluents (Rast and Thornton 1996). For example, the implementation of phosphorus removal at the Blue Plains WWTP in Washington, D.C, resulted in dramatic water quality improvement in the Potomac River.

The 1972 Amendments to the Federal Water Pollution Control Act, known as the Clean Water Act (CWA), established the foundation for wastewater discharge control in the United States. With respect to nonpoint source pollution, section 208 of the CWA Amendments of 1972 stipulated that “substate watersheds in which nonpoint source pollution control, along with the control of point source discharges by required technologies, was to be addressed by a watershed water quality plan” (National Academy of Sciences 1999). Since 1972, the U.S. Environmental Protection Agency (EPA) has awarded $44.6 billion under the sewage-treatment construction grants program. The 1987 CWA Amendments authorized an additional $18 billion, total, for the construction grants program through 1990 and the State Revolving Fund (SRF) program through 1994. Of this amount, $9.6 billion is authorized for continuation of construction grants, and at least $8.4 billion is for use as capitalization grants to set up the SRFs (USEPA 1998).

Although nearly all WWTPs provide a minimum of secondary treatment, conventional secondary biological treatment processes do not remove the phosphorus and nitrogen to any substantial extent. Tertiary treatment can remove nitrogen and phosphorous through carefully designed chemical reactions that generate easily isolated products such as precipitates and gases, though it is considered a costly technology (Carberry 1990).

Advanced treatment technologies can be extensions of conventional secondary biological treatment to remove nitrogen and phosphorus. Biological treatment processes called biological nutrient removal (BNR) can also achieve nutrient reduction, removing both nitrogen and phosphorus. Most of the BNR processes involve modifications of suspended-growth treatment systems so that the bacteria in these systems also convert nitrate nitrogen to inert nitrogen gas and trap phosphorus in the solids that are removed from the effluent (USEPA 2004b).

In addition to conventional tertiary treatment, there are half-tertiary treatment methods that remove some minerals and organic material, such as the method used by the WWTP in Orange County, California. In addition, constructed wetlands are now being looked at as a cost-effective and technically feasible approach to treating wastewater. In 2004 there were approximately 1,000 constructed wetlands in operation in the United States. Constructing wetlands is often less expensive than building traditional wastewater treatment facilities. In addition, wetlands have low operation and maintenance (O&M) costs and can handle fluctuating water levels (USEPA 2004a).

The following discussion provides some examples of nutrient removal technologies that have been used or are currently in use in the United States. The discussion is not intended to be an exhaustive history of nutrient removal from municipal wastewater. EPA recognizes the substantial efforts being undertaken in many parts of the country to reduce nutrients, including many mid-continent locations, Long Island Sound, Puget Sound, and numerous other estuaries on the Atlantic, Gulf of Mexico, and Pacific coasts.

1.2.1 Lake Tahoe

One of the earliest tertiary treatment plants was introduced by South Lake Tahoe, California, and Stateline, Nevada, to protect Lake Tahoe. The threat to the lake became serious in the late 1950s when the population along the south shore of the 21-mile-long lake increased sixfold in addition to an increase in the number of tourists. Septic tanks were used for many years before the construction of WWTPs in the 1960s (Lake Tahoe Environmental Education Coalition 2005).

The South Tahoe Public Utility District was formed in 1950 with the intention of treating sewage from the south shore, including Nevada’s casinos. Later in that decade, the Nevada side formed its own district, and the South Tahoe Public Utility District took responsibility for sewage from the city of South Lake Tahoe and from Eldorado County (Lake Tahoe Environmental Education Coalition 2005).

By 1965 an innovative advanced tertiary treatment plant, which treated sewage to drinking water standards, was installed. Effluent from the plant is not discharged to Lake Tahoe; instead, it is pumped through a 27-mile pipeline over Luther Pass into a storage reservoir in Alpine County. All effluent from the city of South Lake Tahoe and Eldorado County areas has been exported through this system since April 1968 (Lake Tahoe Environmental Education Coalition 2005).

Once the export system was completed, the treatment plant reduced its treatment to advanced secondary treatment, which disinfected the treated effluent water but left trace amounts of nitrogen and phosphorus. This effluent water is stored in a reservoir near Woodfords during the winter and released for use by ranchers to irrigate their pastures and alfalfa crops in the summer (Lake Tahoe Environmental Education Coalition 2005).

On California’s northern and western shores, two local jurisdictions collect all the sewage in the sanitary sewers and pump it to Truckee. The Tahoe City Public Utility District and the North Tahoe Public Utility District cooperated under a Joint Powers Agreement to build a joint sewerage facility in 1968 to treat the sewage of both districts (Lake Tahoe Environmental Education Coalition 2005).

The Tahoe Regional Planning Compact was adopted in 1969 when the California and Nevada legislatures agreed to create the Tahoe Regional Planning Agency (TRPA) to protect Lake Tahoe (Tahoe Regional Planning Agency 1980). The compact, as amended in 1980, defines the purpose of the TRPA:

> To enhance governmental efficiency and effectiveness of the Region, it is imperative there be established a Tahoe Regional Planning Agency with the powers conferred by this compact including the power to establish environmental threshold carrying capacities and to adopt and enforce a regional plan and implementing ordinances which will achieve and maintain such capacities while providing opportunities for orderly growth and development consistent with such capacities (Tahoe Regional Planning Agency 1980).

In 1978 the Tahoe-Truckee Sanitation Agency built a state-of-the-art tertiary treatment plant in Truckee. Since then, the Tahoe City and North Tahoe Public Utility Districts have collected all sewage between D.L. Bliss State Park at Emerald Bay and Kings Beach and have transported it to the Tahoe-Truckee Sanitation Agency plant through pipelines. The tertiary-treated effluent from this plant is injected into the ground (Lake Tahoe Environmental Education Coalition 2005).

1.2.2 Great Lakes

The Great Lakes region led the nation in developing successful nutrient control and contaminant cleanup strategies during the 1970s. The deterioration of Lake Erie in the 1960s due to eutrophication prompted bilateral actions by Canada and the United States to sign the first Great Lakes Water Quality Agreement in 1972. The Agreement outlined abatement goals for reducing phosphorus loads primarily from laundry detergents and municipal sewage effluent. In response to this commitment, Canada and Ontario enacted legislation and programs for controlling point sources. Between 1972 and 1987, Canada and Ontario invested more than $2 billion in sewage treatment plant construction and upgrading in the Great Lakes Basin (Environment Canada 2006).

The 1978 Agreement between the United States and Canada on Great Lakes Water Quality, which followed the original 1972 Agreement, contains the following text:

> 3. Nutrients
>
> (a) Phosphorus
>
> The concentration should be limited to the extent necessary to prevent nuisance growths of algae, weeds and slimes that are or may become injurious to any beneficial water use. (Specific phosphorus control requirements are set out in Annex 3.)
>
> ANNEX 3
>
> Control of Phosphorus
>
> 1. The purpose of the following programs is to minimize eutrophication problems and to prevent degradation with regard to phosphorus in the boundary waters of the Great Lakes System. The goals of phosphorus control are:
>
> (a) Restoration of year-round aerobic conditions in the bottom waters of the Central Basin of Lake Erie;
>
> (b) Substantial reduction in the present levels of algal bio-mass to a level below that of a nuisance condition in Lake Erie;
>
> (c) Reduction in present levels of algal biomass to below that of a nuisance condition in Lake Ontario including the International Section of the St. Lawrence River;
>
> (d) Maintenance of the oligotrophic state and relative algal biomass of Lakes Superior and Huron;
>
> (e) Substantial elimination of algal nuisance growths in Lake Michigan to restore it to an oligotrophic state; and
>
> (f) The elimination of algal nuisance in bays and in other areas wherever they occur.
>
> 2. The following programs shall be developed and implemented to reduce input of phosphorus to the Great Lakes:
>
> (a) Construction and operation of municipal waste treatment facilities in all plants discharging more than one million gallons per day to achieve, where necessary to meet the loading allocations to be developed pursuant to paragraph 3 below, or to meet local conditions, whichever is more stringent, effluent concentrations of 1 milligram per liter total

> phosphorus maximum for plants in the basins of Lake Superior, Michigan, and Huron, and of 0.5 milligram per liter total phosphorus maximum for plants in the basins of Lakes Ontario and Erie.
>
> (b) Regulation of phosphorus introduction from industrial discharges to the maximum practicable extent.
>
> (c) Reduction to the maximum extent practicable of phosphorus introduced from diffuse sources into Lakes Superior, Michigan, and Huron; and a substantial reduction of phosphorus introduced from diffuse sources into Lakes Ontario and Eric, where necessary to meet the loading allocations to be developed pursuant to paragraph 3 below, or to meet local conditions, whichever is more stringent.
>
> (d) Reduction of phosphorus in household detergents to 0.507 percent by weight where necessary to meet the loading allocations to be developed pursuant to paragraph 3 below, or to meet local conditions, whichever is more stringent.
>
> (e) Maintenance of a viable research program to seek maximum efficiency and effectiveness in the control of phosphorus introductions into the Great Lakes (CIESIN 1995).

As reported in the 1995 biennial progress report, all the U.S. and Canadian open-water phosphorus target levels have been achieved through combined efforts to improve the performance of sewage treatment plants, to reduce levels of phosphorus in detergents, and to implement agricultural best management practices. Current loads are clearly below the target loads of the 1978 Agreement for Lakes Superior, Huron, and Michigan and are at or near target limits for Lakes Erie and Ontario. Lake Erie still experiences brief periods of anoxia in some areas of its central basin. The 1997 *State of the Great Lakes Report* reviewed nutrient data since 1994 and concluded that no appreciable change had occurred in the nutrient status of the lakes and that the lakes continued to meet the targets for phosphorus reduction in the agreement. Continuing success is attributed to implementing a number of programs to control soil erosion and sedimentation, as well as other forms of nonpoint source control (USEPA 2006).

1.2.3 The Occoquan Reservoir and the Chesapeake Bay

The Occoquan Reservoir serves as a drinking water supply for a service area of more than 1 million people in the northern Virginia suburbs of Washington, D.C. The *Occoquan Policy* was adopted by the Virginia State Water Control Board in 1971. This Policy commissioned the development of a regional treatment plant, the Upper Occoquan Sewage Authority

(UOSA) Advanced Wastewater Treatment (AWT) Plant. When it went online in 1978, the UOSA AWT Plant replaced 11 conventional secondary treatment plants in Fairfax and Prince William counties. The quality of effluent from these plants was variable, and no provisions were made for the removal of nutrients from the discharges. In contrast, the regional UOSA AWT Plant included monthly average nutrient limits of 0.1 milligrams per liter (mg/L) for Phosphorus and 1.0 mg/L for unoxidized nitrogen measured as total Kjeldahl nitrogen (TKN) (Occoquan Monitoring Lab 2008 and Randall 2008). Developed to protect the drinking water supply (Occoquan Reservoir) for the Virginia suburbs of Washington, D.C., this plant currently treats 50 MGD and is the largest indirect sewage-to-drinking water system in the world. It is unique in that, although originally designed to remove nitrogen by ion exchange removal of NH4-N, most of the nitrogen is discharged in the form of nitrates to a stream that enters the reservoir to reduce the release of phosphorus from the sediments, thereby helping control algae blooms and protecting water quality (Randall 2008).

In the late 1970s and early 1980s, Congress funded scientific and estuarine research in the Chesapeake Bay, pinpointing three areas, including nutrient enrichment, as requiring immediate attention. The Chesapeake Bay Program was created in 1983 after the signing of The Chesapeake Bay Agreement of 1983 (Chesapeake Bay Program 1983). The Chesapeake Bay Program is a unique regional partnership between the states of Maryland, Pennsylvania, and Virginia; the District of Columbia; the Chesapeake Bay Commission; EPA; a tristate legislative body; and citizen advisory groups. It has led and directed the restoration of the Chesapeake Bay since 1983 (Chesapeake Bay Program 2005). However, the *Chesapeake Bay 2006 Health and Restoration Assessment* reports show that the bay's overall health remains degraded, despite significant advances in restoration efforts by program partners through newly focused programs, legislation, and funding (Chesapeake Bay Program 2007).

Since the signing of the *Chesapeake Bay Agreement of 1983,* program partners have adopted two additional bay agreements, the 1987 Chesapeake Bay Agreement and Chesapeake 2000 (Chesapeake Bay Program 2000). The main goals of Chesapeake 2000 are to continue efforts to achieve and maintain the 40 percent nutrient reduction goal agreed to in 1987; to adopt goals for tributaries south of the Potomac River; and to "correct the nutrient and sediment related problems in the Chesapeake Bay and its tidal tributaries sufficiently to remove the Bay and the tidal portions of its tributaries from the list of impaired waters under the CWA" (Chesapeake Bay Program 2000).

EPA has calculated that nitrogen and phosphorus discharges from all sources must be drastically reduced beyond current levels and that municipal WWTPs, in particular, will have to reduce nitrogen discharges by 72 percent (Chesapeake Bay Foundation 2002). On March 21, 2003, the program partners agreed to reduce nutrient pollution by more than twice as much as was accomplished since coordinated bay restoration efforts began nearly 20 years ago. The District of Columbia and states that border the bay have agreed to reduce the amount of nitrogen discharged from the current 275 million pounds to no more than

175 million pounds per year and to reduce the amount of phosphorus discharged from the current 18.8 million pounds to no more than 12.8 million pounds per year (Chesapeake Bay Program 2003). EPA Region 3 has established a nutrient criteria team to implement the National Nutrient Strategy issued by EPA for the Mid-Atlantic region (Chesapeake Bay Program 2003).

1.3 NPDES Permitting

The National Pollutant Discharge Elimination System (NPDES) aids in the attainment of water quality standards by regulating point sources that discharge into the surface waterbodies. Effluent limitations are the primary mechanism in NPDES permits for controlling discharges of pollutants to receiving waters. When developing effluent limitations for an NPDES permit, a permit writer must consider limits based on both the technology available to control the pollutants (i.e., technology-based effluent limits) and limits that are protective of the water quality standards of the receiving water (i.e., water quality-based effluent limits, or WQBELs). A permit writer may find that a discharge causes, has the reasonable potential to cause, or contributes to an exceedance of a water quality standard and that technology-based effluent limits are not adequate to ensure that water quality standards will be attained in the receiving water. In such cases, CWA section 303(b)(1)(c) and the NPDES regulations at 40 CFR 122.44(d) require that the permit writer develop more stringent WQBELs designed to ensure that water quality standards are attained. Developing appropriate effluent limits in NPDES permits is a vital component of the water quality standards-to-permits process.

For more information about the permitting process, see the following documents and Web sites:

EPA's National Pollutant Discharge Elimination System (NPDES) Web site at
http://www.epa.gov/npdes

EPA's *NPDES Permit Writers' Manual*
http://cfpub.epa.gov/npdes/writermanual.cfm?program_id=45

1.3.1 Watershed-based Permitting and Water Quality Trading

To assist in determining how various permitted discharges affect attainment of water quality criteria on a watershed basis, EPA has developed watershed-based permitting and water quality trading as innovative tools that use a watershed approach. EPA expects that watershed-based permitting and water quality trading will be useful tools for implementing WQBELs for nitrogen and phosphorus in NPDES permits.

Watershed-based permitting is an approach to NPDES permitting in which permits are designed to attain watershed goals that reflect consideration of all sources/stressors in a watershed or basin. These permits are developed through a watershed planning framework to communicate with stakeholders and to integrate permit development with monitoring, water quality standards, total maximum daily load (TMDL), nonpoint source, source water protection, and other programs. The ultimate goal of watershed-based permitting is to develop and issue NPDES permits that consider the entire watershed, not just an individual point source discharger.

Water quality trading is a voluntary market-based approach that, if used in certain watersheds, might achieve water quality standards more efficiently and at lower cost than traditional approaches. For a given pollutant, costs for controlling point source discharges compared to costs for controlling nonpoint source runoff often vary significantly in a watershed, creating the impetus for water quality trading. Through water quality trading, facilities that face higher pollutant control costs to meet their regulatory obligations can purchase pollutant reduction credits from other sources that can generate the reductions at a lower cost, thus achieving the same or better overall water quality improvement. Trading also can provide ancillary environmental benefits like flood control, riparian improvement, and habitat.

For more information on new tools developed by EPA that can assist with meeting water quality goals, see the following documents and Web sites:

Watershed-based Permitting Policy Statement and related information
http://www.epa.gov/npdes/pubs/watershed-permitting-policy.pdf

Watershed-Based National Pollutant Discharge Elimination System (NPDES) Permitting Implementation Guidance
http://www.epa.gov/npdes/pubs/watershedpermitting_finalguidance.pdf

Watershed-based National Pollutant Discharge Elimination System (NPDES) Permitting Technical Guidance http://www.epa.gov/npdes/pubs/watershed_techguidance_entire.pdf

Water quality based permitting and related information
http://cfpub.epa.gov/npdes/wqbasedpermitting/wspermitting.cfm

EPA's Water Quality Trading Web site http://www.epa.gov/owow/watershed/trading.htm

Trading policy statement http://www.epa.gov/owow/watershed/trading/tradingpolicy.html

Water Quality Trading Assessment Handbook
http://www.epa.gov/owow/watershed/trading/handbook

Water Quality Trading Toolkit for Permit Writers
http://www.epa.gov/waterqualitytrading/WQTToolkit.html

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CHAPTER 2: Treatment Technologies

2.1 Overview

This chapter presents information about available technologies that can be used to remove nitrogen and phosphorus from municipal wastewater. The technologies are presented along with performance data that show the variability of the effluent concentrations. The purpose of this chapter is to help permit writers develop appropriate discharge permit limits with a full understanding of available technologies, their variability, and their ability to meet the proposed limits in the most sustainable way. The same information is made available to municipal planners and engineers to assist in their preparation of compliance plans for new discharge permits.

The information in this document was obtained from many sources, including publications of the Water Environment Federation (WEF), Water Environment Research Foundation (WERF), and other organizations; reports from the states of Maryland and Connecticut; and data from 30 municipal treatment facilities that participated voluntarily in 2006 and 2007.

The full-scale performance data are expressed on the same statistical basis for easy comparison. The data are accompanied by a discussion of the key factors that contribute to the performance variability and reliability of various systems. These factors are wastewater characteristics; process parameters; environmental parameters such as temperature, chemistry, and biology; and operating parameters. It should be emphasized that the performance data for these facilities reflect differences in operating philosophy, permit limitations, temperature, influent conditions, flow conditions, and the relative plant load compared to design. Thus, the documented performance does not necessarily represent optimum operation of the technologies presented.

Following this chapter, Chapter 3 provides a detailed discussion about the case studies on nine facilities. Cost information is presented in Chapter 4, and design factors that should be considered when selecting a retrofit or a new upgrade process technology are discussed in Chapter 5.

2.2 Nitrogen Removal Processes

2.2.1 Nitrogen Species in Wastewater

For the purposes of this document, only biological nitrogen removal is considered, along with physical removal by filtration of solids containing incorporated nitrogen. Other physical-chemical processes, such as breakpoint chlorination, ion exchange, and air stripping, are not included because they are not usually feasible for municipalities because of technical, regulatory, and cost considerations.

The biological removal of nitrogen is carried out through a three-step process: (1) the conversion of ammonia from organic nitrogen by hydrolysis and microbial activities, called *ammonification*; (2) the aerobic conversion of ammonia to nitrate by reacting the ammonia with oxygen in a process called *nitrification*; and (3) the conversion of nitrate to nitrogen gas by reacting the nitrate with organic carbon under anoxic conditions in a process called *denitrification*. The nitrification process is accompanied by the destruction of alkalinity (e.g., bicarbonate, HCO_3^-, is neutralized to carbonic acid, H_2CO_3). Alkalinity is recovered as part of the denitrification process with the generation of hydroxide.

The chemical equations involved in the biological conversion of nitrogen are as follows:

1. Formation of ammonia from organic nitrogen by microorganisms (ammonification):

   $$\text{Organic nitrogen} \rightarrow NH_4^+$$

2. Nitrification to nitrite by *Nitrosomonas* species and other autotrophic bacteria genera:

   $$NH_4^+ + 3/2\ O_2 + 2HCO_3^- \rightarrow NO_2^- + 2H_2CO_3 + H_2O$$

3. Nitrification to nitrate by *Nitrobacter* species and other autotrophic bacteria genera:

   $$NO_2^- + \tfrac{1}{2}\ O_2 \rightarrow NO_3^-$$

4. Denitrification by denitrifying microorganisms with no oxygen present:

   $$NO_3^- + \text{organic carbon} \rightarrow N_2\ (g) + CO_2\ (g) + H_2O + OH^-$$

The stoichiometry for nitrification shows 4.57 grams of oxygen per gram of ammonia nitrogen and consumption of 7.14 grams of alkalinity as measured as calcium carbonate ($CaCO_3$) per gram of ammonia nitrogen. The stoichiometry for denitrification shows at least 2.86 grams of chemical oxygen demand (COD) required per gram of nitrate nitrogen or 1.91 grams of methanol per gram of nitrate nitrogen. This includes carbon incorporated into biomass production (Metcalfe & Eddy, 2003, p 621). Alkalinity is generated as 3.57 grams per gram of nitrate nitrogen denitrified (WEF and ASCE, 2006, pp. 39 and 71, respectively).

Nitrification operates on ammonia nitrogen and most of the total Kjeldahl nitrogen (TKN). Dissolved organic nitrogen (DON) compounds are typically complex nitrogen-containing molecules that constitute a portion of TKN but are difficult to break down. In most localities, the average DON concentration ranges from 0.5 to 2 milligrams per liter (mg/L) (Demirtas et al. 2008); however, higher DON concentrations can occur in some locations because of industrial sources or natural components in the background. High DON concentrations negatively affect the ability of a plant to achieve a low final total nitrogen (TN) level, even with installation of the processes described in this chapter.

A reasonable solution to address the presence of DON that is based on science and technology is needed. Pagilla (WERF 2007) characterized DON in wastewater and reported that it could be grouped by molecular weight (MW). The DON compound group with an MW of less than 1,000 Dalton (Da) includes urea, amino acids, DNA, peptides, and various synthetic compounds. The DON group with an MW ranging between 1,000 and 1,000,000 Da includes fulvic acids. High-MW DON includes humic substances. Biological wastewater treatment has been considered effective in removing low-MW DON, while high-MW DON is considered refractory to this kind of treatment. Sedlak and Pehlivanoglu-Mantas (2006) indicated that a portion of the recalcitrant DON could have a slow degradation rate, and thus the effect on the receiving stream water would be low. Pagilla (WERF 2007) proposed a short-term test protocol to assess the bioavailability of DON using algae and bacteria. More research is needed to determine the sources and fate of DON in wastewater treatment processes and the method by which bioavailability can be demonstrated.

2.2.2 Nitrogen Removal Factors

Carbon Source

Many have reported from their research the quantity of carbon, in proportion to the nitrogen present in the wastewater, necessary to reduce the amount of nitrogen. They have included the COD-to-TKN ratio, the readily biodegradable COD (rbCOD)-to-TKN ratio, and volatile fatty acids (VFAs). A biochemical oxygen demand (BOD)-to-TKN ratio of 4 or greater is sufficient for biological nitrogen removal to occur (Neethling et al. 2005; WEF and ASCE 1998; Lindeke et al. 2005). Normal domestic wastewater contains a sufficient COD-to-TKN ratio to remove 65 to 85 percent of the nitrogen in a single-pass process like the modified Ludzack-Ettinger (MLE) (Barnard 2006) or in attached-growth systems like biological aerated filters (BAFs) (Stephenson et al. 2004) and moving-bed biofilm reactors (MBBRs) (Rusten et al. 2002). Simultaneous nitrification and denitrification (SND) in oxidation ditches, such as the Orbal oxidation ditch system, has shown up to 90 percent nitrogen removal in the extended-aeration mode. When combined with a pre-anoxic zone, the oxidation ditch can produce an average TN concentration of less than 3.1 mg/L with an 85th percentile value of 4 mg/L (Barnard 2006). The 4-stage Bardenpho process using a carousel aeration basin produced an average TN concentration of 1.9 mg/L, where plant sludge was hauled away to another plant for processing (deBarbadillo et al. 2003). Processes such as the 4-stage Bardenpho can fully utilize influent carbon and meet most limits except for very low targets. This means that they will have cost savings via reduced energy use and no need for supplemental carbon, which results in no additional sludge production.

When it is determined that an additional carbon source is needed to achieve the desired level of nitrogen removal, there are two types of sources—in-plant and external. In-plant sources include primary effluent, which can be step-fed to the activated-sludge process, and fermentation of primary sludge to obtain VFAs and other readily used carbon compounds. Step-feeding reduces the needed tank footprint due to lower hydraulic and sludge detention times; reduces or eliminates the need for an additional carbon source; and, as an extra advantage, provides the wastewater treatment plant (WWTP) operators the ability to better handle wet-weather flows. Successful operation, however, requires oxygen control, an alkalinity supply, and good instrumentation incorporated into the system.

In New York, Chandran et al. (2004) were able to achieve 4 mg/L in a four-pass system with additional carbon in a limited anoxic zone volume. McGrath et al. (2005b) reported three-way split feed with anoxic, swing, and aeration zones that were 35 percent, 15 percent, and 50 percent of the activated sludge tank volume, respectively. During the last 12 months of the study, effluent TN ranged from 3 to 7.25 mg/L, with an annual average of 5 mg/L. Tang et al. (2004) developed a model to obtain the optimal step-feed pattern to use in four plants in the Los Angeles County Sanitation Districts. They found that for a system with two anoxic stages, feeding approximately two-thirds of the flow to the first stage and one-third of the flow to the second stage provided the best overall removal. As the number of anoxic stages increases, less is put into the first stage, but the first stage still receives 40 to 50 percent of the total flow. In general, use of more than four anoxic stages provides minimal benefits. Less primary effluent and return activated sludge (RAS) should be fed to the last stage because any ammonia or nitrate that is not treated in the last stage passes through the secondary clarifiers and increases the TN concentration in the overall discharge. If the BOD-to-TKN ratio is low, however, a higher percentage of the flow (compared to typical BOD-to-TKN ratios) should be fed to the later stages. Alternatively, equal distribution of flow among the splits, with no flow to the last section, has also been found to work well (Metcalf & Eddy 2003).

An in-plant alternative to step-feeding is fermentation. Fermenters are usually associated with plants that perform biological phosphorus removal. It is also noted, however, that when high nitrate concentrations are returned to the anaerobic zone, denitrification occurs, consuming some of the VFAs that the fermenter produced. Fermenters can therefore be used to provide a source of carbon for plants that perform denitrification. Stinson et al. (2002) reported on a pilot study performed by the New York City Department of Environmental Protection that compared denitrification rates achieved by feeding methanol, acetate, and fermentate. The results from the pilot plants operating in the step-feed activated-sludge mode indicate that denitrification rates for fermentate and acetate were similar at 0.15 to 0.30 mg of nitrate nitrogen per milligram of volatile suspended solids (VSS) per day (mg N/mg VSS-day), whereas denitrification rates using methanol were significantly lower at 0.04 to 0.08 mg N/mg VSS-day. Higher denitrification rates resulted in less carbon source addition and therefore less sludge production.

Methanol is most often used as an external carbon source because of its relatively low cost. Methanol, however, is corrosive and combustible and therefore requires special handling to meet Occupational Safety and Health Administration (OSHA) regulations and NFPA-820: *Standard for Fire Protection in Wastewater Treatment and Collection Facilities*. Some plants are considering alternatives to methanol because of these flammability and explosion concerns. Other chemicals, such as MicroC (distributed by Environmental Operating

Solutions, Inc.) can be used. The company that manufactures MicroC recently introduced MicroCG, which replaces the 5 to 6 percent of methanol in MicroC with glycerin.

Communities with a nearby supplemental carbon source, such as molasses or brewery waste, can consider the opportunity to use such sources. The advantage of using an external carbon source is that the operation is independent of the nature and volume of the influent wastewater, giving the operators much more flexibility. The disadvantage is that the additional carbon source will increase capital costs due to the construction of chemical storage, feed pumps, and piping and will require substantial operating costs for purchasing the chemicals and handling the extra sludge generated because of the addition. The carbon source is typically added to the separate-stage anoxic zone or denitrification filters. A study by deBarbadillo et al. (2005) presented external carbon source doses for denitrification filters. Typical methanol doses are between 2.5 and 3 times the amount of nitrate nitrogen to be removed on a mass basis (WEF and ASCE 2006, p. 75).

Number of Anoxic Zones

A single anoxic basin with an internal recycle stream can achieve reasonable rates of TN removal in the range of 6 to 8 mg/L. An example of such a process is the MLE process (described in more detail later). The internal recycle returns nitrates produced by nitrification in the aeration basin to the anoxic zone for denitrification. With the anoxic zone at the beginning of the process, carbon source addition is not usually necessary because domestic wastewater typically provides enough carbon to achieve 65 to 85 percent removal (Barnard 2006).

In general, the denitrification rate increases with increasing internal recirculation, up to a maximum of 500 percent (WEF and ASCE 2006). Denitrification in wastewaters with BOD-to-TKN ratios less than 4-to-1 or COD-to-TKN ratios less than 10-to-1 typically is not benefited by high internal recirculation. This is because such wastewater has insufficient carbon to support an elevated denitrification rate.

Having two anoxic zones allows lower TN effluent concentrations to be achieved because more of the nitrates produced after nitrification in the aeration basin can be treated by an internal recycle to the first anoxic zone or by flowing through the second anoxic zone. Aeration is usually recommended after the second anoxic zone to both strip nitrogen gas formed in the anoxic zone and to decrease the possibility of denitrification in the secondary clarifiers. This has the effect of reducing nitrogen gas release in the secondary clarifier, which could result in rising sludge. In addition, maintaining aerobic conditions prevents phosphorus release in the secondary clarifiers. Adding a carbon source to the second anoxic zone can further increase denitrification by ensuring that sufficient carbon is available for the process to occur. Barnard (2006) has reported that processes involving two anoxic zones, such as the 4- or 5-stage Bardenpho process, can achieve TN concentrations between 2.5 and 3.5 mg/L. Clearwater, Florida, operates the Marshall Street and Northeast Advanced

Pollution Control Facilities. Both plants are achieving annual average TN levels below 3 mg/L—2.32 and 2.04 mg/L, respectively—without a supplementary carbon source.

Temperature

Temperature affects the rate of both nitrification and denitrification. At lower temperatures, the nitrification and denitrification rates decrease, leading to poorer performance in the winter if operational changes are not made to compensate for the decreased kinetic rates. Nitrification can occur in wastewater temperatures of 4 to 35 degrees Celsius (^{o}C). Typical wastewater temperatures range between 10 and 25 °C (WEF and ASCE 2006, p. 41). The nitrification rate doubles for every 8 to 10 °C rise in temperature, meaning that in areas that experience a wide range of temperatures between winter and summer, nitrification rates could differ by a factor of 4 over the course of a year (WEF and ASCE 2006, p. 42). Denitrification is also subject to temperature, although to a lesser extent than nitrification. On the basis of a wastewater temperature range of 10 to 25 °C, the denitrification rate would be expected to vary by a factor of only 1.5 (WEF and ASCE 2006, p. 73). Alternative carbon sources should be explored to determine if an additional carbon supply could provide better denitrification performance in cold weather than others. Alternatively, external carbon might not be needed at all during warm weather conditions, because the process might be able to meet the treatment objectives with the available carbon in the wastewater.

Alkalinity

Alkalinity is consumed as part of the nitrification process because hydrogen ions are created when ammonia nitrogen is converted to nitrate nitrogen. Denitrification restores a portion of the alkalinity during the conversion of nitrate nitrogen to nitrogen gas. The nitrification process consumes 7.14 grams of alkalinity as calcium carbonate ($CaCO_3$) per gram of ammonia nitrogen removed (WEF and ASCE 2006, p. 39). Denitrification produces 3.57 grams of alkalinity as $CaCO_3$ per gram of nitrate nitrogen removed (WEF and ASCE 2006, p. 71). Therefore, to convert 1 gram of ammonia nitrogen to 1 gram of nitrogen gas, approximately 3.6 grams of alkalinity as $CaCO_3$ is consumed. Nitrification can generally occur at pH values between 6.5 and 8.0 standard units (s.u.). The recommended minimum alkalinity in the secondary effluent is 50 mg/L as $CaCO_3$, although as much as 100 mg/L is suggested by standard design manuals (WEF and ASCE 2006, p. 43). If the alkalinity will be below the recommended levels, chemical addition might be necessary. Sodium hydroxide or lime is added at some plants to maintain acceptable alkalinity and pH levels.

Solids Retention Time

The solids retention time (SRT) must be long enough to maintain nitrification. The microorganisms responsible for nitrification have a much slower growth rate than other heterotrophic bacteria. Therefore, doubling of the nitrification microorganism population requires 10 to 20 times longer than for other heterotrophic bacteria (WEF and ASCE 2006, p. 40). Maintaining longer SRTs can also reduce the amount of energy required for mixing if tank sizes and liquid volumes can be reduced. In addition, energy can be saved if denitrification can be achieved in an SND application (Barnard et al. 2004; Kang et al. 2006). Some plants with mechanical aerators and/or concentric ditches operating with the same SRTs have a lower aerator energy-to-volume ratio. The lower energy requirement is because of SND, which results in reduced oxygen needs. For example, Barnard (2004) reported that the aerator energy density was decreased by more than half when the SRT was increased from 8 days to 24 days. Concurrently, the SND rate increased from 30 percent at an 8-day SRT to 70 percent at a 24-day SRT. The optimum SRT for a given WWTP depends on several factors, including wastewater temperature, dissolved oxygen (DO) concentration in the aerobic zone, pH, alkalinity, inhibition from chemicals, and variations in hydraulic flow and organic load. In general, denitrifying microorganisms are relatively slow growing and can be subject to washout at high flows in the bioreactor or clarifier. Therefore, secondary clarifier design that assures maximum retention of the slower growing microorganisms can be critical to maintaining reliable nitrogen removal performance.

Hydraulic Retention Time

Hydraulic retention time (HRT) affects both nitrification and denitrification. The aerobic zone(s) of single nitrification/denitrification processes must be large enough to allow most of the carbonaceous BOD (CBOD) to be consumed before nitrification can begin. The size of the anoxic zone(s) must be sufficient to allow denitrification to occur without consuming the entire carbon source that might be needed for biological phosphorus removal. Anoxic zones are typically 35 to 50 percent of the secondary treatment process by volume. Because of the uncertainty in the required volume, some plants are designed with swing zones that can be operated in an anoxic or aerobic mode. Generally, swing zones can be operated as aerobic during the summer and anoxic during the winter, when low temperatures lower the denitrification rates. However, sufficient aerobic detention time must be maintained for adequate nitrification. To test whether the HRT is limiting, two samples can be collected at the same time from the end of the anoxic zone. One sample can be filtered and analyzed for nitrates immediately, while the second sample can be filtered and analyzed for nitrates after 30 minutes. If the second sample has a lower nitrate concentration than the first, the HRT limits denitrification. If there is little difference between the two samples, it is more likely that lack of an adequate carbon source is limiting the denitrification (Tang et al. 2004).

Dissolved Oxygen

Nitrification requires the presence of sufficient DO, and the rate of nitrification can be limited when the DO concentration is too low to support a sufficiently high oxygen transfer rate. In municipal activated sludge systems with HRTs of 6 to 8 hours, the nitrification rate is maximized when the DO concentration is 2 mg/L or greater (WEF and ASCE 2006, p 42). Fixed-film systems, such as the MBBR or the integrated fixed-film activated sludge (IFAS) system, might need higher DO concentrations to prevent the biomass attached to the media from becoming anaerobic, which could lead to poor system performance.

The presence of DO inhibits some desirable biological processes, particularly denitrification. DO concentrations of 0.2 mg/L or greater decrease the denitrification rate in the anoxic zone (WEF and ASCE 2006, p 183). This is because the energy expended breaking NO_3 to obtain oxygen is greater than that from using molecular oxygen, so most microorganisms preferentially use DO when it is available rather than NO_3. Denitrification is made most efficient when DO is kept low throughout the anoxic zone. Thus, low DO levels in the internal recycle or RAS lines can effectively reduce the required HRT of the anoxic zone. It is recommended that DO concentrations in return flows be limited to about 1 mg/L (WEF and ASCE 2006, p.182).

Additional Design Considerations to Enhance Nitrogen Removal

Additional design considerations that can help make plant operation easier include accounting for return flow and loads as well as external loads in the basis of the design, incorporating a supervisory control and data acquisition (SCADA) system and other online monitoring for process control, and including allowances for flexibility in the design. If the return flow and loadings from side-streams such as filter backwash or sludge handling are included in the basis of the design, the plant will be able to treat the loadings that the processes receive, rather than requiring a portion of the standard design safety factor to account for these return streams. In addition, if the plant receives septage, landfill leachate, or similar hauled or direct waste streams, those loads should be accounted for in the basis of design. Accounting for all these loadings might lead to larger tank sizes, blowers, or chemical storage facilities.

Online monitoring allows the recording of real-time process information that can be used to make operating decisions. SCADA allows automatic control of the process in conjunction with online monitoring. The SCADA system can help in process optimization, and it can result in operational savings in situations where too much air is being added by automatically adjusting the amount of air delivered to the process.

Operational flexibility is gained by installing swing zones in anticipation of uncertainties in the future wastewater characteristics and operating conditions. The swing zone should be equipped with both mixers and aerators, which can be operated as either an anoxic or aerobic zone, depending on conditions at the plant (which might vary seasonally as well as diurnally between the day and night hours). The swing zone is known as a performance enhancer as well as an electricity saver when mixers are used instead of aerators during low-flow periods. This flexibility in design increases costs but can contribute to the long-term compliance and sustainability of biological nutrient removal as the wastewater and flow characteristics at the facility will change in the future.

2.2.3 Nitrogen Removal Technologies

Nitrification/Denitrification

Some facilities are required to remove only ammonia-N or TKN, with no current requirements to remove nitrate or nitrite. In such instances, the biological conversion of ammonia (or TKN) to nitrate is readily accomplished by increasing the SRT in the biological system, increasing the HRT, and ensuring that there is sufficient oxygenation to accomplish the conversion. System designs in the past incorporated a *two-sludge* strategy: first, BOD was removed in an activated-sludge reactor with a clarifier, and then a second activated-sludge system was run in series to accomplish nitrification. Current practice is to do both BOD removal and nitrification in a single sludge system, especially in retrofit situations, thereby saving land requirements by avoiding a new set of clarifiers and making the operation simpler. Significant energy savings is also anticipated in a single sludge system due to a reduced volume requiring aeration. McClintock et al. (1988, 1992) and Randall et al. (1992) report a reduction of 20 percent in volume requiring aeration, with a 40 percent or greater reduction in sludge production in single sludge systems compared to separate systems.

All the technologies described below except the denitrifying filters can be used to accomplish nitrification; if nitrate removal is not required, that portion need not be implemented. This could result in savings of capital or operation and maintenance (O&M) costs. During the planning and design of a nitrification retrofit, it would be logical to leave space for future denitrification in case future permits require it. In addition, there are advantages to accomplishing some denitrification, if it is supported by influent BOD, because doing so results in reduced overall sludge generation.

Denitrification Filters

Denitrification filters are usually placed after the secondary treatment process. One of their advantages is that in addition to providing nitrogen removal, they act as an effluent filter. Denitrification filters have a more compact area compared to other add-on denitrification processes. The filters can be operated in downflow or upflow configurations. As the process is performing denitrification after most of the BOD has been removed from the wastewater, a carbon source, such as methanol, must be supplied.

There are two general types of denitrifying filter. One is an adaptation of conventional deep-bed filters, as provided by Leopold Co. and Severn-Trent. These employ sand, gravel, anthracite, or other filter media in some combination, at a depth of 8 to 12 feet (WEF and ASCE 2006). They are typically operated in a downflow mode (with water directed down from the top of the bed); this means that nitrogen gas generated within the bed could be trapped between the grains. Thus, the beds require periodic *bumping*, whereby the flow of water or air is directed upward in the bed. This releases the trapped nitrogen gas. Less frequently, a full backwash is performed to remove accumulated solids. The backwash is usually returned to the head of the plant or to the secondary treatment process. Existing sand filters can be retrofitted to become denitrification filters if there is sufficient space to increase the bed depth. Conventional denitrification filters are typically loaded at 2 to 3 gallons/min/ft^2, with removal to 1 to 2 mg/L nitrate nitrogen.

The second general type of denitrifying filter is exemplified by the Biofor system from Infilco/Degremont and the Biostyr system from Kruger/Veolia. They are operated in an upflow configuration and use plastic media for microbial attachment. These types of filters are often paired with biological activated filters (BAFs), which accomplish BOD removal and nitrification. Because they are operated in upflow mode, pumping is required, which increases operating costs. The plastic media can be hydraulically loaded at 8 to 9 gallons/min/ft^2, with removal to 1 to 1.5 mg/L nitrate nitrogen. However, the plastic media are not as efficient at capturing solids as the conventional granular media. A biological upflow filter is illustrated in Figure 2-1.

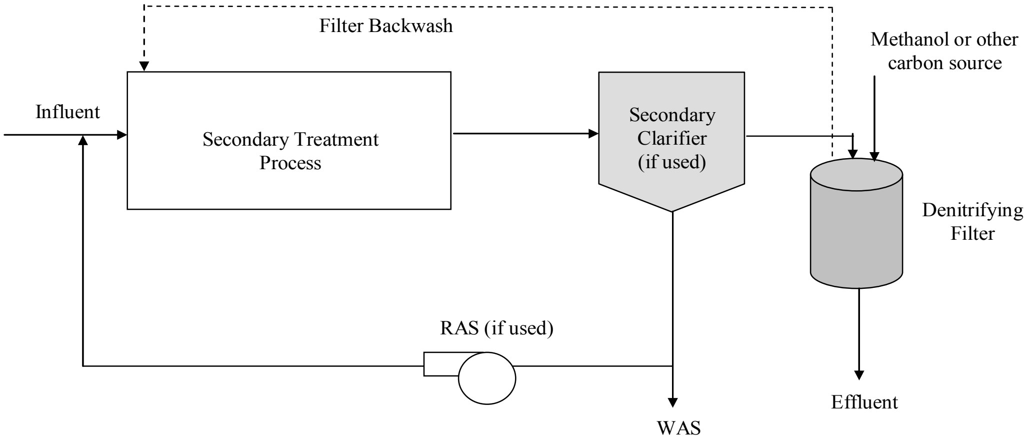


Figure 2-1. Denitrifying filter process.

The advantage of both types of denitrifying filters is that they can accomplish complete nitrate removal with a very small footprint in a building, and possibly no additional footprint if an older conventional filter can be retrofitted. A disadvantage of this system is that an additional carbon source (typically methanol) is usually required, with associated increased sludge generation. In certain cases, additional pumping and electrical costs might be required. The denitrifying filter process is illustrated in Figure 2-2.

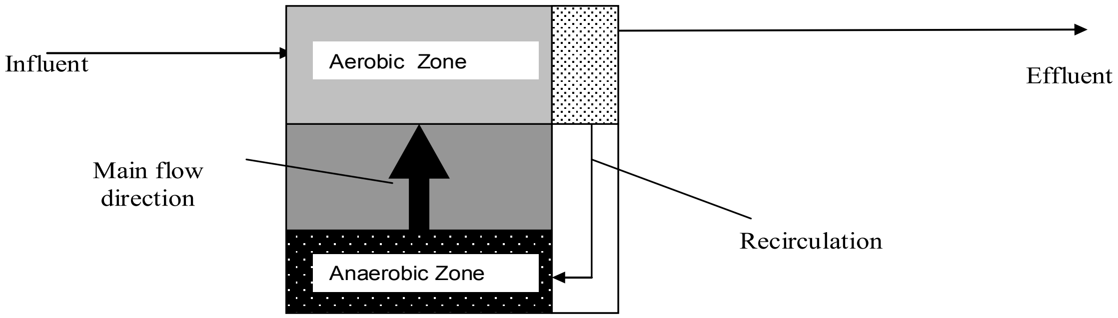


Figure 2-2. Biological upflow filter.

Modified Ludzack-Ettinger Process

The MLE consists of an anoxic basin upstream of an aerobic zone. An internal recycle carries nitrates created during the nitrification process in the aerobic zone along with mixed liquor to the anoxic zone for denitrification. RAS is mixed with the influent to the anoxic zone. The extent of denitrification is tied to the mixed liquor recycle flow; higher recycle rates increase denitrification. Because only recycled nitrate has the opportunity to be denitrified, the MLE alone cannot achieve extremely low final nitrogen concentrations. The maximum denitrification potential is approximately 82 percent at a 500 percent recycle rate (WEF and ASCE 2006). TN effluent concentrations typically range from 5 to 8 mg/L (Barnard 2006). Actual denitrification might be limited by other factors, such as carbon source availability, process kinetics, and anoxic or aerobic zone sizes. Furthermore, oxygen recycled from the aerobic zone can negatively affect the denitrification rate in the anoxic zone (WEF and ASCE 2006). Performance factors include limitations due to the single anoxic zone and the internal recycle rate that returns nitrates to the anoxic zone. Selection factors include the possibility of constructing walls in existing basins to create an anoxic zone; additional pumping, piping, and electricity to accommodate the internal recycle; and the possible need for an additional carbon source to promote denitrification. The modified Ludzack-Ettinger process is illustrated in Figure 2-3.

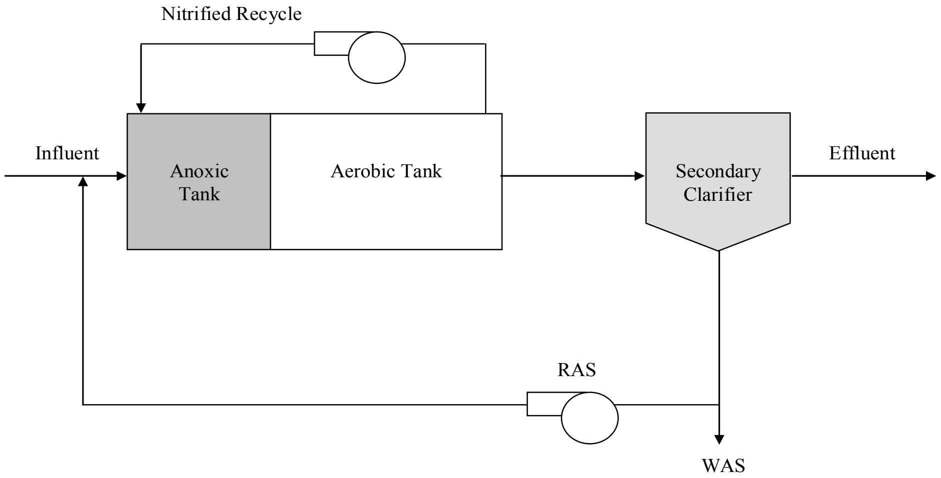


Figure 2-3. Modified Ludzack-Ettinger process.

Cyclically Aerated Activated Sludge

In a cyclically aerated activated-sludge system, the aeration system is programmed to turn off periodically, allowing denitrification and nitrification to occur in the same tank. This conversion is thus easy to achieve, with little or no capital expenditure required. The cyclically aerated activated-sludge system can be used to retrofit existing plants if sufficient SRTs can be maintained to allow nitrification to occur. The length of the cycle time depends on the loading rate and the target limit, with the HRT being 2 to 4 times the cycle time (Ip et al. 1987). If the aerobic SRT time is sufficient to achieve nitrification, the cyclic process can reduce TN in the effluent. Aeration can be provided by diffusers or surface aerators. Sequencing batch reactors (SBRs) and oxidation ditches can be designed to operate as cyclically aerated activated-sludge systems (WEF and ASCE 1998). The cyclically aerated activated-sludge process is illustrated in Figure 2-4. The range of TN effluent concentrations found at the case study facilities for this study was 3.1 mg/L to 10.4 mg/L (see Table 2-1 in Section 2.5).

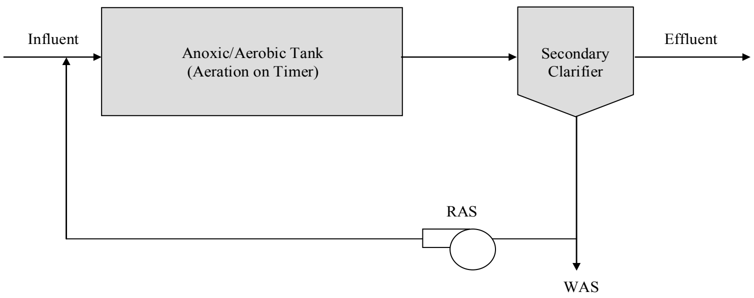


Figure 2-4. Cyclically aerated activated-sludge process.

Four-Stage Bardenpho Process

The 4-stage Bardenpho process involves an anoxic zone, followed by an aerobic zone (with an internal recycle to the first anoxic zone), which is followed by a second anoxic zone and a small aerobic zone. The first two tanks are similar to the MLE process. With pumps and zone sizes typically set large enough to accommodate a 400 percent internal recycle rate, the first anoxic zone accomplishes the bulk of the denitrification. The second anoxic zone removes nitrates from the first aerobic zone that are not recycled to the first anoxic zone. A carbon source, such as methanol, might need to be added to the second anoxic zone to achieve good denitrification. The second aerobic zone removes the nitrogen gas from the wastewater before the wastewater enters the secondary clarifiers. By aerating, the possibility of denitrification in the clarifier is removed. The sludge tends to settle better, and overall operation of the secondary clarifier is improved (WEF and ASCE 2006).

The Bardenpho process has been used in numerous underlying configurations, including plug flow, complete mix, and oxidation ditch reactors; some configurations have used existing oxidation ditches for the first two basins and additional constructed tanks for the secondary basins (WEF and ASCE 2006). Selection factors include a large process footprint involving several large basins; retrofit of existing basins is possible but unlikely. Additional piping, pumping, and electricity are needed for the internal recycle streams. Although the available carbon source might be adequate, it is likely that the second anoxic zone will require supplemental carbon, with the associated generation of additional sludge and increased O&M costs. Performance factors include the presence of two anoxic zones and the internal recycle rate, which runs as high as five times of the influent flow rate. The 4-stage Bardenpho process is illustrated in Figure 2-5. The range of TN effluent concentrations found in the literature evaluated for this study was 3.5 mg/L to 12.1 mg/L (see Table 2-1 in Section 2.5).

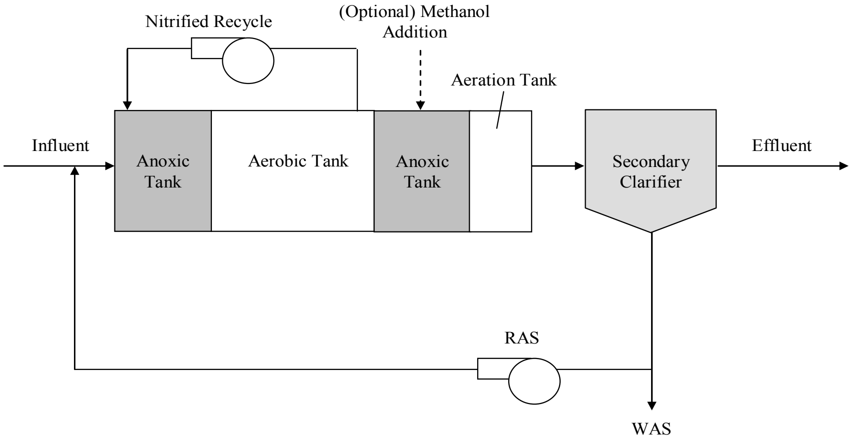


Figure 2-5. Four-stage Bardenpho process.

Oxidation Ditch Processes

Oxidation ditch processes use channels in a loop to provide continuous circulation of wastewater and activated sludge. Surface or submerged aerators provide aeration, and anoxic zones are made possible by judicious placement of the aerators. As mentioned earlier, oxidation ditches can also be operated as cyclically aerated activated-sludge systems. As an alternative to internal anoxic zones, anoxic basins can be constructed before or after an existing ditch. If there are aerobic zones downstream of anoxic zones, provision should be made for an internal recycle or an external anoxic basin to allow denitrification of any additional nitrates. Selection factors include a potentially large footprint. Compared to an existing system that is not denitrifying, electricity usage could be reduced because some aerators would be turned off to create anoxic zones. With ditches, no additional piping and pumping are needed unless external basins are used. Such basins might be needed to achieve very low nitrogen levels. In such cases, an additional carbon source would likely be needed for the denitrification reactor. Performance can be increased by using automatic DO controls that are capable of turning blowers on or off as necessary to maintain the desired set points. The oxidation ditch process is illustrated in Figure 2-6. The range of TN effluent concentrations for processes evaluated for this study were 6 to 10 mg/L as TN. Sen et al. (1990) reported less than 4 mg/L TN after optimizing aeration.

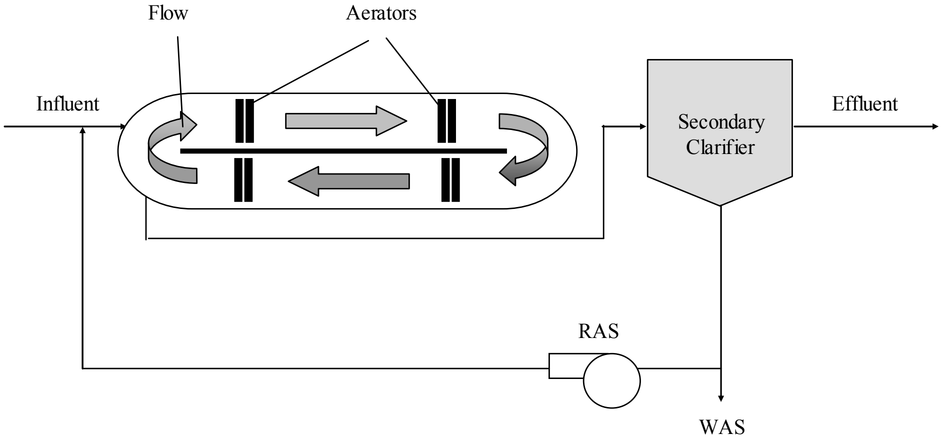


Figure 2-6. Oxidation ditch process.

Fixed Film Processes

Fixed-film, or attached growth systems are possible alternatives to suspended growth systems for nutrient removal. The medium can either be in a packed configuration, such as in a trickling filter, or suspended, such as in an IFAS or MBBR system. Fixed films provide an advantage for slow-growing bacteria, such as those involved in nitrification and denitrification processes, because the attachment gives those organisms longer residence times in the reactor. Fixed-film systems are also less prone to washout or toxic upsets.

Trickling filters can be used as an *add-on* process to provide nitrification. Additional information on trickling filters can be found in *Wastewater Engineering: Treatment and Reuse* (Metcalf & Eddy 2003). Use of a fixed-film carrier media allows the growth of additional biomass compared to a similarly sized activated sludge system, which can result in increased nitrification and denitrification in the given volume. Carrier media can be retrofitted into existing suspended-growth systems, provided that sufficient volume is available in the reactors. Separate aerobic and anoxic zones could be required if both nitrification and denitrification are desired.

Integrated Fixed-Film Activated Sludge

IFAS systems are hybrids that have attached growth media included in an activated-sludge basin. The media for attached growth can be a fixed type or a floating type. Fixed media include rope or some other stranded material (example brand names include Ringlace® and Bioweb®). Floating media can include sponges (Captor®, Linpor®), packing material, such as the saddles that would be used in packed towers for air stripping or gas absorption; or plastic media similar to those used in an MBBR (Kaldnes®, Hydroxyl®, or Bioprotz®) (Copithorn 2007; Welander and Johnson 2007). Floating media can be free-floating or retained in the basin by enclosing them in cages or installing screens.

IFAS systems usually have higher treatment rates and generate sludges with better settling characteristics and lower mass than activated-sludge systems. Because the media can be held in zones within the overall basin, there is the possibility of having nitrification and denitrification occur in the same basin, in separate anoxic and aerobic zones. To obtain very low nitrogen concentrations in the effluent, two distinct anoxic zones should be provided so that one can be used for final polishing. Alternatively, separate zones can be eliminated with tight control of DO. Selection factors include a medium footprint using the existing activated-sludge system, with construction within the aeration basin to install the selected method of media retention. No additional piping and pumping are typically needed, as long as the flow pattern is set to obtain sequential aerobic/anoxic conditions. If no additional pumping is needed, electrical costs are likely to be reduced for zones no longer being aerated. If a carbon source needs to be fed to secondary anoxic zones to maintain denitrifying activity, some additional sludge will be generated, and O&M costs will increase.

The IFAS system reduces the footprint significantly by providing additional surface area for attached growth to occur within the same basin area as a comparable activated-sludge process. As such, a retrofit IFAS is frequently an alternative to adding tank capacity to an activated-sludge system (Johnson et al. 2005). A fine screen (3 to 6 mm) is recommended upstream of the secondary process to prevent material like hair from interfering with the surface area of the medium. To promote nitrification and denitrification, a higher DO concentration might be required in the aerobic zone when compared to an extended-aeration process. The extra oxygen could be required to penetrate the biomass growth on the medium so that it does not become anaerobic. Aeration is often done by diffused air (fine or coarse bubble), as mechanical aeration could interfere with the fixed-film media (Johnson et al. 2005). Higher RAS rates tend to promote better TN removal. The IFAS process is illustrated in Figure 2-7. The range of TN effluent concentrations at plants with IFAS technologies in the literature evaluated as part of this study was 2.8 mg/L to 17.0 mg/L (see Table 2-1 in Section 2.5).

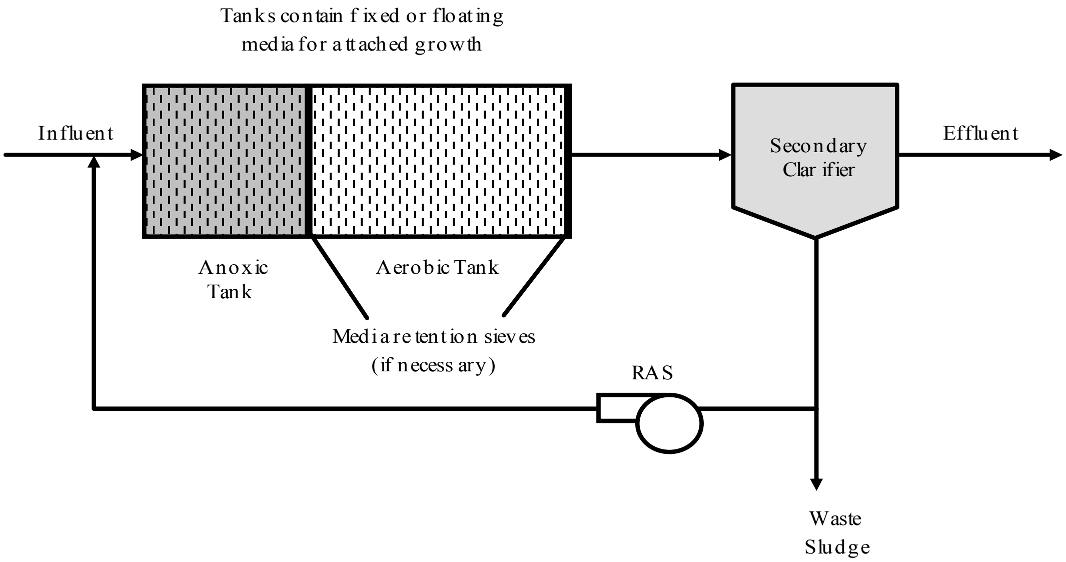


Figure 2-7. Integrated fixed-film activated sludge process.

Moving-Bed Biofilm Reactor

An MBBR consists of small plastic media (carrier elements) in an anoxic or aerobic zone that allow attached growth to occur. The plastic is typically polyethylene with a specific gravity slightly less than 1.0. The carrier elements from most manufacturers are shaped like cylinders or wheels with internal and external fins. These shapes provide a high surface area per unit volume that is protected from shear forces, allowing better biofilm growth. The MBBR process can be retrofitted into an existing activated-sludge basin. This process combines the technologies of activated sludge and biofilm processes and is frequently used for upgrading an existing plant, especially when space is limited. Such high-rate biofilm processes are highly efficient in removing organic and nitrogen loads.

MBBRs can be used within separate aerobic and anoxic zones. Slow-speed, submersible mixers are used in anoxic zones; air is supplied in aerobic zones by coarse bubble diffusers because fine bubbles tend to coalesce in the plastic media. A sieve is used to retain the media in the designated basin. MBBR technology does not involve any return flows and does not rely on suspended growth to provide additional treatment. Selection factors include a medium footprint (however, including a recycle stream would increase the footprint), with construction of only screens for retaining the carrier elements needed in the basins. Depending on how the system is being operated and the target limit, there might or might not be a need for an additional carbon source or post-reactor treatment; if no additional carbon is needed, no additional sludge will be generated, reducing O&M costs. The MBBR process is illustrated in Figure 2-8. The range of TN effluent concentrations for MBBR found in the literature evaluated as part of this study was 2.8 mg/L to17 mg/L (see Table 2-1 in Section 2.5).

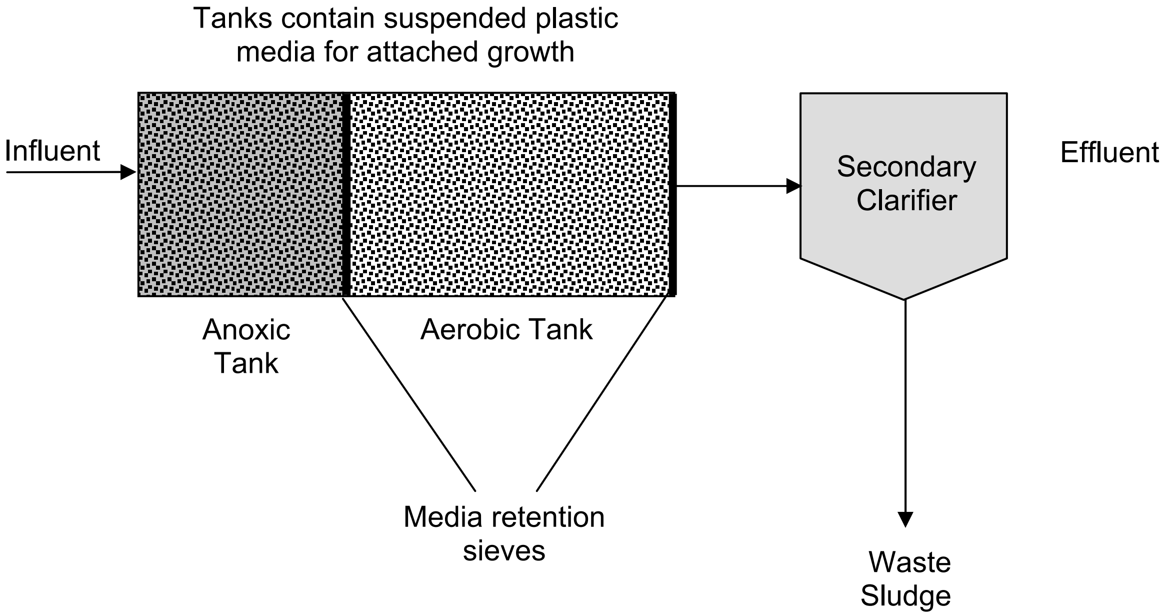
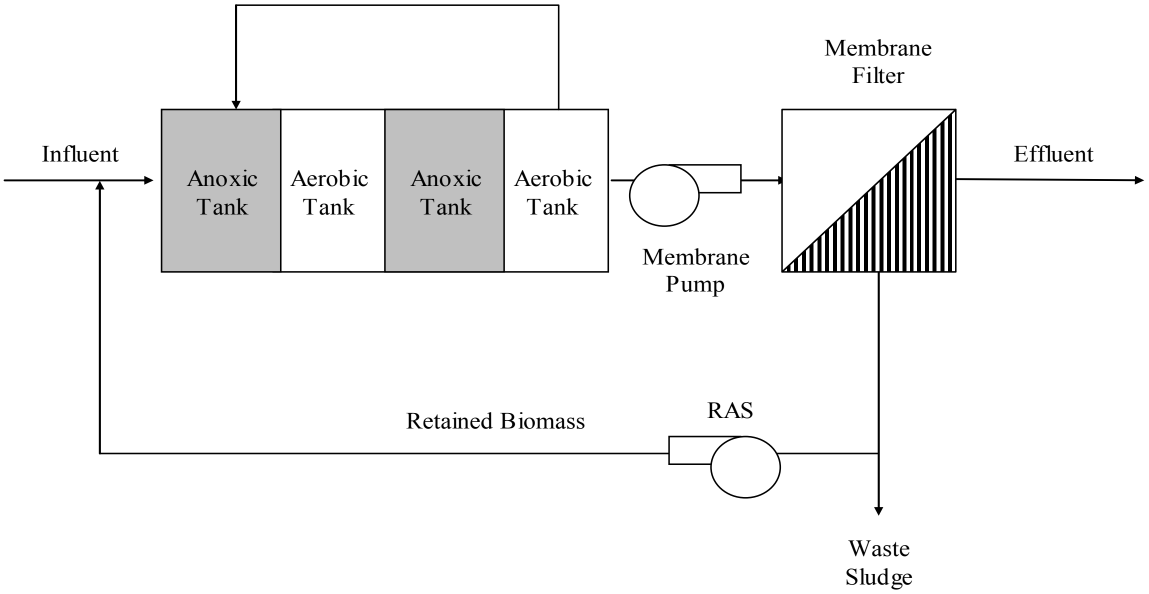


Figure 2-8. Moving-bed biofilm reactor process.

Membrane Bioreactor

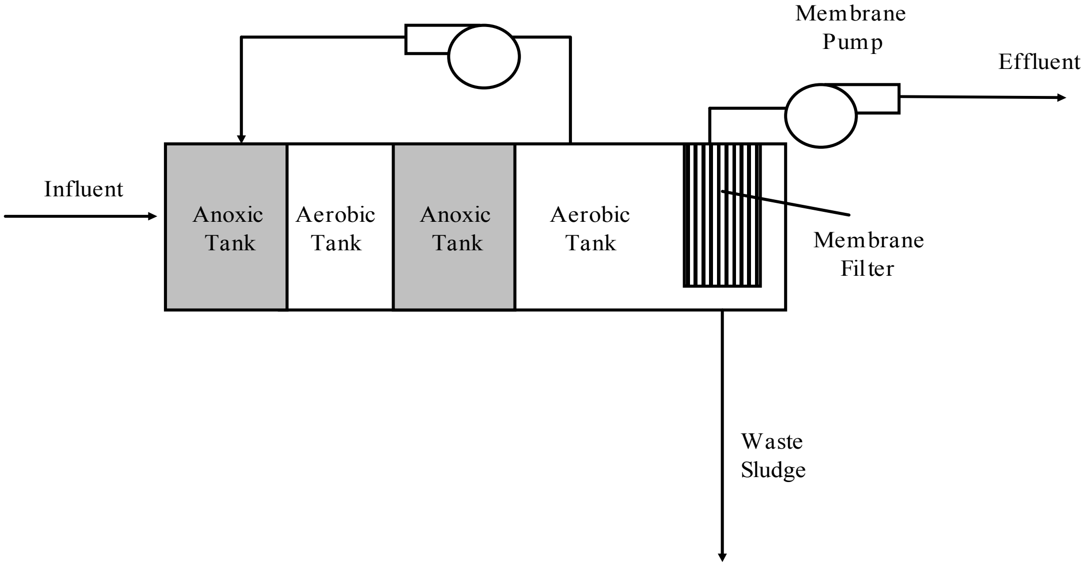
The membrane bioreactor (MBR) consists of anoxic and aerobic zones followed by a membrane that filters the solids from the mixed liquor, taking the place of secondary clarifiers. The membranes can be immersed in the final activated-sludge basin, or they can be set up in a separate vessel. The membranes can function in an inside-out mode, where only clean water exits the membranes, or in an outside-in manner, where only clean wastewater can enter the membranes. Usually wastewater must be pumped through the membranes, but in certain circumstances gravity feed can be used. By removing the need for settling, MBRs can operate at higher mixed liquor suspended solids (MLSS) concentrations (8,000 to 18,000 mg/L) than a comparable activated-sludge system (2,000 to 5,000 mg/L). The membranes are typically configured as hollow fiber tubes or flat plates. The membranes can either be immersed in the final tank, or they can be operated as a separate, stand-alone unit. If they are operated as a separate unit, an internal recycle returns a portion of the solids retained by the membranes to the anoxic zone.

Biofouling of the membranes is typically the most significant operational issue. Some manufacturers schedule a relax period or a back pulse to reduce the growth. The membranes must be cleaned periodically (quarterly to semiannually) using citric acid or sodium hypochlorite, depending on the type of fouling and the system manufacturer's recommendations. In addition, reducing the sludge age tends to reduce the formation of extracellular polymers, which tend to be the major cause of fouling. MBR system selection factors include a smaller footprint than conventional activated-sludge systems, with construction in existing basins or clarifiers. Lower effluent nitrogen is obtained with second anoxic zones, potentially accompanied by carbon source addition. As mentioned earlier, additional pumping is likely to be required to get water through the membrane, thus requiring additional electricity. The additional chemicals result in more sludge formation, but MBR systems overall provide less sludge formation than conventional systems because of their higher SRTs and higher sludge concentrations. New facilities are being designed and built to meet limits of 3 mg/L TN. The MBR process is illustrated in Figure 2-9.



(a)

Figure 2-9. MBR process. (a) External filter in lieu of clarifier



(b)

Figure 2-9. MBR process. (b) In-tank filter

Step-Feed Activated Sludge

The difficulty with carrying out full nitrification/denitrification with a sequence of aerobic and anoxic reactors is that little carbon is available to maintain microbial populations in the secondary reactors. As described earlier, this problem can be overcome by feeding supplemental carbon. Alternatively, feed can be provided directly to each anoxic zone to ensure that sufficient carbon is available. This *step-feed* strategy has the additional advantage of providing a means for the plant to better handle wet-weather events by reducing the solids loading to the clarifiers. The biomass inventory is maintained preferentially more in the first section and then less in the second section and the succeeding sections in proportion to the flow split going into each section. The last section maintains the least amount of biomass in inverse proportion to the flow and thus the MLSS concentration is the lowest. The solids loading rate to the clarifiers thus will be the lowest for the flow. The microorganisms in the later stages treat not just the fresh feed but also anything coming from upstream zones. The selection factors are a relatively large footprint for the basins, with in-basin retrofit of additional piping and possibly pumping. Extra head could be required, depending on the hydraulic configuration of the system. If additional pumping is required, electrical costs will be increased. However, no additional chemicals should be required because the food needs of the secondary microorganisms are accounted for. The step-feed activated sludge process is illustrated in Figure 2-10. The range of TN effluent found in the literature evaluated as part of this study was 1.0 mg/L to 14.0 mg/L (see Table 2-1 in Section 2.5).

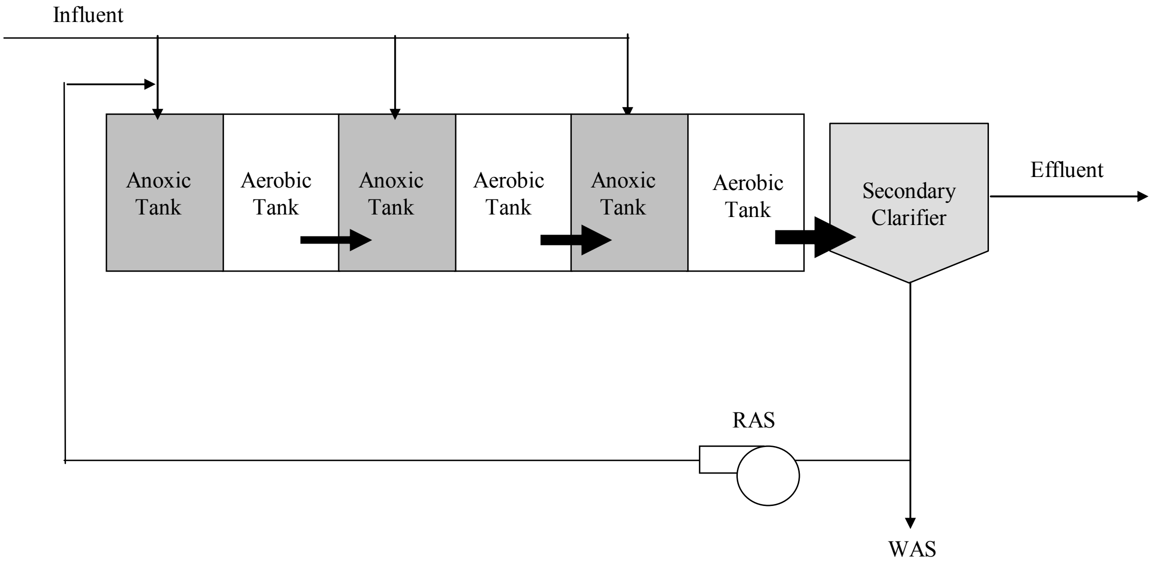


Figure 2-10. Step-feed activated sludge process.

Biodenitro Process

The Biodenitro process, a variation of the oxidation ditch, consists of two oxidation ditches side by side. Influent is fed alternately to the ditches, allowing anoxic and aerobic zones to form for nitrification and denitrification. The ditches are alternately aerated and not aerated, and mixing is maintained by the flow in the ditches. The ditches periodically switch modes, and the overall result is that the water passes through multiple aerobic and anoxic zones before discharge. Because oxidation ditches require large footprints, having multiple ditches requires a great deal of land. Some pumping might be required, depending on the hydraulic profile. Because the influent is continually switched, the need for additional food is typically reduced or eliminated. To accomplish low effluent nitrogen concentrations, an external anoxic zone following the oxidation ditches might be needed. The Biodenitro process is illustrated in Figure 2-11. The range of TN effluent concentrations for a phased isolation ditch (PID) found at the case study facilities for this study was 1.8 mg/L to 7.0 mg/L and in the literature evaluated as part of this study was 1.6 mg/L to 5.4 mg/L (see Table 2-1 in Section 2.5).

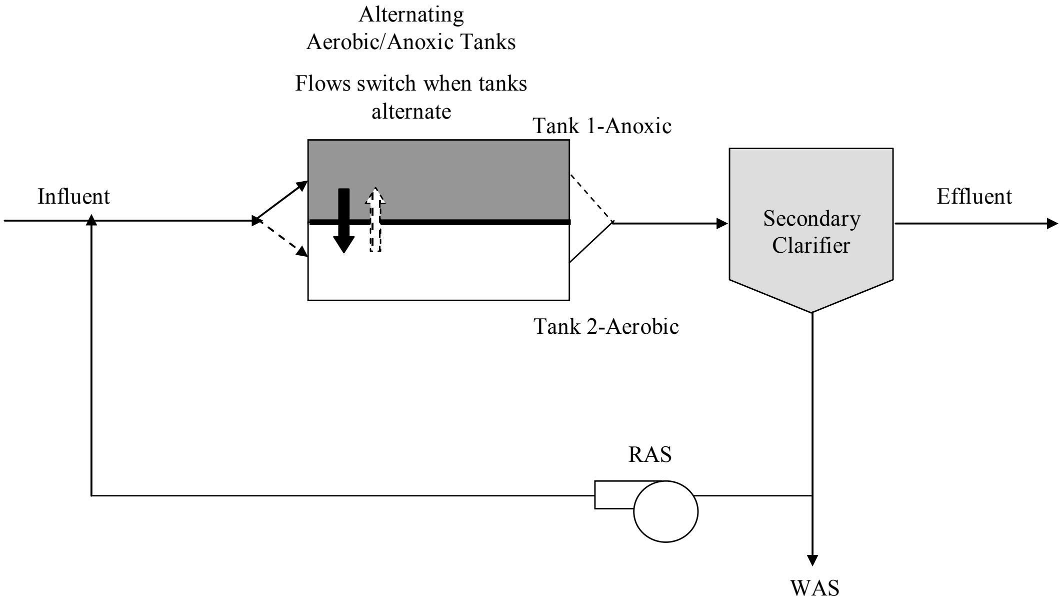


Figure 2-11. Biodenitro process.

Schreiber Process

The patented Schreiber countercurrent aeration process can provide nitrification and denitrification in one basin. The wastewater enters a circular basin equipped with a rotating bridge that provides mixing. Aeration is provided by fine-bubble diffusers attached to the bridge. Should sequencing between aerobic and anoxic conditions be required, the aeration can be turned off while the bridge continues to keep the tank mixed. The system footprint is approximately that of a conventional activated-sludge system but with additional equipment built in. Because it includes alternating aerobic/anoxic conditions in one tank, very low effluent TN concentrations are possible, when optimized. The performance could be further enhanced with additional anoxic zones downstream of the primary reactor. In the standard Schreiber configuration, the need for additional food is typically reduced or eliminated because everything is done in one tank. The Schreiber countercurrent aeration process is illustrated in Figure 2-12. The TN effluent concentrations for the Schreiber process in the literature evaluated as part of this study was 8.0 mg/L (see Table 2-1 in Section 2.5).

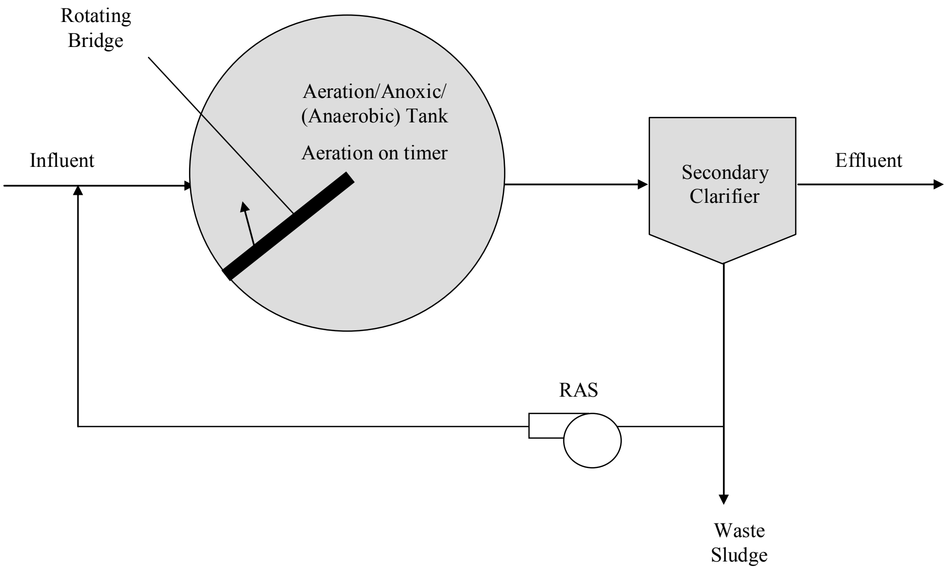


Figure 2-12. Schreiber countercurrent aeration process.

Sequencing Batch Reactor

Facilities with small or intermittent flows might benefit from using an SBR. Such reactors are filled over the course of time, and then the contents are processed under the conditions deemed necessary to achieve the required treatment. In the case of nitrogen removal, the conditions would include both aerobic and anoxic time. Multiple SBR units allow for continuous feeding of wastewater and batch processing. SBRs typically have four phases: a fill phase, during which mixing is maintained; a react phase with alternating aerobic and anoxic cycles; a settle phase, during which mixing is turned off so the microorganisms can settle; and a decant phase, when the effluent is drained. Depending on the length of time needed for filling, reacting, and decant, multiple SBR units allow for receiving wastewater continuously. The footprint for SBRs can be small, depending on the required number of reaction vessels. The need for additional food is typically low because everything is done in one vessel. Very low effluent nitrogen can be obtained through the use of multiple aeration and anoxic steps, with food added if needed. An SBR is illustrated in Figure 2-13. The range of TN effluent concentrations for the SBR found in the literature evaluated as part of this study was 1.6 mg/L to 13.6 mg/L (see Table 2-1 in Section 2.5).

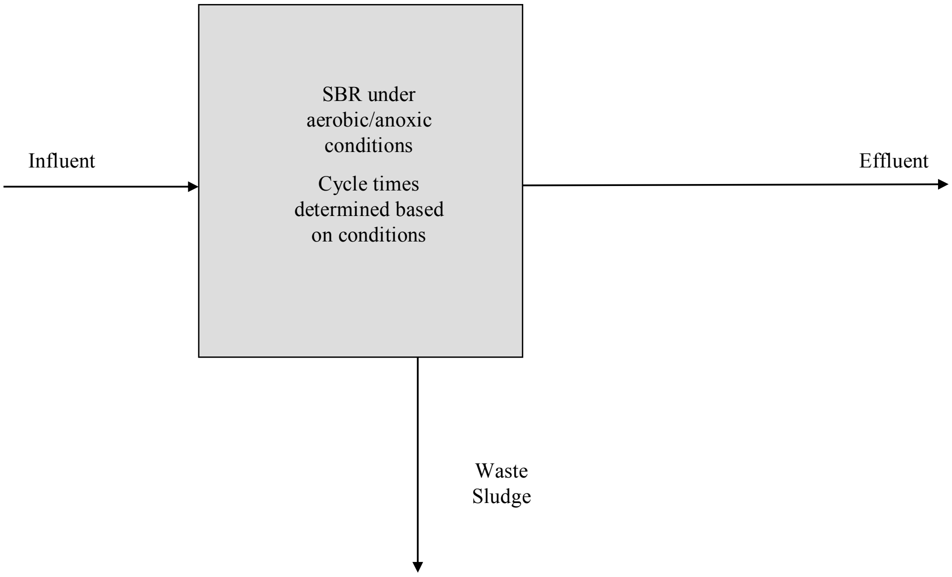


Figure 2-13. Sequencing batch reactor.

Sidestream Processes

One issue that needs to be decided at plants where low TN is required in the effluent is whether and how to treat recycle nitrogen loads coming from sludge processing and other activities and deciding whether to include a separate treatment of those streams. These streams typically contain high concentrations of ammonia nitrogen and would make treatment in the main biological treatment system difficult to manage when the sludge handling processes send large loads.

Typical solutions for high sidestream loads would be equalization; diurnal control of the sidestream flow so that it is fed during lower mainstream load periods; or dedicating one mainstream train to handle the sidestream. Often, a better solution is to treat the sidestream before recycle. The following systems have been proposed for such sidestream nitrogen treatment and have been implemented at some places in the United States and Europe.

InNitri Process—Nitrification

The ammonia-laden water is treated in a separate nitrification reactor before recycling to the plant headworks. This therefore reduces the ammonia-nitrogen load in the recycle stream. The recycle stream then provides a seed of nitrifying bacteria to the main reactor. Such a supply would often not be available in the mainstream because of having to maintain too low a SRT so that the nitrifiers would wash out of the system (Philips and Kobylinski 2007). This constant feed of nitrifiers is thus beneficial for facilities that must nitrify or achieve low effluent concentrations year-round. The sidestream reactor size can be small, and it can be operated at an elevated temperature compared to the main reactor. By seeding that main system with nitrifiers, the main SRT can be reduced, which can thus reduce overall capital and O&M costs. The process has been piloted in the United States in Arizona (Warakomski et al. 2007). The process is illustrated in Figure 2-13A.

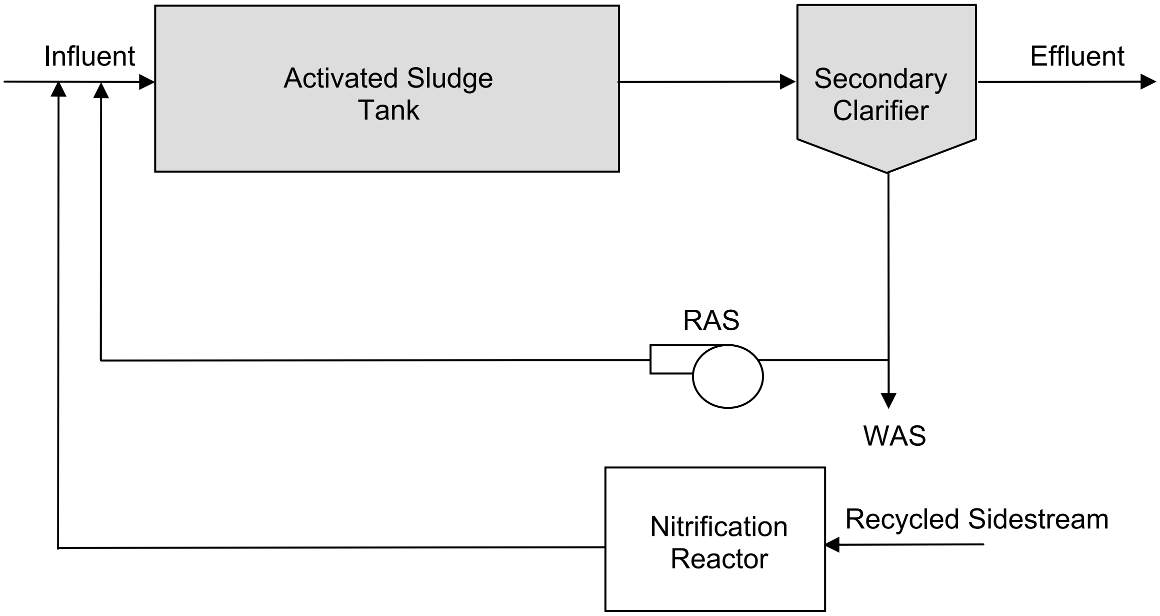


Figure 2-13A. InNitri process.

Bio-Augmentation Batch Enhanced (BABE)—Nitrification

The BABE process is a variation on the InNitri process. In BABE, the reactor is a batch system that is fed batches of return sludge from the main activated sludge system along with the sidestream. This batch reactor is operated both aerobically and anoxically and therefore both nitrifies and denitrifies the sidestream. This means that alkalinity lost during the nitrification process will be partially recovered during denitrification, and the sidestream will not require neutralization before reintroduction to the mainstream. The effluent of the batch reactor contains nitrifiers, which will enhance the population in the main system. Full-scale testing has been done in the Netherlands (Philips and Kobylinski 2007). The process is illustrated in Figure 2-13B.

There are a number of other bio-augmentation processes that have been developed in Europe and elsewhere. Examples include the Mainstream Autotrophic Recycle Enabling Enhanced N-removal (MAUREEN), the Bio-augmentation R (regeneration) (BAR) process, the Aeration Tank 3 (AT-3) Process, and the biofilm activated sludge innovative nitrification (BASIN) process (Parker and Wanner 2007). The BAR and AT-3 processes have been proven in demonstration scale, while others show promise for the future.

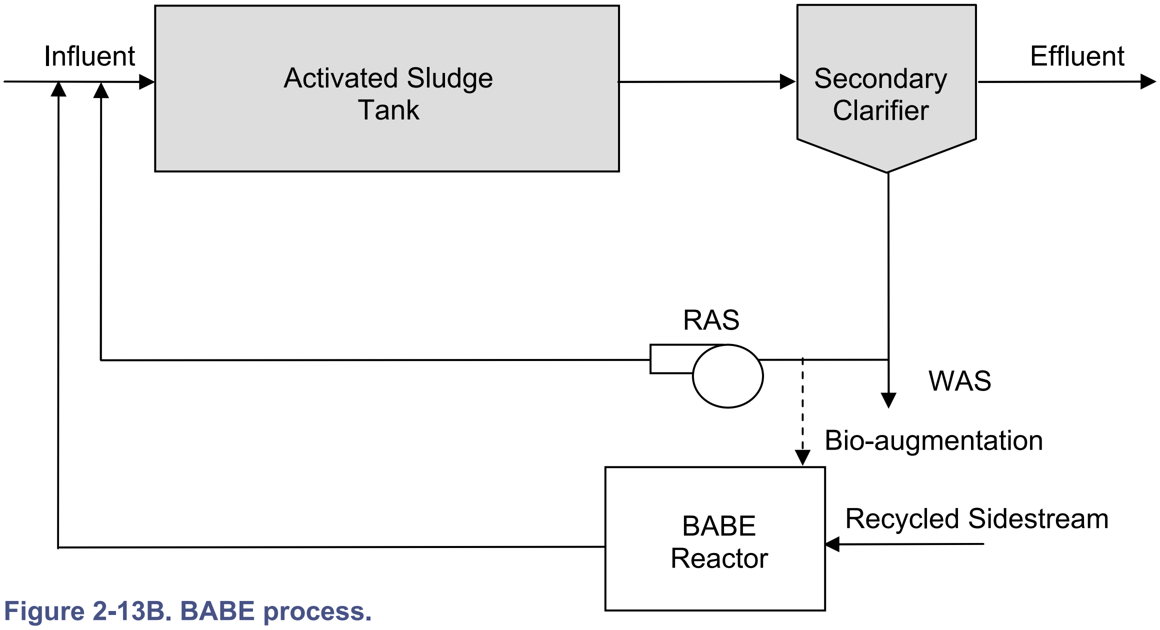


Figure 2-13B. BABE process.

Nitritation–Denitritation

A recently developed alternative to conventional nitrification/denitrfication is the process of nitritation, where only nitrite is produced aerobically. This process is sold under the proprietary name SHARON, or Single-reactor High-activity Ammonia Removal over Nitrite. The process is run at elevated temperatures (30–35 °C) at lower SRTs to favor growth of ammonia oxidizers (such as *Nitrosomonas* sp.) over nitrite oxidizers (such as *Nitrobacter* sp.). Denitrifiers are then encouraged to convert the nitrite to nitrogen gas. By not oxidizing all the way to nitrate, oxygen and energy usage is reduced (Warakomski et al. 2007). Methanol can be used for dentrification, if necessary. The process is used at several locations in Europe and, as of 2007, was being installed at the New York City Ward Island Water Pollution Control Facility. The SHARON process is illustrated in Figure 2-13C.

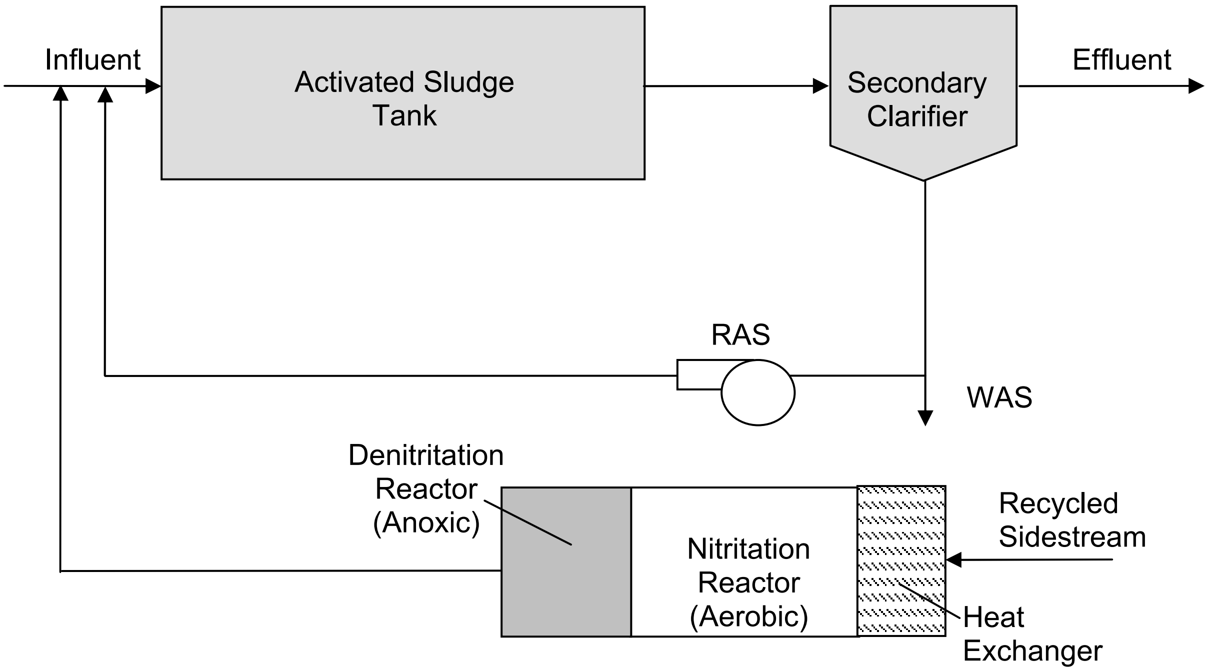


Figure 2-13C. SHARON process

Another process for converting ammonia to nitrite uses a newly discovered group of autotrophic microorganisms that can anaerobically oxidize ammonia using nitrite. This is called the ANAMMOX process, for Anaerobic Ammonia Oxidation. These microorganisms are favored by elevated temperatures (above 35 °C), and grow very slowly. In this case, a strategy for obtaining nitrite is to use the first step of the SHARON process to produce nitrite, then oxidizing a bypassed ammonia stream with the nitrite. ANAMMOX systems have been implemented in Europe, in particular in Rotterdam (Warakomski et al. 2007). The ANAMMOX process is presented in Figure 2-13D. Two fixed-film processes using similar strategies to SHARON and ANAMMOX—the Oxygen Limited Aerobic Nitrification-Denitrification (OLAND) and Completely Autotrophic Nitrogen Removal Over Nitrite (CANON)—are under development (Stensel 2006).

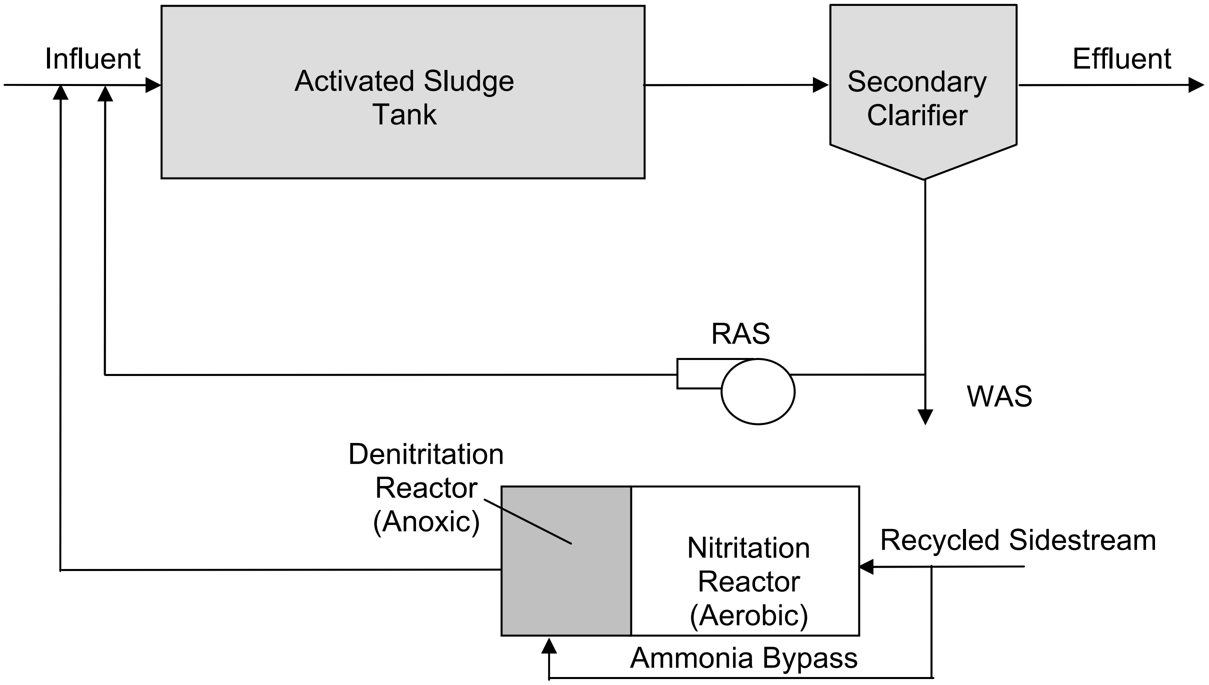


Figure 2-13D. ANAMMOX process.

2.3 Phosphorus Removal Processes

Phosphorus can be removed from wastewater by biological uptake by microorganisms and by chemical precipitation with a metal cation. Depending on the target concentration, a plant process might employ both technologies. Such a combined approach might be of particular benefit if the target concentration is very low and the starting concentration is high. In such a case, biological removal is used to remove the bulk of the phosphorus, and chemical polishing follows to achieve the final concentration; such an approach tends to reduce sludge formation.

2.3.1 Biological Phosphorus Removal

Biological Phosphorus Species

The reactions involved in biological phosphorus removal are as follows (Pattarkine and Randall 1999):

1. Anaerobic biological phosphorus release by Phosphate Accumulating Organisms (PAOs):

$$\text{PAOs} + \text{stored polyphosphate} + Mg^{++} + K^{+} + \text{glycogen} + \text{VFA} \rightarrow$$
$$\text{PAOs} + \text{stored biopolymers} + Mg^{++} + K^{+} + \_CO_2 + H_2O + PO_4^{3-} \text{ (released)}$$

2. Aerobic biological phosphorus uptake by PAOs:

$$\text{PAOs} + \text{stored biopolymers} + Mg^{++} + K^{+} + O_2 \text{ (or } NO_3) + PO_4^{3-} \rightarrow$$

$$\text{PAOs} + \text{stored polyphosphate} + Mg^{++} + K^{+} + \text{glycogen} + CO_2 + H_2O$$

Biological phosphorus removal works by encouraging the growth of phosphate-accumulating organisms (PAOs), which are then subjected first to anaerobic conditions and then to aerobic conditions. Under anaerobic conditions, the microbes break the high-energy bonds in internally accumulated polyphosphate, resulting in the release of phosphate (PO_4^{3-}) and the consumption of organic matter in the form of volatile fatty acids (VFAs) or other easily biodegraded organic compounds. VFAs include short-chained carboxylic acids such as acetic, proprionic, butyric, and valeric, among others. When the microbes are then put under aerobic conditions, they take up phosphate, forming internal polyphosphate molecules. This *luxury uptake* results in more phosphate being included in the cells than was released in the anaerobic zone, so the total phosphate concentration in solution is reduced. When the micro-organisms are wasted, the contained phosphate is also removed.

Beyond the luxury uptake of phosphate, microorganisms also remove phosphate as part of their normal BOD removal. A small amount of phosphorus is removed in the conventional activated-sludge process during BOD removal. This amount is typically 1.5 to 2 percent on a dry weight basis (WEF and ASCE 1998).

When these organisms enriched in polyphosphate are wasted, the contained phosphate is also removed. PAOs that are exposed to an anaerobic environment followed by an aerobic zone can exhibit phosphorus removal levels 2.5 to 4 times higher than those for conventional activated-sludge systems (WEF and ASCE 1998). Thus, the fraction of phosphorus in wasted dry solids could be at least 8 percent or more.

A sufficient supply of VFAs is the key to removing phosphorus biologically. Barnard et al. (2005) reported that when using a mixture of acetic and propionic acids produced by on-site fermentation, the COD-to-TP ratio in the plant influent could be as low as 8. As the phosphorus permit limits have been lowered in recent years, chemical polishing, often combined with better filtration processes, has become necessary. A discussion of significant factors that affect the phosphorus removal process follows.

Biological Phosphorus Removal Factors

Volatile Fatty Acid Availability in Wastewater

A key factor in determining the cost-effectiveness of biological phosphorus removal is the relative amount of organic material that can be used by the PAOs. That is because if VFAs or rbCOD is not present in a sufficient ratio to ortho-phosphorus, the process becomes less reliable and phosphorus removal can be reduced.

For facilities where retrofitting is being considered, the data most likely available are COD and BOD. Literature indicates that a COD-to-TP ratio of 45 and a BOD-to-TP ratio of 20 are the minimum values needed to meet a 1 mg/L TP limit (McGrath 2005a; WEF and ASCE 2006). If data are available about rbCOD, literature shows that good biological phosphorus removal was observed at rbCOD-to-TP ratios of about 15 (Barnard et al. 2005). The rbCOD can be converted to short-chain VFAs in the anaerobic zone and then used by PAOs (Lindecke et al. 2005). Finally, if VFAs have been determined, it has been found that a minimum VFA-to-TP ratio of at least 4 is recommended to obtain good biological phosphorus removal (Neethling et al. 2005).

In-plant generation of VFAs is made possible in many different ways at existing facilities—adding a new fermenter (for either primary or RAS), converting a sludge thickener, or returning supernatant from an existing anaerobic digester. Fermenters can be single-stage or two-stage. They are optimized by adding a mixer that allows VFAs in the solids to enter the anaerobic zone directly. A source of elutriating water, either primary effluent or final effluent, is fed to the fermenter to flush out the VFAs produced and sent to the anaerobic zone. The VFA production from fermenting primary sludge typically yields 0.066 to 0.15 g VFA/g total solids (both expressed as COD), although values up to 0.3 g VFA/g solids have been reported (Barnard et al. 2005). Complete-mix fermenters are typically designed with an HRT of 6 to 12 hours, an SRT of 4 to 8 days, and a solids concentration between 1 and 2 percent (WEF and ASCE 2006, p 326).

Temperature

Although the biological phosphorus removal process is not significantly affected by temperature, the fermentation process is slower at low temperatures (Lindeke et al. 2005). Therefore, lower phosphorus removal might occur in the winter because of reduced VFA production in the plants that use fermenters. The temperature effect was reported as directly influencing the sludge age needed for adequate generation of VFAs (Baur et al. 2002). At 24 °C, a 1-day sludge age was sufficient. A 4-day sludge age was required to generate sufficient VFAs at 14 °C.

At the upper end of the temperature range, the performance of PAOs showed reduced phosphorus uptake activity above 30 °C and seriously inhibited activity at 40 °C (Panswad et al. 2003; Rabinowitz et al. 2004). At temperatures above 30 °C, glycogen-accumulating organisms (GAOs) were reported as a detriment to enhanced biological phosphorus removal (EBPR) (Barnard 2006).

Solids Retention Time

Very good biological phosphorus removal performance was reported when SRT values of 16 and 12 days were provided for wastewater at 5 °C and 10 °C, respectively. The system performance was not affected when the SRT was varied between 16 and 24 days at 5 °C.

Similarly, varying the SRT between 12 and 17 days at 10 °C did not affect the biological phosphorus removal (Erdal et al. 2002).

Secondary Phosphorus Release

Secondary release is of concern in EBPR plants. Biological phosphorus removal occurs in a two-step process, in which phosphorus is released in the anaerobic zone by PAOs and then taken up by the same PAOs in the aerobic zone. The microorganisms favored in the anaerobic zone are capable of absorbing more phosphorus in the aerobic zone than was released in the anaerobic zone, leading to a net reduction in phosphorus when these microorganisms are removed from the process through settling and wasting. However, if the PAOs are put under anaerobic conditions following phosphorus update, there will be unintended secondary release of phosphorus. Anaerobic conditions can arise in secondary clarifiers with long SRTs, inside the tertiary filters, and in some sludge-handling operations. In addition, anaerobic conditions can arise in the activated sludge unit if aeration is not maintained at a sufficiently high level or if the extent of oxygen tapering through a plug flow system is too great. The released phosphorus is often returned to the head of the secondary process, via the RAS from the secondary clarifiers or supernatant/filtrate from sludge-handling operations. This recirculated phosphorus thus increases the load on the secondary process and decreases the overall biological phosphorus removal that can be achieved. Secondary phosphorus release can be reduced by minimizing the amount of time that mixed liquor or return sludge is held before recirculation, reducing return flows from sludge-handling operations, and treating the sludge-handling return streams before introduction to the secondary process. In some cases, a small dose of alum is added to the tertiary filter to minimize the secondary release (see the Clark County, Nevada, Case Study in Chapter 3). Sidestream treatment has been proposed for some large facilities, including facilities in New York (Constantine 2006) and Washington, D.C. (Constantine 2005).

Nitrates in Return Streams

Nitrates in the return streams, such as RAS or internal recycle lines, can negatively affect biological phosphorus removal. The nitrates cause consumption of the VFAs needed for biological phosphorus removal and introduce a source of oxygen that is used before phosphorus release, minimizing the amount of biological phosphorus that can be removed. Because 1 mg/L nitrate-N is the equivalent of 2.86 mg/L DO, and nitrate does not have a maximum concentration like DO, nitrate has the potential to cause even more disruption to biological phosphorus removal than DO. In a step-feed system, McGrath et al. (2005) determined that 6 mg/L of nitrate nitrogen was the upper limit tolerable for successful EBPR in one particular full-scale operation. If sufficient rbCOD is available, higher concentrations of return stream nitrate could possibly be tolerated with adequate phosphorus removal.

Dissolved Oxygen in Return Streams

Similar to nitrates, DO entering the anaerobic zone negatively affects biological phosphorus removal. The recycled DO essentially reduces the anaerobic HRT, because it needs to be reduced to zero before anaerobic metabolic activities for biological phosphorus removal can occur. Because the PAOs are facultative aerobes, they will use any available oxygen present in the anaerobic zone to aerobically metabolize VFAs. This results in a reduced anaerobic phosphorus release and, thus, reduced phosphorus uptake in the aerobic zone. In addition, heterotrophic aerobic organisms present in the anaerobic zone will compete for the VFAs, further reducing PAO anaerobic metabolism. If DO sources in the return stream cannot be minimized, additional VFAs might be required (Benisch 2004).

2.3.2 Chemical Phosphorus Removal

Chemical Phosphorus Species

Current chemical phosphorus removal design is based on equilibrium precipitation theory (WEF 1997, 1998; USEPA 1987a, 1987b). The chemical precipitate of ortho-phosphorus is carried by treatment with a trivalent metal cation, typically ferric ion (Fe^{3+}) or aluminum (Al^{3+}). Ferric ion is typically supplied in the form of ferric chloride ($FeCl_3$). Aluminum is supplied as alum (aluminum sulfate). When a source like waste pickle liquor is available, the ferrous ion (Fe^{2+}) can be used as the metal cation. The precipitation reaction depends on the various phosphate species (e.g., $H_2PO_4^{1-}$, HPO_4^{2-}) being converted to PO_4^{3-}, with the consumption of alkalinity (or OH^-, hydroxide ion). This means that sufficient alkalinity must be present for the chemical precipitation reaction to be completed.

1. Conversion of phosphate species to phosphate ion:

$$H_nPO_4^{(3-n)-} + nOH^- \rightarrow PO_4^{3-} + n\ H_2O$$

2. Chemical phosphorus removal by alum (aluminum sulfate):

$$Al_2\ (SO_4)_3 \rightarrow 2\ Al^{3+} + 3\ (SO_4^{2-})$$

$$Al^{3+} + PO_4^{3-} \rightarrow AlPO_4\ (s)$$

3. Chemical phosphorus removal by ferric chloride:

$$FeCl_3 \rightarrow Fe^{3+} + 3\ Cl^-$$

$$Fe^{3+} + PO_4^{3-} \rightarrow FePO_4\ (s)$$

4. Ferrous ion conversion to ferric ion:

$$2Fe^{2+} + \tfrac{1}{2}\ O_2 + 2H^+ \rightarrow 2Fe^{3+} + H_2O$$

Chemical addition occurs in primary clarifiers (when present) or in the secondary process, in the aeration basin or upstream of the secondary clarifiers, or in tertiary clarifiers or other treatment processes. Feeding chemicals to the primary clarifiers usually requires less chemical use than feeding to the secondary or tertiary process. Feeding chemicals at both the primary and secondary clarifiers results in less chemical use than feeding to the secondary process alone, allowing some phosphorus to be removed in the primary clarifiers and polishing to occur in the secondary process.

Recent research shows that in addition to equilibrium precipitation, sorption is a predominant phenomenon. The factors that promote sorption, such as sorbent characteristics, variable stoichiometry, alkalinity, mixing at point of dosage, diffusion, and time-based kinetics, are processes that must be considered in designing systems to achieve a low total phosphorus (TP) concentration (Smith et al. 2007).

Chemical Phosphorus Removal Factors

Phosphorus Species at Application Point

Phosphorus in the raw wastewater is found in three forms—organically bound phosphorus, polyphosphate, and orthophosphate. Organically bound phosphorus can be settled in the primary clarifiers or transformed into orthophosphates by the microorganisms in the secondary process. Polyphosphates are soluble and pass through the primary clarifiers. In the secondary process, the polyphosphates are converted biologically into orthophosphates. Orthophosphates are also soluble and will pass into the final effluent if they are not removed biologically or chemically. Orthophosphates readily form a precipitate following the addition of metal salts, forming a floc that can then be settled or filtered from the wastewater.

Metal-to-Phosphorus Ratio

In general, the molar ratio of the metal to influent phosphorus concentration (Al to P or Fe to P) increases as the target phosphorus effluent concentration decreases. The curve is flatter at higher effluent phosphorus concentrations, but it becomes particularly steep as the target concentration decreases below 0.5 mg/L (WEF and ASCE 1998). For a given amount of phosphorus to be removed, the amount of chemical that needs to be fed is less for plants that employ tertiary chemical treatment following biological phosphorus removal, as compared to plants that use one- or two-point chemical phosphorus removal. When simultaneous precipitation is practiced, the MLSSs will contain a high phosphorus concentration, and it could easily exceed the concentration for an EBPR sludge.

Choice of Chemical

Alum and ferric chloride are often used for chemical phosphorus removal. The choice of which chemical to use should be made on the basis of jar testing and chemical costs. The impact on downstream processes is another factor to consider. Alum sludge can be more difficult to thicken and dewater than sludge from ferric chloride. Ferric chloride is more corrosive and requires special piping. In addition, ferric chloride can cause problems with ultraviolet disinfection if the chemical is baked onto the quartz sleeves. Ferrous sulfate is a viable alternative if a source is available. The presence of other metals or species that react with phosphate can cause simultaneous precipitation of multiple metal phosphates or complexation effects that would thus require higher aluminum or ferric doses than expected.

Feed Point Location

Chemicals for phosphorus removal can be added to the primary clarifiers, the secondary treatment system (either to the aeration basin or upstream of the secondary clarifiers), or the tertiary treatment process. The chemical should be well mixed with the wastewater at the feed location to form a good floc. If the chemical is not fed to a well-mixed location, an in-line static mixer might be needed.

Phosphorus is a micronutrient and is needed by the microorganisms in the aeration basin to remove BOD and nitrogen. Care must be taken not to remove too much phosphorus in the primary clarifiers. A general rule of thumb is that the phosphorus in the influent to the activated-sludge basin should be 1 to 1.5 percent of the BOD to be removed. For example, if 200 mg/L of BOD is to be removed in the aeration basin, at least 2 to 3 mg/L of phosphorus is required in the primary effluent. This rule-of-thumb ratio, however, can vary according to sludge age and the chemical dosage. If too much phosphorus is removed in the primary clarifiers, the operation of the aeration basin might be negatively affected.

Number of Feed Points

Alum or ferric chloride can be fed at one, two, or more locations in the plant. Single-point application works well for plants that need to achieve moderate phosphorus removal concentrations (approximately 0.5 mg/L). Two feed points, which might be either primary and secondary clarifiers or secondary clarifiers and tertiary treatment, can achieve lower phosphorus concentrations and can use less chemicals than dosing in one location. Several of the tertiary treatment processes operate more efficiently if the TP concentration in the secondary effluent is less than 1 mg/L. For plants that do not have the ability to remove phosphorus biologically, two chemical feed points might be required because the tertiary treatment process typically requires alum or ferric chloride to form a floc before removal. Three feeding locations might be appropriate for plants that have a relatively high TP raw influent concentration (greater than 6 mg/L), which might occur in communities with significant commercial and industrial discharges.

Mixing Requirements

Rapid mixing is required when the chemical is added to the wastewater to allow the molecules to react. In addition, the density and viscosity of the metal salts are larger than that of the wastewater, which allows the chemical to settle to the bottom. If rapid mixing conditions do not exist at the chemical injection location, an in-line static mixer might be required.

After the initial rapid mixing, gentle mixing is required to allow larger flocs to form. Usually, the movement of the wastewater through the treatment plant is sufficient for the floc formation. However, turbid areas of the plant might break up the floc, which would decrease phosphorus removal through settling or filtration.

pH

Phosphorus solubility varies between iron and aluminum ions. Theoretically, the lowest effluent phosphorus concentration achievable with iron salts is 0.07 mg/L at a pH of approximately 6.9 to 7.0 s.u. Recent laboratory results indicate that uniformly high removal may occur at pH values between 5.5 and 7 (Smith 2007). Aluminum salts are theoretically capable of achieving effluent phosphorus concentrations down to 0.01 mg/L in a pH range of 6.6 to 7.2 s.u. (Kang et al. 2001; WEF and ASCE 1998). The actual effluent phosphorus concentration achieved and the optimum pH range will be site-dependent because of other chemical reactions that occur in the wastewater. Recent research indicates that the highest phosphorus removal efficiency occurs at a pH of between 5.5 and 7.0 s.u. However, the pH of the wastewater does not have a significant effect on chemical phosphorus removal (Smith et al. 2007). Alum and ferric chloride are acidic and therefore capable of lowering the effluent pH. If sufficient alkalinity is not available to adequately buffer the wastewater, pH adjustment might be needed.

Suspended Solids Removal

Capturing the floc formed during chemical precipitation plays an important role in phosphorus removal. Sludge at WWTPs that use biological phosphorus removal contains 4.5 percent phosphorus on a dry weight basis compared to 1.5 percent phosphorus for plants that use chemicals (USEPA 1987a). The phosphorus in the suspended solids in the effluent will require additional removal of the total suspended solids (TSS) in the final effluent in meeting low phosphorus limits. Tertiary processes like clarifiers or filters are more efficient at capturing solids. Feeding metal salts in conjunction with the tertiary process improves phosphorus removal.

Sludge Handling

Chemical phosphorus removal generates additional sludge. Phosphorus can be released from the chemical sludge if the sludge is exposed to or stored in an anaerobic environment. Return flows, such as supernatant or water from dewatered sludge, can contain high levels of phosphorus. Maintaining aerobic conditions in the sludge is recommended. If this is not feasible, treatment of the sludge return flows should be considered.

Recent analysis of phosphorus precipitation (Smith et al. 2007) suggests possible thermodynamic and kinetic factors in achieving extremely low (less than 0.01 mg/L) effluent phosphate concentrations. These factors include pH, metal (aluminum or iron) used for precipitation, the presence of additional metals, materials that can complex with the phosphate such as COD and TSS, alkalinity, degree of mixing, and process time.

2.3.3 Phosphorus Removal Technologies

Enhanced Biological Phosphorus Removal Technologies

Fermentation

If biological phosphorus removal is desired, the process is carried out under anaerobic conditions with VFAs providing the carbon source for the microorganisms. If the influent has too low a VFA concentration for adequate phosphorus removal (greater than approximately 4-to-1 mass of VFAs to mass of phosphorus in the influent to be removed) the concentration must be increased. The VFAs can be supplied from an outside source or can be formed by on-site fermentation of primary sludge or RAS. Fermenting sludge to generate VFAs has the additional advantage of reducing the amount of sludge to be disposed of. Fermenting the primary sludge is preferred to using secondary sludge because doing so reduces the amount of phosphate released from the sludge. Fermentation is particularly helpful for large plants in cold climates and for plants where the strength of the wastewater tends to be low. In some situations, fermentation also occurs in the collection system, as well as in the anaerobic zone. Fermentation is illustrated in Figure 2-14.

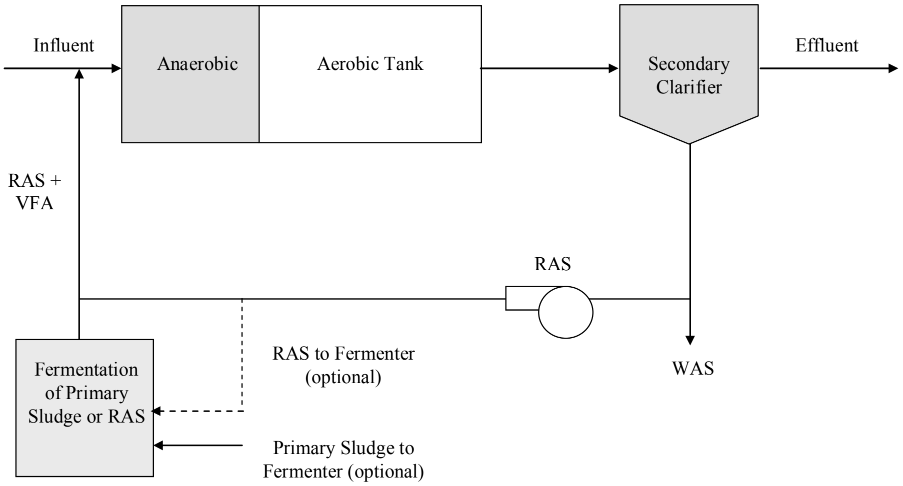


Figure 2-14. Fermentation process.

Anaerobic/Oxic (A/O), or Phoredox, Process

The anaerobic/oxic (A/O) process consists of an anaerobic zone upstream of an aerobic zone. The RAS enters the head of the anaerobic zone with the influent. In the anaerobic zone, PAOs release phosphorus, which is subsequently taken up in the aerobic zone. One potential problem for A/O operation is that any nitrates recycled from the aerobic zone of side streams can inhibit anaerobic growth (selection) of PAOs. To reduce this effect, the anaerobic zone is often split into an anoxic chamber for nitrate denitrification and a series of anaerobic zones for phosphorus release. The process has a medium-sized footprint and is relatively easy to retrofit into an existing basin by installing baffle walls and mixers to produce an anaerobic zone. If sufficient VFAs are available, an additional carbon source is not needed. Because there is some additional pumping, there are some additional electrical costs; however, less sludge is generated under anaerobic conditions. To obtain extremely low phosphorus (less than 0.1 mg/L), chemical addition should be examined. The A/O (Phoredox) process is illustrated in Figure 2-15. The range of TP effluent concentrations evaluated in the literature reviewed for this study was 0.025 mg/L to 2.3 mg/L (see Table 2-5 in Section 2.5 for details).

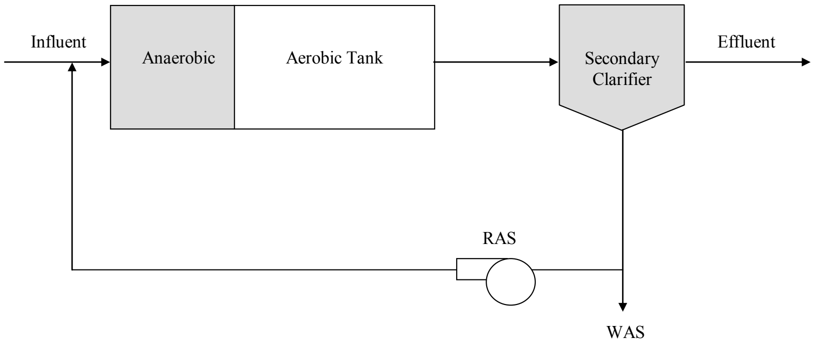


Figure 2-15. A/O (Phoredox) process.

Oxidation Ditch

Oxidation ditches were discussed earlier in the *Nitrogen Removal* section. The design and operation of an oxidation ditch for phosphorus removal is much the same, with the requirement that an anaerobic zone be established. The anaerobic zone (sometimes called a *selector*) can be set up within the ditch or as an external tank upstream of the ditch. The oxidation ditch does not necessarily need to be operated with anoxic zones, although doing so can aid in the partial recovery of alkalinity. As with the A/O process, additional carbon in the form of VFAs is needed only if sufficient rbCOD is not already present in the influent. To obtain very low phosphorus (under 0.1 mg/L), additional carbon is required. The carbon should be added upstream of the secondary clarifier to avoid depleting that nutrient from the biological process. Lower TP concentrations can be achieved by close monitoring and regulation of the anaerobic zone flow and DO levels. An oxidation ditch with an anaerobic zone is illustrated in Figure 2-16. TP effluent concentrations range between 1 and 2 mg/L with the anaerobic zone. See *Concentric Oxidation Ditch* elsewhere in this chapter for its performance.

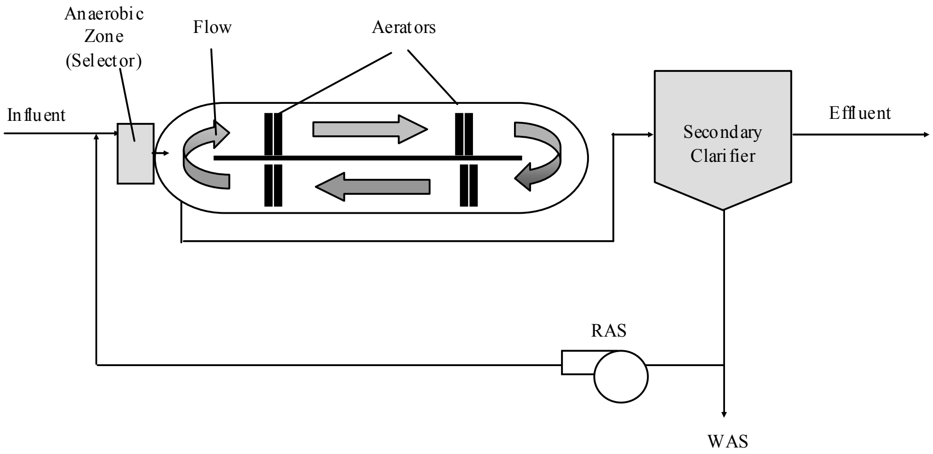


Figure 2-16. Oxidation ditch with anaerobic zone.

Physical-Chemical Technologies

Chemical Addition

Aluminum or iron salts—most commonly alum (aluminum sulfate) and ferric chloride—can be added to precipitate phosphorus in the secondary clarifiers or tertiary filters. Adding chemicals results in a limited capital investment initially, but chemical costs and the additional sludge generated increase the O&M costs of the process. Chemicals can be added at a number of locations in the wastewater treatment process; the most common are the primary clarifiers, the secondary basins just before the secondary clarifiers, and upstream of the tertiary filters. Care must be taken in the first two cases not to add so much chemical as to take the phosphorus below the concentration needed to sustain the secondary treatment microorganisms.

Chemical addition allows the facility to obtain extremely low effluent concentrations because more chemical can always be used with no significant impact on the microorganisms. However, using more chemicals creates a greater volume of sludge. Filtration can be used to remove fine precipitate particles and achieve lower concentrations. The footprint for chemical addition is small, and only limited piping and pumping are required (see Chapter 5 for more details). This means the capital costs are low, but the operating costs can be high for chemicals and sludge disposal. The phosphorus chemical/filter process is illustrated in Figure 2-17. The range of TP effluent concentrations from combined chemical addition followed by filtration from the case studies and evaluated in the literature reviewed for this study was 0.1 mg/L to 2.3 mg/L (see Table 2-5 in Section 2.5 for details).

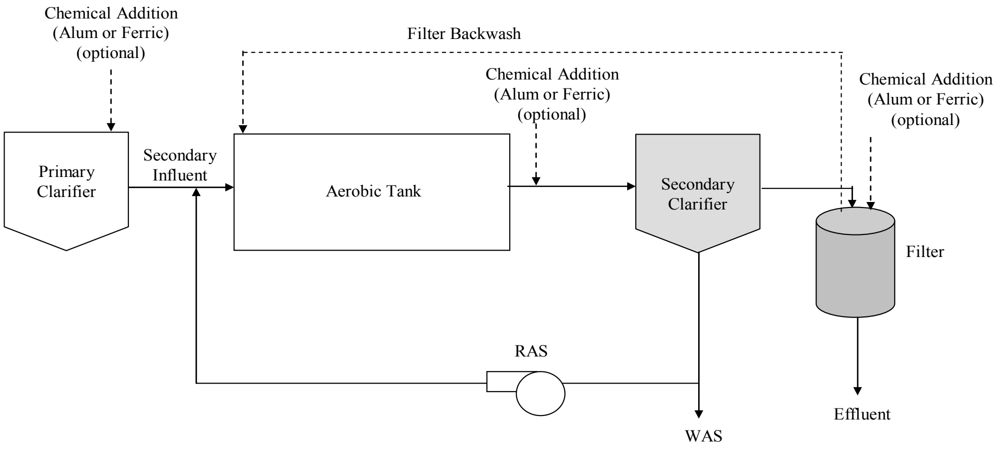


Figure 2-17. Phosphorus chemical/filter process.

Chemical Removal with Conventional Filtration Technologies

Sand filters are used to polish effluents by removing suspended solids. To the extent that phosphate is associated with suspended solids, sand filters can provide effective removal. Rapid mixing should occur before entering the filter to ensure that the chemicals are distributed throughout the secondary effluent, allowing floc to form. Descriptions of the some of the filtration alternatives are provided below. They have particular application downstream of chemical addition and when the discharge standard is below 0.1 mg/L.

Filters can be used to capture phosphorus in the solid phase. Adding chemicals can increase the effectiveness of the filters by precipitating the soluble phosphorus into a solid form that can be captured by the filters. Several varieties of filters can be used in multiple configurations. Filter media can range from mono-media, such as sand, to multimedia, such as sand with anthracite or gravel. Standard and deep-bed filters are available. Cloth filters, which use cloth to capture the solids rather than sand, can also be used. Filters can be operated in series to improve removal. For example, a deep-bed filter could be operated in series with a standard bed filter. A clarifier is used to remove solids from the filter backwash before the treated backwash is returned to the head of the plant.

Filter Media

Media within the filters trap particles between the pores. The smaller the pore spaces, the smaller the particles that can be removed. Descriptions of the most common filter media follow.

Sand Filters

Sand is typically used in single-medium filters. The size and uniformity of the sand depend on the application and characteristics of the secondary effluent at the individual plant. The uniformity of the sand is important because varying sizes can provide smaller pores for greater filtration; however, during backwashing the media can become stratified, with the smaller particles toward the top of the filter. Large solids particles in the secondary effluent might fill the pores between the small particles, decreasing the amount of water that can pass through the filter, increasing head loss, and leading to more frequent backwash cycles.

Dual-Media Filters

Dual-media filters typically use anthracite in combination with the sand. Anthracite can adsorb organic compounds in addition to filtering solids. It is softer than sand, and abrasion during backwashing can decrease the size of the anthracite particles. These smaller particles could cause blinding of the filter by trapping larger solids on the surface of the media, which would block the passage of the water if the anthracite is not removed during the backwash cycle. Other media combinations in dual-media filters include activated carbon and sand, resin beds and sand, and resin beds and anthracite.

Multimedia Bed Filters

Multimedia beds can contain anthracite, sand, and garnet or ilmenite. Garnet and ilmenite have a higher density than sand, allowing them to settle to the bottom of the filter after backwashing. In addition, garnet and ilmenite can have a smaller pore size than sand, allowing the smallest solids to be trapped by the garnet before the wastewater exits the filter when operated in a downflow mode.

Backwash Methods

Filters can be backwashed using water, air, or a combination of air and water. Filters can be backwashed using water alone. Air backwashing can be used as part of the backwashing process, but it is usually not effective in removing solids particles without the use of water. Descriptions of various backwash protocols are provided below.

The water acts to fluidize the bed, thereby releasing the particles trapped between the pores. The most effective way to backwash a bed is to force the filter media to rub together. This assists in removing the secondary effluent particles, which can adhere to media because of the biological nature of the solids, which can be *sticky*. Backwashing the filter using air before or during the water backwash cycle allows scouring of the media.

Air and water can be used simultaneously to backwash the filter. This process reduces the amount of water required to backwash the filter and also requires less time, reducing the volume of backwash water generated per cycle. Simultaneous air and water backwash can produce a cleaner filter, which will extend the run time of the filter compared to water alone or air scour followed by water. In addition, stratification does not occur with simultaneous air and water backwashing, so the smaller media grains do not accumulate at the surface of the filter bed. The smaller grains at the filter surface can become clogged by large particles in the wastewater, which reduces the filter run time. In an unstratified bed, the media grain sizes are distributed uniformly throughout the filter, allowing the suspended solids to be trapped throughout the filter depth, further increasing the run time of the filter.

Filtered effluent can be used to backwash and remove the solids trapped in the filter media. The filter typically needs to be backwashed at a high rate, approximately 15 to 20 gpm/ft^2, for at least 15 to 20 minutes. Backwashing generates a large volume of reject water to be returned to the head of the plant. The slug of backwash water, which contains solids and phosphorus, can negatively affect the operation of the treatment plant. To minimize the impact of the filter backwash, the water can be stored in an equalization basin and slowly fed back to the head of the plant; treatment with chemicals to remove some of the solids is also possible.

Bed Depth

The media depth can vary with the type of filter selected. Traveling bridge and pulse bed filters have relatively shallow bed depths of 10 to 12 inches. Standard beds typically contain approximately 2 feet of filter media, while deep beds contain approximately 4 to 6 feet of media. The deeper the bed, the longer the filter can operate before backwashing. However, deeper beds require a higher backwash rate to release the solids trapped in the media.

Gravity Filters

Wastewater is applied to the top of the media bed, allowing the water to flow downward by gravity. The wastewater is then collected in underdrains that allow the wastewater to enter but retain the filter media. Once a set amount of time passes or a predetermined head loss is measured in the filter, a backwash cycle is initiated. The filter is backwashed by either water or an air/water combination that removes the trapped solids from the media. The solids are directed to the head of the plant, while the filter media are retained. After the backwash cycle is complete, the filter is placed back into service. (Usually, the entire filter must be taken out of service to be backwashed.) A supply of filtered effluent (stored in a clear well) to backwash the filter might be required. If the production rate of the filters that remain in service is greater than the required backwash rate, the clear well might not be needed. Alternatively, some filters are divided into two to four cells, which allows only one cell to be taken out of service at a time while the remaining cells continue to produce water that can be used to backwash the out-of-service cell.

If the sand or alternate medium contains a variety of particle sizes, the smaller grains might accumulate at the top of the filter. The smaller grains have smaller pore sizes, which might be filled or blinded by the wastewater at a faster rate than the larger grains. This accumulation might lead to more frequent backwashing of the filters. Using an air/water backwash system, rather than water alone, might minimize this problem by not fully fluidizing the bed.

Moving-Bed Filter

A moving-bed filter cleans a portion of the granular media continuously so that operation does not need to stop to perform a backwash cycle. The filter can operate in a downflow or upflow mode. In the downflow mode, wastewater enters the top of the filter and flows downward through the media. The solids filtered from the wastewater are drawn downward with the sand. An airlift pump transfers the solids and sand to the top of the filter, where a filter washbox is located. The sand is separated from the solids by gravity. The cleaned sand is returned to the top of the filter, and the solids are returned to the plant headworks or directed to disposal.

In an upflow moving-bed filter, the wastewater enters through the bottom of the filter and is pumped upward through the sand. Solids captured in the sand move downward and are airlifted to a reject compartment through the center of the filter. The turbulence created by the air lift pumps separates the solids from the sand. The clean sand is separated from the solids by gravity. The solids are directed to the headworks of the plant, and the clean sand is deposited on top of the filter. The advantage of an upflow filter is that the wastewater encounters the sand containing the most solids first and passes through the cleanest sand before it exits the filter over a weir.

Pulsed-Bed Filters

Pulsed-bed filters have a relatively shallow bed containing approximately 10 inches of sand or alternate medium. Secondary effluent is applied to the top of the filter. Diffusers are located on the top of the bed surface. Once the water level reaches the *air mix* probe, air is supplied to the diffusers, suspending the larger particles above the bed; filtration continues during this process. As operation continues, solids might settle onto the surface of the bed, causing a rise in the water level to the *pulse mix* probe. When the bed is pulsed, effluent is not discharged from the filters. Air that is trapped in the underdrains is released by backwash pumps and travels upward through the clogged media. The solids are released at the top of the bed and suspended by the diffusers on the media surface. The effluent valve is then reopened, and operation of the filters can continue. After a set number of pulses, a full backwash of the filter occurs to remove the trapped solids from the filter. The pulses are designed to extend the operation of the filter and decrease the number of backwash cycles required compared to a conventional filter.

Traveling Bridge/Automatic Backwash Filter

The automatic backwash filter or traveling bridge filter has a relatively shallow sand depth of 12 inches. The width of the unit is typically fixed at approximately 16 feet. The length of the traveling bridge filter is determined by the amount of surface area required for a given application. Wastewater is applied to the top of the sand and filters downward through the filter. The head loss across the filter is relatively low at less than 4.9 feet (WEF and ASCE 1998, pp. 16–19). A traveling bridge and backwash hood move along the filter to backwash one cell of the filter at a time. This allows the remaining filter cells to stay in operation while a portion of the filter is backwashed. The rate of backwash water generated is less because of the shallow sand bed and smaller area backwashed when compared to a conventional sand filter. In a conventional filter, the backwash might need to be stored to allow the solids to be fed slowly back to the head of the process to avoid slug loads. A filter then removes solids that pass through the tertiary clarifier. The traveling bridge filter produces a relatively constant amount of backwash when the filter is in operation.

Cloth Filters

Cloth filters, as the name implies, use specially designed cloth to filter the wastewater, rather than sand or other granular media. The cloth panels are installed vertically inside a steel or concrete tank. The wastewater submerges the cloth panels and travels horizontally through them. Solids accumulate on the outside of the cloth panels, while filtered water is collected on the inside of the panels and directed to the effluent chamber. The solids on the outside of the cloth form a mat, and the water level in the filter rises. When the water reaches a preset level, the filter is backwashed by liquid suction. The cloth filters are rotated during the backwash process. Two cloth filter panels are backwashed at a time, allowing the other panels to continue filtering water, thereby eliminating the need to have a tank to store flows for backwashing the filter. The backwashed solids are directed to the headworks of the plant. Larger solids settle to the bottom of the basin, from which they are periodically pumped out and directed to the headworks of the plant or the solids-handling process.

Cloth filters can operate at a higher hydraulic loading rate than granular media filters, resulting in a smaller footprint. The backwash rate is also reduced because there is no need to fluidize the bed as required with granular media filters. Cloth filters are usually installed in small plants (average flows less than 5 to 10 MGD); however, the Orange County, Florida, WWTP uses cloth filters at an average annual flow of 29.5 MGD and plans to expand to accommodate 43 MGD.

Tertiary Clarification with Filtration

The practice of adding tertiary clarifiers upstream of filters can further achieve low solids concentrations and thus low phosphorus effluent levels. Scott and Laurence (2007) performed a pilot study using the combination of tertiary clarification followed by filtration to obtain effluent concentrations in a municipal wastewater consistently below 0.05 mg/L, and sometimes below 0.01 mg/L, when using alum doses of 75 mg/L with polymer addition. Tertiary clarifiers that could be used include solids contact clarifiers, upflow buoyant-media clarifiers, tube clarifiers, plate clarifiers, and a second set of secondary clarifiers. To improve performance through the tertiary clarifiers, a coagulant, such as alum or ferric chloride, and a polymer can be added upstream of the unit.

Solids contact clarifiers mix the secondary effluent with coagulants and previously settled solids, forming larger floc. After mixing in the center of the clarifier, the wastewater moves outward to the settling zone, where the solids move downward to the bottom and the treated water exits the unit over a weir. Periodically, solids are removed from the clarifier for treatment and disposal. A filter then removes solids that pass through the tertiary clarifier.

The upflow buoyant-media clarifier mixes the coagulant and polymer with the secondary effluent, allows flocculation to occur, and provides clarification. A tube clarifier has inclined tubes in a portion of the clarifier. The water flows up through the tubes, and solids flow downward to the bottom of the clarifier, from which they are pumped out. Similarly, a plate clarifier has inclined plates installed in a portion of the clarifier. The water flows upward between the plates, and solids settle onto the plates and slide down to the bottom of the clarifier. For the tertiary clarification process to be successful, the velocity through the unit must be low enough to allow the solids in the secondary effluent to settle.

Membrane Filtration Technologies

Membrane filters can be used externally to remove suspended solids or can be incorporated into the activated-sludge process as an MBR. MBR systems employ a suspended-growth biological reactor, from which effluent is passed through a membrane filter. By so doing, suspended solids are effectively removed from the effluent. Phosphate is retained in the reactor as polyphosphate taken up by the microorganisms (if biological phosphate removal is operating) or as chemically generated suspended solids. Crawford et al. (2006) examined phosphorus removal at several U.S. plants, including one in Traverse City, Michigan. In all configurations, the membrane is associated with the final aerobic step, and microorganisms are recirculated with a portion of the membrane concentrate. If chemicals are needed to achieve low effluent phosphorus, they can be added before the membrane to precipitate whatever the microorganisms have not taken up. At the Hyrum WWTP in Utah, an MBR produced effluent concentrations of 0.07 mg/L for the annual average. The facility uses an aluminum salt for coagulation and chemical phosphorus removal. Another MBR facility at Lone Tree Creek, Colorado, achieves an annual average of 0.027 mg/L.

Specialty Filter Descriptions

Dynasand D2 Advanced Filtration System

The Dynasand D2 advanced filtration system consists of deep bed and standard bed upflow filters in series. The deep bed filter contains coarse sand and uses a proprietary process called continuous contact filtration, which allows coagulation, flocculation, and separation to occur in the filter. The standard bed filter is filled with a finer sand mix. Both filters are continuously backwashed. The backwash water from the filters is treated in a lamella gravity settler, a high-rate gravity plate settler, to remove solids before being returned to the head of the plant. Alum is added upstream of both filters. The process is depicted in Figure 2-18.

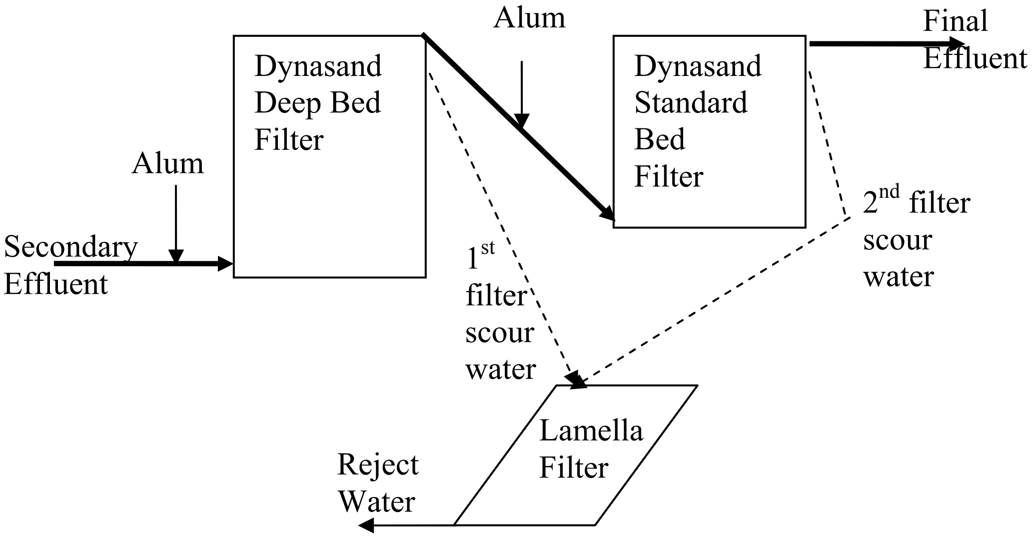


Figure 2-18. Parkson Dynasand D2 advanced filter system.

Actiflo

Actiflo is a sand-ballasted flocculation process. Metal salt and polymer are added upstream of the coagulation tank. The pH is adjusted to optimize phosphorus removal on the basis of the wastewater characteristics at the specific site and the type of polymer used. The wastewater is then mixed with fine sand and polymer. The fine sand, referred to as microsand, provides a large surface area to which the formed floc can attach; it also increases the sedimentation rate by acting as ballast. The solids are settled in a clarifier equipped with lamellar tubes. The microsand is recovered in a cyclonic separator and returned to the process. A sand filter can follow the Actiflo process to capture additional solids and further reduce the effluent phosphorus concentration.

DensaDeg

DensaDeg is a high-rate solids contact clarification process that consists of a reactor zone, a presettling/thickening zone, and a clarification zone. Metal salts are mixed with the influent to the process, and the pH is adjusted to optimize phosphorus removal based on the wastewater characteristics at the specific site. The wastewater then enters the base of the reactor and is mixed with sludge returned from the solids contact clarifier. The reactor tank contains a turbine and draft tube that promote floc formation and separate the solids. Polymer is added to increase the sludge density. In the presettling/thickening zone, the sludge settles to the bottom because of the increased density and continues to thicken. Sludge is returned to the reactor zone or removed from the process for further treatment. In the clarification zone, the supernatant flows up through the settling tubes as the effluent from the process. Additional phosphorus can be removed by following the process with filtration.

Enhanced Biological Phosphorus Removal with Filtration Technologies

Chemical addition, filtration, or both can be used to polish effluent from biological phosphorus removal processes. Drury et al. (2005) reported chemical polishing of EBPR effluent ahead of the tertiary filter at Clark County, Nevada. An upper feed limit was established at 10 mg/L alum to prevent plugging in the filter, while the effluent phosphorus concentration was regularly as low as 0.1 mg/L. At Durham, Oregon, Stephens (2004) reported effluent concentrations as low as 0.07 mg/L with chemical polishing and filtration. Any of the biological phosphorus removal technologies described previously could be combined with chemical addition, conventional filtration, membrane filtration (external to the activated-sludge process), or the specialty filtration processes mentioned earlier.

Emerging Technologies

The following processes have undergone testing, treating a portion of the flow at a minimum of one plant. Full-scale data are not available for these processes, although the processes are being considered for upgrades at several facilities.

CoMag Process

The CoMag process consists of ballasted flocculation, solids contact clarification, and high-gradient magnetic separation. The flocculation tank has three compartments. In the first compartment, wastewater is mixed with a metal salt and the pH is adjusted to optimize phosphorus removal on the basis of the wastewater characteristics at the specific site. Then fine magnetic particles are added to increase the density of the floc. In the third compartment, polymer is added to increase flocculation. The wastewater enters a solids contact clarifier. Most of the solids are returned to the flocculation tank. The remaining solids are wasted from the system. A magnet separator captures solids that passed through the clarifier by attracting the magnetic particles that were added in the second compartment of the flocculation tank. The effluent is then sent to the disinfection process for the treatment plant. The backwash from the magnet separator is mixed with the wasted sludge, which passes through a magnetic ballast recovery system to minimize the loss of the magnetic ballast. The remaining sludge is sent to the sludge-handling system for the plant. The recovered magnetic ballast is returned to the flocculation tank (Tozer 2007). The CoMag process is illustrated in Figure 2-19.

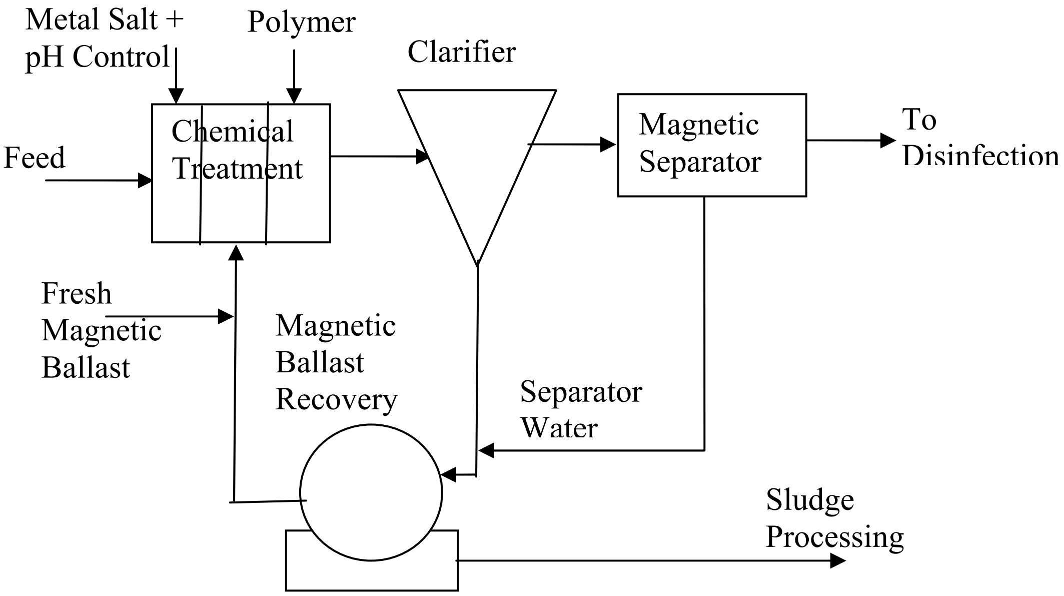


Figure 2-19. CoMag process.

Blue PRO Process

The Blue PRO process consists of addition of a chemical, typically ferric chloride to form ferric phosphate precipitate. This step is followed by a proprietary pre-reactor zone and moving-bed filter. The process uses a Centra-flo continuous backwashing filter. Unlike most filters, which rely on trapping solids between the media particles to remove phosphorus, the Blue PRO filtration system contains a bed of hydrous ferric oxide-coated media, in which ferric phosphate and other pollutants are filtered. The abrasion of the sand particles against one another in the moving bed filter exposes new adsorption sites on the media. The process can be operated in a dual-stage mode with two Blue PRO filtration systems in series. In the Blue PRO-CEPT system, the reject from the filters is returned to the head of the plant. The process is illustrated in Figure 2-20.

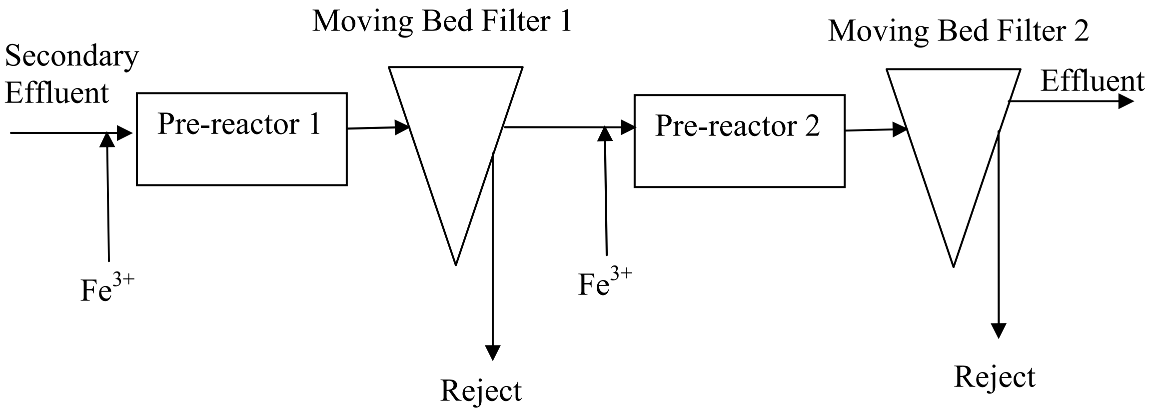


Figure 2-20. Blue-PRO process.

Trident HS

The Trident HS system consists of two clarification processes followed by filtration. Metal salts and polymer are added upstream of the tube clarifier, which contains a recycle flow of precipitated solids to decrease the variation of the influent quality entering the unit. From the tube clarifier, additional polymer is added before the wastewater enters an adsorption clarifier. The adsorption clarifier consists of a buoyant-media bed to remove additional solids before filtration. The media in the clarifier do not have any adsorption properties. The unit is an upflow filter containing coarse media. The accumulated solids are flushed from the clarifier using air and water from the tube clarifier. A mixed-media gravity filter follows the two-stage clarification process for applications designed to meet phosphorus concentrations of less than 0.1 mg/L. The filter is backwashed using air and water simultaneously. The process is depicted in Figure 2-21.

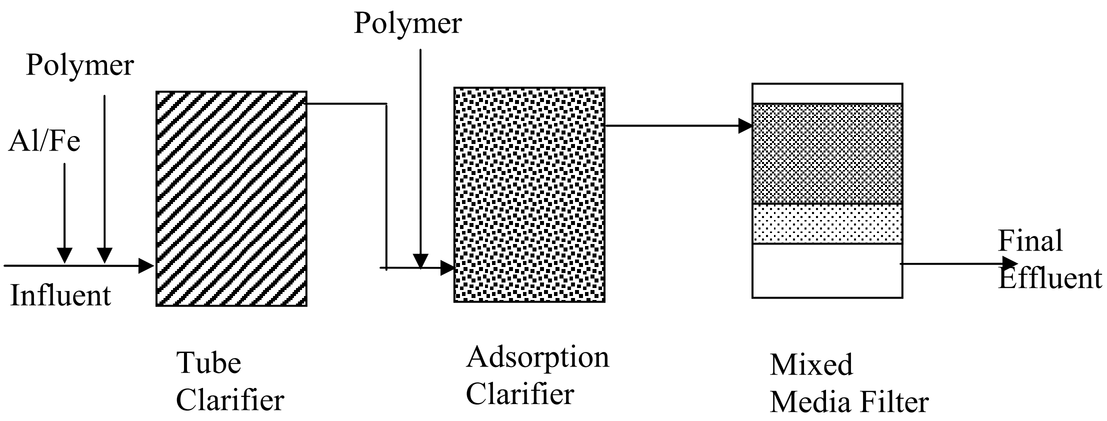


Figure 2-21. U.S. Filter Trident process.

Infiltration Basin/Land Application

Use of an infiltration basin or land application is the state of the science in phosphorus removal technology. Phosphorus is further removed through land application, before discharge to surface water. In addition, some soils are capable of adsorbing phosphorus, thereby extending the limit of technology for phosphorus removal. Land application requires a large amount of available space. A system designed for the Brighton, Michigan, plant employs approximately 25 acres/MGD treated. In that system, after biological treatment in an oxidation ditch, chemical treatment to 0.2 mg/L, and tertiary clarification, the wastewater is sprayed onto the land and collected through a system of underdrains. The treated water is then discharged to a surface water. The treated water has a phosphorus concentration of less than 0.01 mg/L at all times. The soil should be tested before constructing a land application system to verify the adsorption capacity of the soil and predict the length of time that the field can be used. In the case of Brighton, the design flowrate for the infiltration basin is 2.54 gal/day/ft^2. As designed, the soil had a cation exchange capacity of 2.0 milliequivalents/100 g; with the initial phosphorus content in the soil, the mini-column test results presented in the engineering report indicate an estimated useful life of the soil of at least 40 years.

2.4 Nitrogen and Phosphorus Removal Processes

2.4.1 Nitrogen and Phosphorus Removal Factors

The removal factors for processes that achieve both nitrogen and phosphorus removal are a combination of the removal factors for processes that target removal of only one of the nutrients.

Balancing the various factors might be necessary to achieve adequate TN and TP removal. For instance, nitrates in the RAS stream might need to be minimized to promote biological phosphorus removal in the anaerobic zone. This is more likely to be an issue at plants with a single anoxic zone because the nitrates created after nitrification in the aeration zone do not have an opportunity to denitrify. To avoid this problem, the RAS could be held in an equalization tank before return to the anaerobic zone. The equalization zone might also reduce the DO in the RAS, which would further improve the performance of the anaerobic zone. The retention time, however, would have to be limited to prevent secondary release of phosphorus. Alternatively, the RAS could be diverted to the anoxic zone rather than to the anaerobic zone.

Plants achieving both nitrogen and phosphorus removal must be designed and operated to avoid the release of phosphate without either VFAs or nitrates present. Anaerobic or anoxic zones that are too big remove carbon and nitrates, resulting in the release of phosphate that is not taken up in the aerobic zone. Similarly, if nitrate removal has been enhanced, there might not be any way to prevent phosphorus release in the lower portions or deep in the sludge blanket of the secondary clarifier. If that happens in the sludge, the release might not immediately affect the effluent directly, but it could result in a buildup of phosphorus in the secondary treatment system that would eventually raise the final phosphorus concentration. This trade-off between nitrate and biological phosphorus removal requires well-considered design, analysis, and process control. As an alternative, nitrate removal can be made extremely efficient in the secondary process, and chemical precipitation and filtration can be used subsequently to remove phosphorus to an extremely low level.

There are several other factors to be aware of in a combined system. In general, the COD-to-TKN ratio determines which zones need to be large and which should be small. Oxygen in the feed to anaerobic or anoxic zones should be minimized because it will inhibit those reactions. Temperature can also affect the processes: under cold conditions, fermentation might not provide sufficient VFAs for the less temperature-sensitive phosphate uptake reaction, and denitrification can be slowed. The result would be reduced phosphorus and nitrogen removal. Because phosphate removal and nitrification consume alkalinity while denitrification supplies alkalinity, it could be that some alkalinity will have to be supplied to allow all processes to proceed normally. In addition, phosphorus is a necessary nutrient for microbial growth. If TP is reduced too low before a biological process like denitrification, the growth of the microorganisms could be inhibited, thereby requiring the addition of supplemental phosphorus.

The biological removal of phosphorus and of nitrogen compete for available carbon at certain plants with two anoxic zones, such as those in the 5-stage Bardenpho process. The available carbon in the influent or return side streams can be used for biological phosphorus removal and nitrification. However, some plants might require methanol or an alternative carbon source to be added to the second anoxic zone. Plants that are required to meet low effluent phosphorus limits (less than 0.1 mg/L) will likely require an alternative carbon source and tertiary filtration.

An evaluation comparing the 5-stage Bardenpho process and the 4-stage Bardenpho process might be warranted in situations that would require methanol to be fed for nitrogen removal, as well as alum or ferric chloride for additional phosphorus removal. In the 4-stage Bardenpho process, phosphorus could be removed only chemically. Depending on the operation of the aerobic zones, sufficient carbon could be available at the second anoxic zone to eliminate the need to feed methanol. The 4-stage Bardenpho process would have no additional sludge from the nitrogen removal process because methanol would not be added; however, there would be more sludge from the chemical phosphorus removal. By using chemical phosphorus removal only, some of the balancing act described previously between biological phosphorus and nitrogen removal would not be necessary, making operation of the plant easier. If methanol addition can be avoided, it can save some costs and O&M problems associated with its handling.

2.4.2 Nitrogen and Phosphorus Removal Technologies

Anaerobic/Anoxic/Oxic

The anaerobic/anoxic/oxic (A^2O) process consists of an anaerobic zone, an anoxic zone, and an aerobic zone. An internal recycle stream returns nitrates from the aerobic zone to the anoxic zone, as in the MLE process. RAS is recycled to the head of the anaerobic zone along with the secondary influent. This process thus allows for simultaneous nitrogen and phosphorus removal. With the inclusion of the anoxic zone, the concentration of nitrates in the return sludge is reduced (compared to the A/O process), meaning that the anaerobic process is more efficient. The existing activated-sludge basin can be modified to include the anaerobic and anoxic zones, assuming sufficient volume remains in the aerobic zone to perform nitrification. With two recycle streams, some piping and pumping are needed, but extra head might not be required depending on the hydraulics of the plant. With only one anoxic zone and if VFAs are sufficient, there might not be a need to supplement the carbon. With A^2O, as with all combined nitrogen-phosphorus biological systems, some phosphorus is taken up in the anoxic zone by the PAOs, and the sludge residence time in each zone must be sufficient to allow complete phosphate release or uptake. The clarifier also must be operated to regularly waste solids to avoid release of phosphate by the endogenous respiration of PAOs. The A^2O process is illustrated in Figure 2-22. The ranges for TN and TP removal technologies evaluated in this study are 7.3 mg/L to 9.0 mg/L and 0.025 mg/L to 0.98 mg/L, respectively (see Tables 2-1 and 2-5). These results are for effluent from a system such as that shown in Figure 2-22. Another facility using A^2O followed by chemical addition and filtration achieved TP concentrations of 0.13 mg/L annual average (see Table 2-2 in Section 2.5).

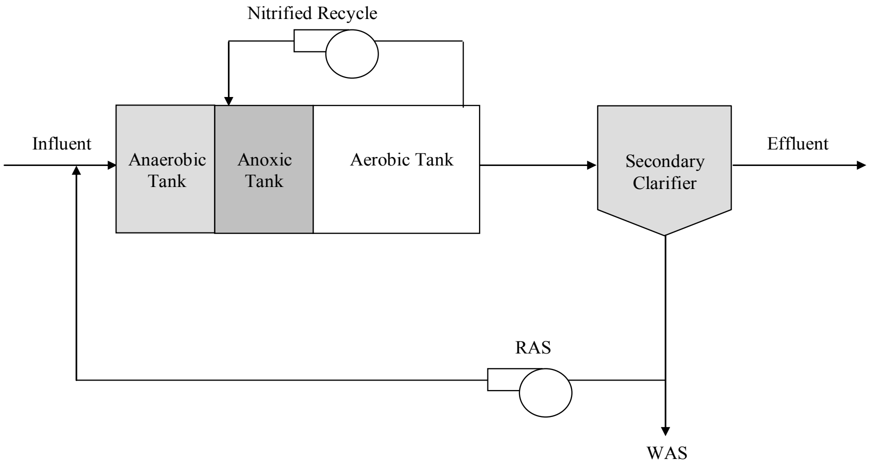


Figure 2-22. A^2O process.

Five-Stage Bardenpho Process

The 5-stage Bardenpho process is similar to the 4-stage Bardenpho process with the exception that an anaerobic zone is added ahead of the 4-stage system. The internal recycle from the first aerobic zone to the first anoxic zone remains in place. RAS is returned to the head of the anaerobic zone and is fed with the secondary influent (or primary effluent if primary settling is implemented). Methanol might need to be fed to the second anoxic zone to provide a carbon source for denitrification. As with the 4-stage Bardenpho, the footprint is large and internal construction is needed during retrofits to define the zones. Extra head is not typically needed; however, nitrate recycle between the first aerobic zone and anoxic zone might require additional pumping. The second anoxic zone could require additional carbon in the form of methanol or as VFAs if such are being added to support phosphorus uptake. For extremely low phosphorus concentrations, chemical addition, preferably in a downstream system so as to not remove all phosphate from the biological system, could be used. The 5-stage Bardenpho process is illustrated in Figure 2-23. The ranges of TN and TP effluent concentrations from the case studies that used 5-stage Bardenpho with chemical addition were 0.87 mg/L to 5.59 mg/L in TN and 0.06 mg/L to 1.09 mg/L in TP (see Table 2-3 in Section 2.5 for details).

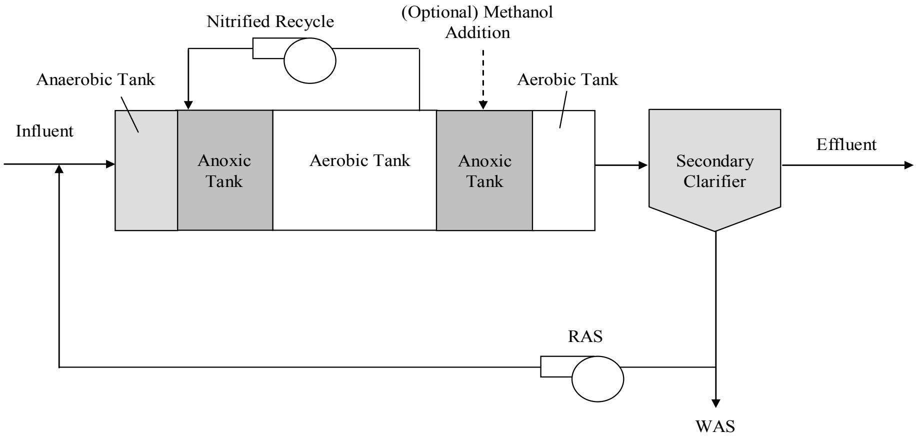


Figure 2-23. Five-stage modified Bardenpho process.

University of Cape Town Process

The University of Cape Town (UCT) process, a variation of the Phoredox process, consists of anaerobic, anoxic, and aerobic zones. An internal recycle returns nitrates from the aerobic zone to the head of the anoxic zone. A second internal recycle returns wastewater from the anoxic zone to the head of the anaerobic zone. RAS is directed to the head of the anoxic zone to minimize the amount of nitrates entering the anaerobic zone. The intent of the design is to keep the concentration of VFAs and the phosphate-accumulating reactions high, without competition from denitrification reactions using the VFAs. This process has a medium-sized footprint and could be set up in existing basins, depending on the volume available. As with other processes in this section, no additional head would be needed, but there would be extensive piping and pumping for the recycle streams. If sufficient VFAs are present, no supplemental carbon sources are required. Achieving a very low phosphate concentration requires downstream chemical precipitation and filtration. The UCT process is illustrated in Figure 2-24. The TN and TP effluent concentrations found in the literature for the UCT process with filtration were 8.9 mg/L to 10.0 mg/L in TN and 0.3 mg/L in TP (see Tables 2-5 and 2-8 in Section 2.5 for details).

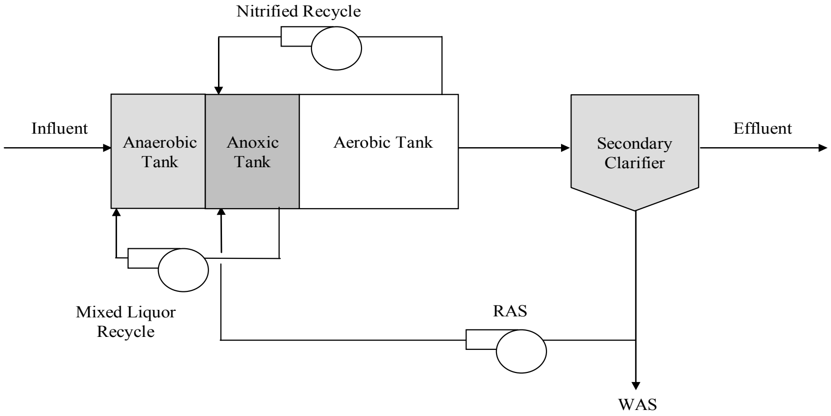


Figure 2-24. University of Cape Town process.

Modified University of Cape Town Process

The modified UCT process, another variation of the Phoredox process, has an anaerobic zone followed by two anoxic zones and an aerobic zone upstream of the secondary clarifiers. The two anoxic zones in series are designed to operate such that no nitrates are returned to the anaerobic zone. The nitrates from the aerobic zone are returned to the head of the second anoxic zone, while a second internal recycle returns flow from the end of the first anoxic zone to the head of the anaerobic zone. RAS is directed to the head of the first anoxic zone. This process has a medium-sized footprint and could be set up in existing basins if sufficient volume is available. As with other processes in this section, no additional head is needed, but extensive piping and pumping are needed for the recycle streams. If sufficient VFAs are present, no supplemental carbon sources are required. Achieving a very low phosphate concentration requires downstream chemical precipitation and filtration. The modified UCT process is illustrated in Figure 2-25. The ranges of TN and TP effluent concentrations found in the literature for the modified UCT process with VFA addition were 5.0 mg/L to 6.0 mg/L in TN and 0.1 mg/L to 2.7 mg/L in TP (see Table 2-8 in Section 2.5 for details).

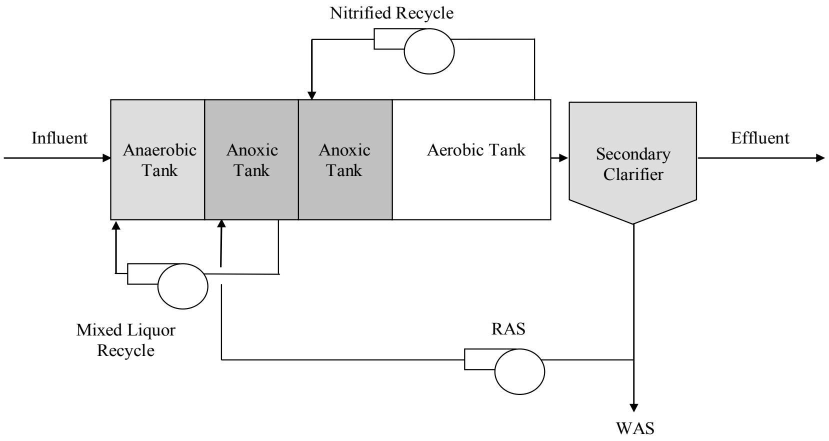


Figure 2-25. Modified University of Cape Town process.

Virginia Initiative Process

The Virginia Initiative process (VIP) is similar to the modified UCT process and is another variation of the Phoredox process. The nitrates from the aerobic zone are returned to the head of the first anoxic zone, instead of the second anoxic zone as is done with the modified UCT process. The second return is from the end of the second anoxic zone to the head of the anaerobic zone. RAS continues to enter the head of the first anoxic zone. The VIP process allows for additional denitrification and thus minimizes the introduction of nitrate to the anaerobic zone. Nitrate in the anaerobic zone would interfere with phosphorus release, and so would reduce the opportunity for subsequent phosphorus uptake in the aerobic zone. The VIP process is operated in a high-rate mode, allowing for small tank volumes, which require less space than other similar processes. This process has a medium-sized footprint and could be set up in existing basins. As with other processes in this section, no additional head is needed, but extensive piping and pumping are needed for the recycle streams. As with the other processes, if sufficient VFAs are present, no supplemental carbon sources are required. Achieving a very low phosphate concentration requires downstream chemical precipitation and filtration. The VIP is illustrated in Figure 2-26. The ranges of TN and phosphorus effluent concentrations found in the literature for the VIP were 3.0 mg/L to 10.0 mg/L in TN and 0.19 mg/L to 5.75 mg/L in TP; for the VIP with VFA addition the ranges were 5.0 mg/L to 10.0 mg/L in TN and 0.6 mg/L to 0.8 mg/L in TP (see Tables 2-1, 2-2, and 2-3 in Section 2.5 for details).

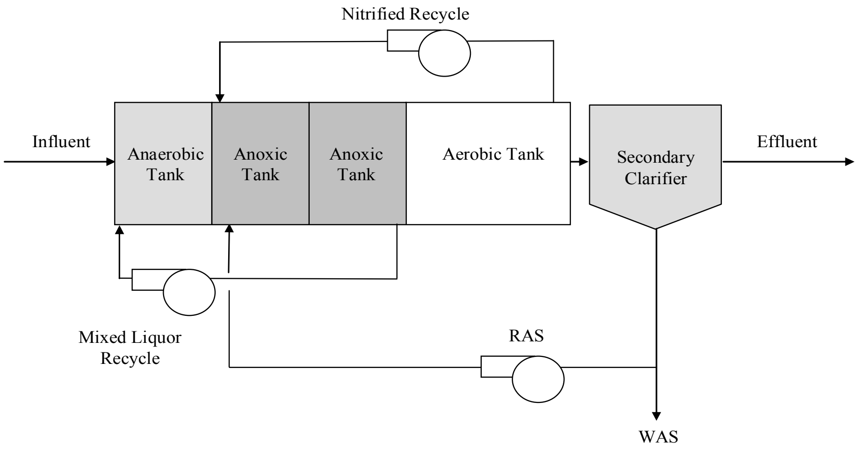


Figure 2-26. Virginia Initiative process.

Johannesburg Process

The Johannesburg process, another variation of the Phoredox process, consists of anaerobic, anoxic, and aerobic tanks in series. An internal recycle returns nitrates from the end of the aerobic zone to the head of the anoxic zone. An anoxic zone on the RAS line allows denitrification to occur, reducing the amount of nitrates that enter the anaerobic zone. Denitrification in the anoxic tank on the RAS line can be limited by a lack of carbon, which can be overcome by bringing sludge from the end of the anaerobic zone to the RAS-line anoxic zone. The dedicated anoxic zone allows a smaller footprint than some other systems. As with other processes in this section, no additional head is needed, but extensive piping and pumping are needed for the recycle streams. If sufficient VFAs are present, no supplemental carbon sources are required. Achieving a very low phosphate concentration requires downstream chemical precipitation. The Johannesburg process is illustrated in Figure 2-27. The range of TN for one application of the Johannesburg process was 2.03 mg/L to 11.44 mg/L TN (see Table 2-1 in Section 2.5 for details).

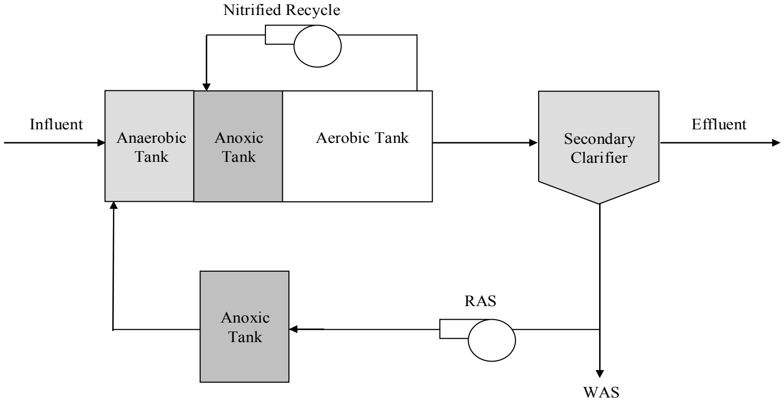


Figure 2-27. Johannesburg process.

Biodenipho Process

The PID, or Biodenipho, process is similar to the Biodenitro process, with the exception that an anaerobic tank is placed upstream of the two oxidation ditches, which are operated in a cyclical manner to promote denitrification and nitrification. The result of phasing (or cycling) is that organic carbon in the wastewater is used for both denitrification and biological phosphorus removal. If sufficient carbon is present, no supplemental source is required. The RAS is directed to the anaerobic zone. As with the Biodenitro process, the footprint of the system can be large. Achieving a very low phosphate concentration might require downstream chemical precipitation and filtration. The Biodenipho process is illustrated in Figure 2-24. The ranges of TN and TP effluent concentrations found in the literature for the Biodenipho process were 1.78 mg/L to 7.02 mg/L in TN and 0.09 mg/L to 1.99 mg/L in TP (see Table 2-3 in Section 2.5 for details).

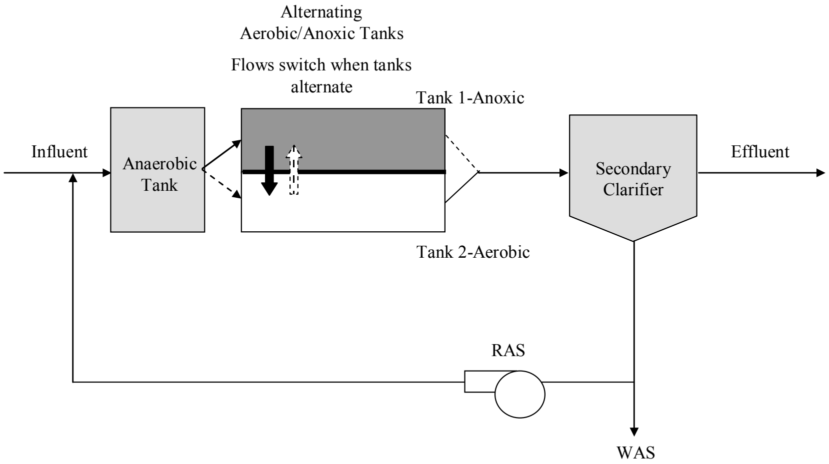


Figure 2-28. Biodenipho (phased isolation ditch) process.

Blue Plains Process

The Blue Plains process was a retrofit to the existing nitrification activated-sludge process at the Washington, DC, facility. A new anoxic zone was created inside the aeration tank with an HRT of 0.8 hour from the nominal 3.3 hours in the total basin. The design sludge age was 13 days. The existing return activated-sludge system remained unchanged in this retrofit. Methanol was fed directly into this new anoxic zone for a target nitrogen concentration of 7.5 mg/L (Kang et al. 1992; Sadick et al. 1998). Phosphorus is removed by ferric chloride addition and tertiary filtration. The Blue Plains process is depicted in Figure 2-29. The TN and TP effluent concentrations found in the literature for the Blue Plains process were 7.5 mg/L in TN and 0.12 mg/L in TP (see Table 2-1 in Section 2.5 for details).

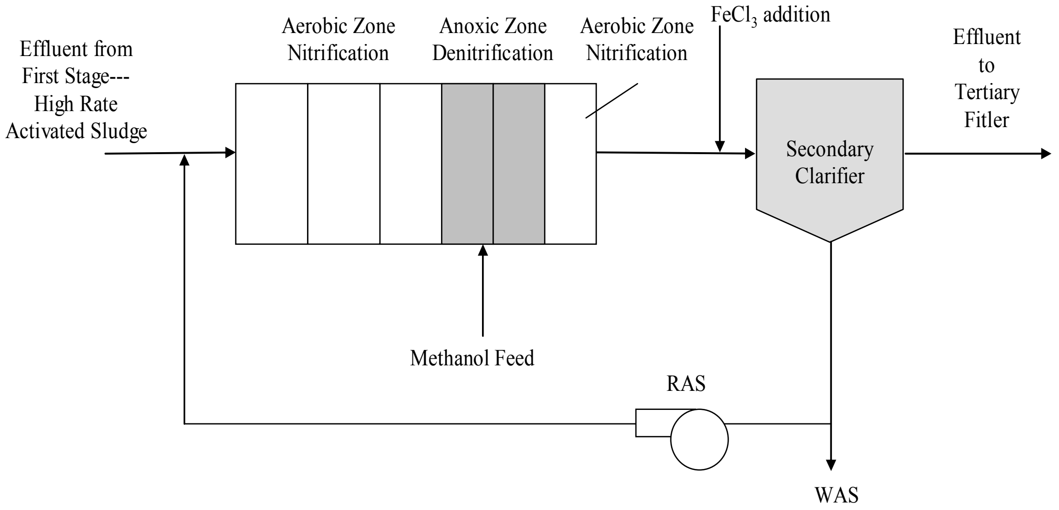


Figure 2-29. Blue Plains process.

Westbank Process

The Westbank process is a modification of a five-stage Bardenpho, with elimination of both the second anoxic zone and the reaeration zone. The process uses a step-feed arrangement for distributing primary effluent and fermenter supernatant (VFA-enriched) to the anaerobic and anoxic zones, as shown in Figure 2-30. The process consists of a small pre-anoxic zone, followed by an anaerobic zone, an anoxic zone, and an aerobic zone. The pre-anoxic zone minimizes the DO and nitrates entering the anaerobic zone, thereby maximizing the release of phosphorus. RAS is fed to the anoxic zone. Primary effluent is divided between the pre-anoxic zone (to denitrify the RAS), anaerobic zone (to stimulate phosphorus release), and anoxic zone (to stimulate denitrification). The direct feeding of the primary effluent to the anoxic zone increases the denitrification rate, thereby reducing the required size of the anoxic zone compared to that in a 5-stage Bardenpho system. The fermenter supernate, containing VFAs, is fed directly to the anaerobic zone. An internal recycle at a flow ratio of up to 600 percent directs the nitrates from the aerobic zone to the anoxic zone for denitrification. In the specific case of Kelowna, British Columbia, the effluent from the Westbank process is gravity filtered. The TN and TP effluent concentrations found during the case study period for the Westbank process with filtration ranged from 2.7 mg/L to 5.8 mg/L in TN, with an average of 4.4 mg/L in TN, and were 0.05 mg/L to 1.88 mg/L in TP, with an average of 0.14 mg/L TP (see Tables 2-1 and 2-2 in Section 2.5 for details).

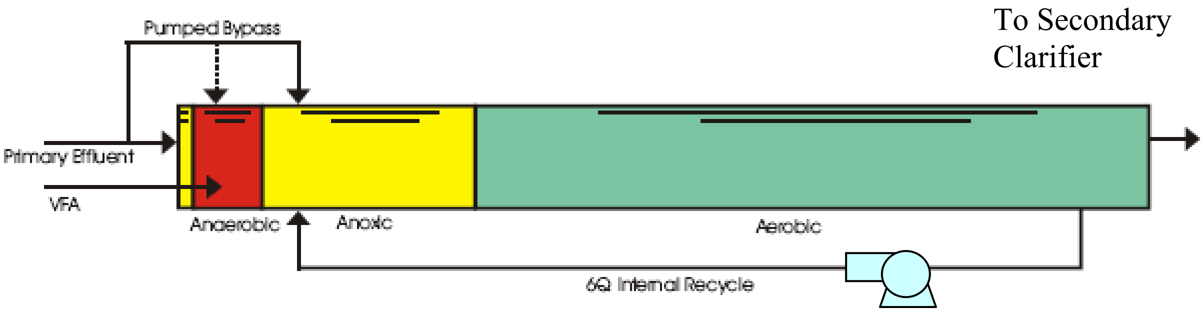


Figure 2-30. Westbank process (Kelowna, British Columbia).

2.5 Full-Scale Nutrient Removal Process Cases

2.5.1 Nitrogen Removal Matrix and Variability Data

The variability or reliability of nitrogen control technologies is summarized in Table 2-1. For plants for which a full year of daily nitrogen removal data were obtained, a statistical summary is presented for the annual average, maximum month, maximum week, and maximum day. These correspond to the points at 50, 92, 98, and 99.7 percent, respectively, when plotted on probability paper (based on the number of data points). This summary is the first known in the literature to compare full-scale technologies on an equal basis. Monthly, weekly, and daily maximums may be set by regulatory authorities for facilities that discharge to particularly sensitive waters. If TN effluent data were collected weekly because that was all that the permit required, the daily maximum value was not available. The performance levels shown from reported sources are documented in descending order for the technologies reported. It should be emphasized that, for this table as well as Tables 2-5 and 2-8, the performance results reflect specific operating philosophy, permit limitations, temperature, influent conditions, flow conditions, and the relative plant load compared to design. Thus, they do not necessarily represent optimum operation of the technologies presented. Most of the selected periods appeared to be typical; however, climate and weather variations could significantly affect performance. Similarly, the performance results reflected in the curves in Figures 2-31, 2-33, 2-34, 2-35, and 2-36 reflect site-specfic situations and do not necessarily represent optimum operation of the technologies.

Table 2-1. Process Performance Data: Nitrogen Removal—Plant Effluent

	Nitrogen removal							
TN (ppm)	Technology	Plant effluent observed value or range (mg/L)	Variability (mg/L)					Reference
			Std dev./ COV %	Annual average (50%)	Max. month (92%)	Max. week (98%)	Max. day (99.7%)	
10	Johannesburg	2.03 to 11.44	1.66/21	7.86	10.41	11.57	13.28	Hagerstown, Maryland
	A^{2}O	7.3 to 9.0[a]	--	--	--	--	--	Maryland Report[b]
	VIP	6.12[a]	--	6.12	--	--	--	Neethling 2005
	Step-feed AS		1.80/27	6.70	8.62	9.82	13.05	Cumberland, Maryland
5	IFAS	4.9 to 11.3[a]	--	--	--	--	--	Masterson 2004 McQuarrie 2004
	MBBR	5.8 to 6.8[a]	--	--	--	--	--	Täljemark et al. 2004
	MLE	2.2 to 15[a]	1.00/23	4.35	5.54	6.13	7.76	Leesburg Westminster, Maryland
	4-stage Bardenpho	3.5 to 12.1[a]	--	--	--	--	--	Maryland Report
	Schreiber system	8[a]	--	--	--	--	--	Maryland Report
	Blue Plains process	7.5[a]	--	--	--	--	--	Washington DC[c] (Kang et al. 1992) (Sadick et al. 1998)
	PIDs, clarifiers, aerobic digestion	1.8 to 7.0	0.51/14	3.67	4.46	5.87	--	North Cary, North Carolina[d]
	PIDs	1.4 to 11.3	1.81/42	4.2	7.3	11.3	--	Jewett City, Connecticut[c]
	SBR	1.6 to 13.6	2.31/50	4.59	6.84	10.68	14.35	Thomaston, Connecticut[c]
	Cyclic on-off	3.1 to 10.4	1.17/25	4.59	6.15	7.62	8.64	Ridgefield, Connecticut[c]
	Westbank	2.7 to 5.8	0.51/12	4.38	4.9	5.84	--	Kelowna, British Columbia[c,d]
	Step-feed AS	3.7 to 7.4	0.63/12	5.25	6.15	8.01	--	Fairfax, Virginia[c,d]

Table 2-1. Process Performance Data: Nitrogen Removal—Plant Effluent *(continued)*

	Nitrogen removal							
			Variability (mg/L)					Reference
TN (ppm)	**Technology**	**Plant effluent observed value or range (mg/L)**	**Std dev./ COV %**	**Annual average (50%)**	**Max. month (92%)**	**Max. week (98%)**	**Max. day (99.7%)**	
3	Biological aerated filters	1.4 to 6.8	2.24/62	3.61[e]	7.13[e]	9.80[a]	13.91[a]	Cheshire, Connecticut[c]
	Concentric oxidation ditch	1.6 to 5.4	0.95/32	3.0	4.24	5.29	6.46	Hammonton, New Jersey
	Step-feed AS	1 to 14	1.48/57	2.58	4.30	5.89	9.16	Piscataway, Maryland[c]
	5-stage Bardenpho	1.24 to 4.29	0.35/16	2.32	3.10	3.75	4.36	Clearwater, Florida-MS[d,f]
			0.86/42	2.04	3.10	3.90	5.44	Clearwater, Florida-NE[f]
	Denitrification filters	0.47 to 3.76	0.36/16	2.14	2.77	3.13	4.25	Johnston County, North Carolina [c,d]
	Denitrification filter	0.13 to 6.50	0.56/28	1.71	2.61	3.90	--	Lee County, Florida[d]
	Denitrifying activated sludge	0.4 to 10.4	0.59/36	1.63	2.46	4.22	--	Western Branch, Maryland[d]

Notes:

A^2O = anaerobic/anoxic/oxic

AS = activated sludge

COV = coefficient of variation

IFAS = integrated fixed-film activated sludge

MBBR = moving-bed biofilm reactor

MLE = modified Ludzak-Ettinger

PID = phased isolation ditch

SBR = sequencing batch reactor

VIP = Virginia Initiative process

Performance periods are listed in Attachment at end of References section.

[a] Data obtained from literature; data were not reviewed as part of project.

[b] George Miles & Buhr, LLC, and Gannett Fleming 2004.

[c] Retrofit application.

[d] Case study plant is explained in more detail in Chapter 3.

[e] Values are based on 8 months of data, rather than 12 months.

[f] Clearwater, Florida, has two facilities—Marshall Street (MS) and Northeast (NE)

The data reflect specific operating philosophy, permit limitations, influent conditions, flow conditions and the relative plant loadings compared to their design at these facilities. Thus, they do not necessarily represent optimum operation of the technologies presented.

Source: Table format adapted from WEF and ASCE 1998.

Figure 2-31 shows nitrogen removal technologies that have been demonstrated to produce low-range TN average results. Four technologies are shown in this group: denitrification filter, 5-stage Bardenpho, concentric oxidation ditch with a high recirculation rate, and step-feed activated sludge. All of these technologies produced effluent that was under 3 mg/L TN for an annual average, and each facility exhibited low variability. In addition, the monthly maxima (92 percent) points for all five technologies were under 5 mg/L; therefore, it is conceivable that all of these technologies could meet that value for a monthly maximum limit.

Table 2-2, as well as Tables 2-3, 2-4, 2-6 and 2-7, presents information about the case study facilities included in Figures 2-31, 2-33, 2-34, 2-35, and 2-36, respectively. These tables show the number of data points used in developing the referenced curves, temperature information, and average flow compared to design load. This number of data points show that the curve is based on a full year. The loading shows what fraction of the overall capacity was in use during the selected year. The temperature shows the range over which the plant was operated during the year. The reader is cautioned that these curves represent actual performance but not necessarily the optimal performance level for the given treatment technologies. This also means that the coefficient of variation (COV) could be higher or lower for different actual applications. Additional details, including permitted discharge limits, are presented in Chapter 3.

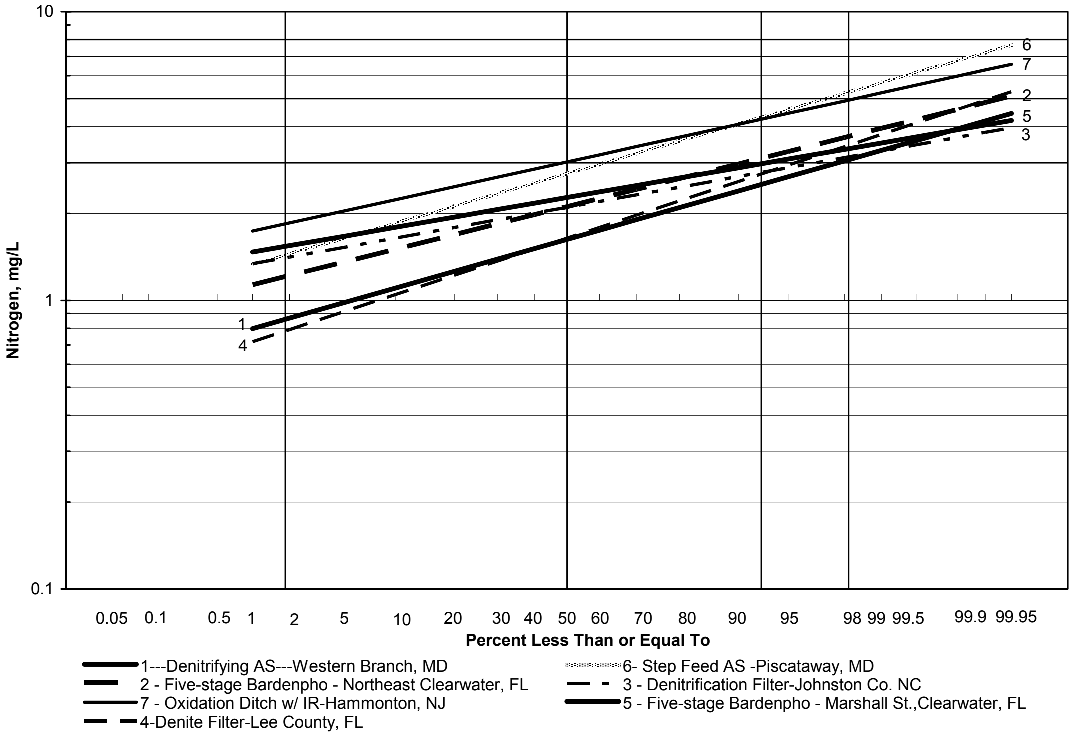


Figure 2-31. Monthly average frequency curves for TN: low-range removal.

Table 2-2. Detailed curve information for Case Study Facilities in Figure 2-31

Plant	# Data points	Min temp	Max temp	Ave. temp	Ave. flow MGD	Design flow MGD
Western Branch, MD	236	51 °F	76 °F	64 °F	19.3	30
Lee Co., FL	261	--	--	30.4 °C*	3.2	5.0
Central Johnston Co., NC	249	13.1 °C	27.9 °C	20.4 °C	4.1	7.0
Marshall St., Clearwater, FL	260	22.1 °C	31.1 °C	27.5 °C	5.5	10.0

* Only one available in July 2006.

The annual average and the maximum month concentrations, respectively, are shown for each facility:

1 = Denitrifying activated sludge: 1.63 mg/L and 2.46 mg/L with a COV of 36 percent at Western Branch, Maryland

2 =5-stage Bardenpho: 2.04 mg/L and 3.10 mg/L with a COV of 42 percent at the Clearwater, Florida, northeast plant

3 =Denitrification filter: 2.14 mg/L and 2.77 mg/L with a COV of 16 percent at the Central Johnston County, North Carolina, plant

4= Denitrification filter: 1.71 mg/L and 2.61 mg/L with a COV of 28 percent at the Lee County, Florida, plant

5 =5-stage Bardenpho: 2.32 mg/L and 3.1 mg/L with a COV of 16 percent at the Clearwater, Florida, Marshall Street plant

6 =Step-feed activated sludge (AS): 2.58 mg/L and 4.30 mg/L with a COV of 57 percent at the Piscataway, Maryland, plant

7 =Concentric oxidation ditch with internal recycle: 3.0 mg/L and 4.24 mg/L with a COV of 32 percent at the Hammonton, New Jersey, plant

A brief summary of the selected facilities that meet the low annual nitrogen limit of 3 mg/L is presented below. The 4-stage and 5-stage Bardenpho processes can achieve low effluent nitrogen concentrations because of the presence of two anoxic zones, allowing nitrates created in the first aeration basin to be denitrified. A review of literature results suggests that a 4-stage Bardenpho should do as well as or better than a 5-stage Bardenpho in nitrogen removal (deBarbadillo et al. 2003), but no facility was available for this investigation. A high internal recycle rate (four times the average flow) is also important with this process. A step-feed activated sludge process has proven reliable in nitrogen removal because of a new understanding of increased denitrification at the anoxic zone in the presence of primary effluent as a natural carbon source. Denitrification filters can also be used to achieve low effluent nitrogen levels. The process requires that methanol or an alternative carbon source be provided because the secondary treatment process usually has removed most of the BOD, which is needed to convert nitrates to nitrogen gas. In a concentric oxidation ditch, SND has been successful.

Clearwater, Florida

Clearwater, Florida, operates two 5-stage (one anaerobic zone followed by two anoxic and two aerobic zones) Bardenpho WWTPs, the Marshall Street and Northeast advanced pollution control facilities. The second anoxic zone provides an opportunity to denitrify the nitrates created in the aeration zone, allowing lower TN effluent concentrations to be achieved when compared to a plant with only one anoxic zone. Some 5-stage Bardenpho plants require that a carbon source, such as methanol, be fed to the second anoxic zone (the fourth reactor zone). However, neither of the plants in Clearwater requires methanol addition to achieve average TN concentrations below 3 mg/L. This is because of high internal recirculation, which aids efficient use of the carbon source in accomplishing denitrification (WEF 1998). The internal recycle rate is 400 percent of the average flow (4Q), which is relatively high. The RAS rate is approximately 90 percent at the Marshall Street plant and 110 percent at the Northeast plant. The food-to-microorganism ratios are relatively low—0.024 and 0.04—at the Northeast and Marshall Street plants, respectively.

Johnston County, North Carolina

The Central Johnston County Regional WWTP in North Carolina employs biological nitrogen removal through an MLE configuration followed by denitrification filters supplied by F.B. Leopold to achieve an annual average TN effluent concentration of 2.06 mg/L. The biological treatment consists of an anoxic zone followed by an aerobic zone. An internal recycle returns some of the nitrates from the aerobic zone to the anoxic zone for denitrification, as in the MLE process. Methanol is fed upstream of the denitrification filters at a methanol-to-nitrate ratio of 4.5 lb to 1 lb.

Piscataway, Maryland

The Piscataway Advanced Wastewater Treatment Plant in Accokeek, Maryland, has two step-feed activated sludge treatment trains to remove TN. The first train consists of five sets of anoxic and aerobic basins in series, with the capability to feed primary effluent to the first four anoxic zones. There are four sets of anoxic and aerobic zones in series. Primary effluent can be fed to the first three anoxic zones. RAS enters the first anoxic zone in both trains. The annual average TN concentration in the effluent from the Piscataway plant is 2.58 mg/L. Methanol or an alternative carbon source is not needed because primary effluent can be fed to each anoxic zone, with the exception of the last zone in each train. In addition, internal recycles are not necessary because of the multiple anoxic/aerobic zones in series. Sodium hydroxide is added upstream of the step-feed activated sludge process to increase the alkalinity of the wastewater. This is necessary since alkalinity is consumed as part of the nitrification process but only a portion is returned following denitrification.

Hammonton, New Jersey

The Hammonton WWTP has a concentric oxidation ditch consisting of three channels (Figure 2-32). The outer and middle channels are operated at a low DO concentration to promote anoxic conditions. Surface aerators provide aeration and mixing to limit anaerobic conditions. The inner channel is operated at a DO concentration of 2 to 2.5 mg/L to promote nitrification. The plant uses automatic DO control. If the DO is outside the programmed range, the number of aerators operating in each channel is automatically adjusted. If the DO is too high, an aerator is shut down; if the DO in the channel is too low, an aerator turns on. As with other oxidation ditches, the operating level of the tanks can be adjusted to provide additional HRT, as needed, within the operating limits of the aerators. The external RAS rate is 100 percent of the average flow (1Q). The unique aspect of the oxidation ditch is the internal recycle that directs mixed liquor from the inner channel to the outer channel. In the summer the internal recycle is 400 percent of the average flow (4Q). Because of lower temperatures in the winter, which affect the kinetics of the nitrification and denitrification processes, the internal recycle is increased to 500 percent of the average flow (5Q).

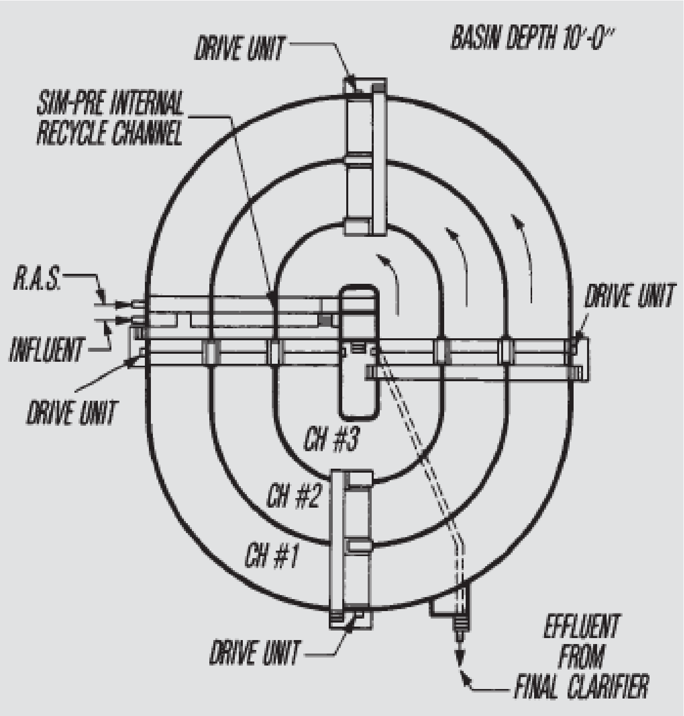


Figure 2-32. Concentric oxidation ditch.

Lee County, Florida

The Fiesta Village Advanced Treatment Facility employs oxidation ditches followed denitrification filters to achieve an annual average concentration of 1.38 mg/L with a COV of 40 percent. Methanol is fed upstream of the denitrification filters at a dosage of 3 lb methanol per lb of nitrate.

Western Branch, Maryland

This facility employs a unique denitrifying activated-sludge stage following two upstream activated-sludge stages, making a three-stage system overall. Methanol is fed upstream of the denitrifying stage. The annual average effluent concentration was 2.52 mg/L, with COV of 36 percent.

The next group of technologies is shown in Figure 2-33. These facilities have different TN permit limits, and some provide TN removal on a voluntary basis; thus, a wide variation is shown in their performance. These technologies could be extended to achieve effluent concentrations in the range of 3 to 8 mg/L on an annual average basis. Annual averages and the maximum month concentrations were as follows:

1 =Phased isolation ditch (Biodenipho or PID): 3.67 mg/L and 4.46 mg/L, COV of 14 percent, at North Cary, North Carolina

2 =Biological aerated filter (BAF): 3.61 mg/L and 7.13 mg/L, COV of 62 percent, at Cheshire, Connecticut

3 =Phased isolation ditch: 4.2 mg/L and 7.3 mg/L, COV of 42 percent, at Jewett City, Connecticut

4 =Westbank: 4.38 and 4.9, COV of 12 percent, at Kelowna, British Columbia

5 =Modified Ludzak-Ettinger (MLE): 4.35 mg/L and 5.54 mg/L, COV of 23 percent, at Westminster, Maryland

6 =Cyclic on-off activated sludge (AS): 4.59 mg/L and 6.15 mg/L, COV of 25 percent, at Ridgefield, Connecticut

7 =Sequencing batch reactor (SBR): 4.59 mg/L and 6.84 mg/L, COV of 50 percent, at Thomaston, Connecticut

8 =Step-feed activated sludge (AS): 6.70 mg/L and 8.62 mg/L, COV of 27 percent, at Cumberland, Maryland

9 =Johannesburg: 7.86 mg/L and 10.41 mg/L, COV of 21 percent, at Hagerstown, Maryland

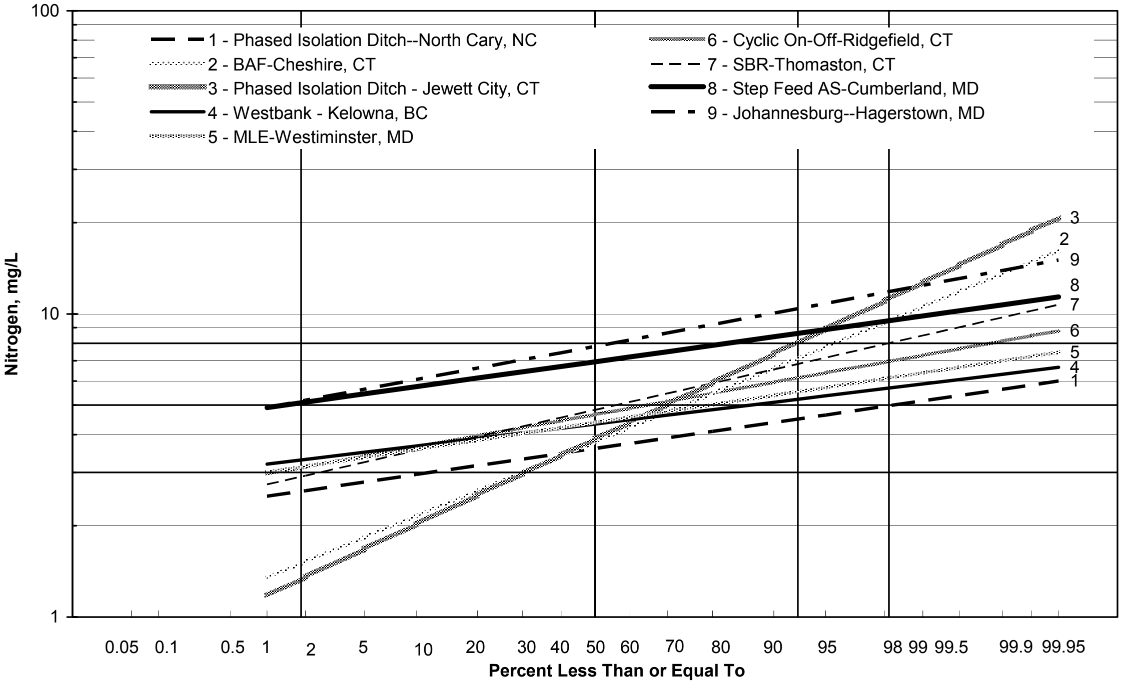


Figure 2-33. Monthly average frequency curves for TN—mid-range removal.

Table 2-3. Detailed curve information for Case Study Facilities in Figure 2-33

Plant	# Data points	Min temp	Max temp	Ave. temp	Ave. flow MGD	Design flow MGD
North Cary, NC	155	16 °C	27 °C	21 °C	7.0	12.0
Kelowna, BC	52	13 °C	22 °C	17.3 °C	8.5	10.6

Low variability was shown by the North Cary PID and the Westbank process, with COVs of 14 percent and 12 percent, respectively. Common elements of these two facilities were two anoxic zones and a good carbon supply for denitrification, preceded by an anaerobic zone for phosphorus removal. Although the other systems would meet an annual average limit of 5 mg/L, they would not meet a monthly maximum limit of 5 mg/L because of higher annual averages or higher COVs.

Some facilities have to meet ammonia nitrogen limitations without meeting a TN limit. In general, removal of ammonia nitrogen involves ensuring enough sludge residence time and hydraulic residence time to allow nitrification to occur in all seasons covered by the permit. Figure 2-30 reflects facilities that have been used to meet ammonia limitations, all of which are discussed further in the case studies in Chapter 3. The figure shows that all the facilities met their ammonia limits, ranging between 0.5 and 2.0 mg/L at all times, with relatively low COVs.

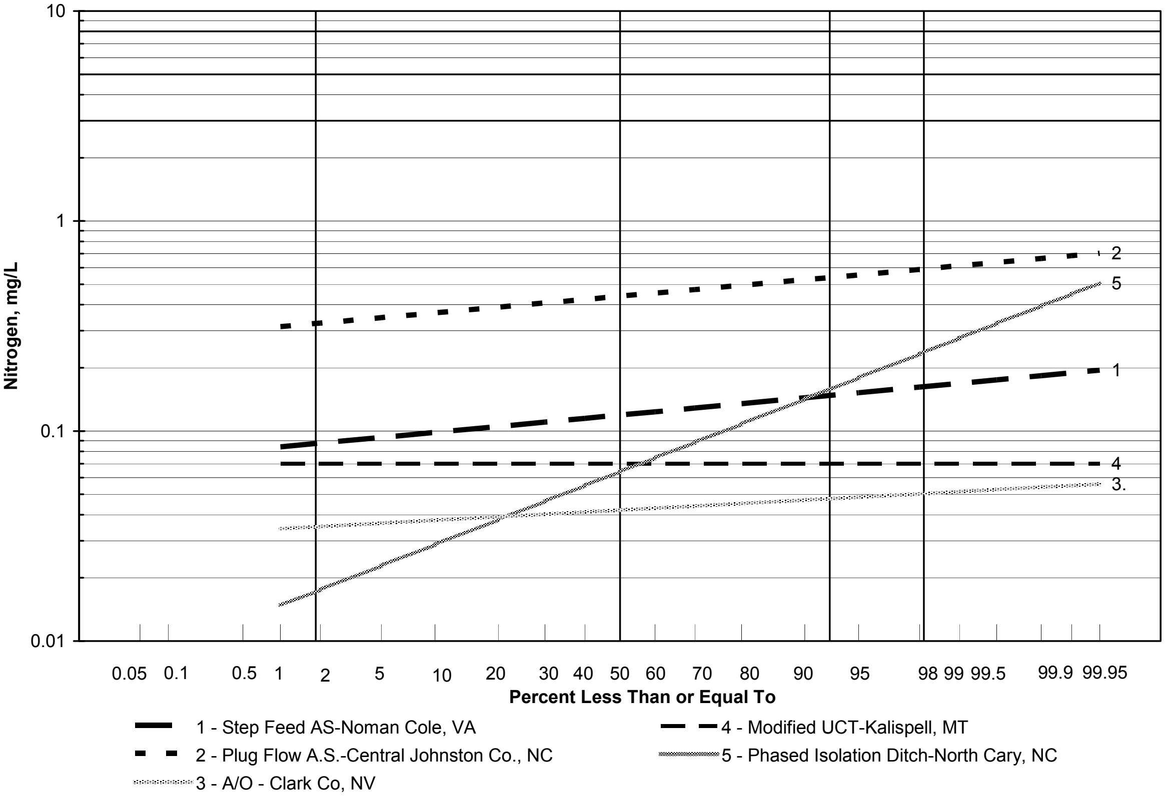


Figure 2-34. Monthly average frequency curves for ammonia nitrogen.

Table 2-4. Detailed curve information for Case Study Facilities in Figure 2-34

Plant	# Data points	Min temp	Max temp	Ave. temp	Ave. flow MGD	Design flow MGD
Clark Co., NV	365	17 °C	30 °C	25 °C	98	110
North Cary, NC	249	16 °C	27 °C	21 °C	7.0	12.0
Noman Cole, Fairfax Co., VA	365	14 °C	28 °C	21 °C	47.4	67
Central Johnston Co., NC	249	13.1 °C	27.9 °C	20.4 °C	4.1	7.0
Kalispell, MT	12	8.5 °C	19.6 °C	14.3 °C	2.9	3.1

* Only one available in July 2006.

2.5.2 Phosphorus Removal Matrix and Variability Data

Table 2-5 shows variability data for phosphorus removal technologies from 14 selected facilities. The figures following the table show variability data from well-operated full-scale facilities for a full year's operation. The results are shown in the order of descending effluent phosphorus concentration.

Table 2-5. Process Performance Data: Phosphorus Removal—Plant Effluent

	Phosphorus removal							
P (ppm)	Technology	Range of values observed (mg/L)	Variability (mg/L)					Reference
			Std dev./ COV %	Annual average (50%)	Max. month (92%)	Max. week (98%)	Max. day (99.7%)	
2								
1								
	A^{2}O with VFA addition, chemical addition, tertiary clarifier, and filtration[a]	0.025 to 0.98	0.044/33	0.132	0.18	0.646	0.98	Durham, Oregon[d]
0.5	VIP	0.19 to 5.0[c]	--	0.40	1.75	3.6	7.5	VIP, Neethling 2005
	A/O	0.03 to 0.43	0.12/50	0.24	0.36	0.44	0.75	Genesee County, Michigan[d]
	UCT with filter	0.3[c]	--	--	--	--	--	Penticton, British Columbia Barnard, 2006
	Westbank with fermenter and filters	0.05 to 1.88	0.03/21	0.14	0.20	0.25	--	Kelowna, British Columbia[b,d]
0.1	Chemical addition and flocculating clarifiers	0.07 to 0.23	0.01/14	0.09	0.11	--	--	Chelsea, Michigan
	PhoStrip	< 0.1 Ortho-P[c]	--	--	--	--	--	Truckee Meadows, Nevada Barnard et al. 2006
	Modified UCT with fermenter	0.03 to 0.37	0.023/19	0.12	0.15	0.31	0.36	Kalispell, Montana[b]
	Tertiary clarifier with chemical addition and filters[a]	0.026 to 0.24	0.036/63	0.058	0.12	0.17	0.23	McMinnville, Oregon[a]
	A/O with chemical addition to filters	0.03 to 2.3	0.03/30	0.10	0.17	0.41	0.56	Clark County, Nevada[b]
	Step-feed AS with fermenter and filter	0.02 to 0.26	0.02/21	0.09	0.12	0.16	0.26	Fairfax, Virginia[b,d]
	MBR	0.011 to 0.554	0.075/107	0.070	0.17	0.29	0.54	Hyrum, Utah
	Denitrification filter with alum	0.02 to 1.34	0.05/35	0.102	0.19	0.39	--	Lee County, Florida[b]

Table 2-5. Process Performance Data: Phosphorus Removal—Plant Effluent *(continued)*

	Phosphorus removal							
			Variability (mg/L)					
P (ppm)	Technology	Range of values observed (mg/L)	Std dev./ COV %	Annual average (50%)	Max. month (92%)	Max. week (98%)	Max. day (99.7%)	Reference
	5-stage Bardenpho oxidation ditch with chemical addition and filters	0.02 to 0.078	0.011/34	0.031	0.061	0.078	--	Pinery Water, Colorado
	MBR	0.01 to 0.083	0.0074/27	0.027	0.038	0.053	--	Lone Tree Creek, Colorado
	EBPR with high rate solids contact clarifier, chemical addition, and filters	to 0.02[c]	--	0.01	0.02	--	--	Breckenridge, Colorado
	Chemical addition, tertiary clarifiers, filter, infiltration basin	0.01 (monthly averages)	0/0	0.01	0.01	0.01	0.01	Brighton, Michigan

Notes:
A^2O = anaerobic/anoxic/oxic
A/O = anoxic/oxic
AS = activated sludge
EBPR = enhanced biological phosphorus removal
MBR = membrane reactor
UCT = University of Cape Town process
VFA = volatile fatty acid
VIP = Virginia Initiative process
Performance periods are listed in Attachment at end of References section.

[a] Seasonal permit limit of 0.07 mg/L; results based on only seasonal data.

[b] Subject of case study described in Chapter 3.

[c] Data obtained from literature; data were not reviewed as part of report.

[d] Retrofit applications.

The data reflect specific operating philosophy, permit limitations, influent conditions, flow conditions and the relative plant loadings compared to their design at these facilities. Thus, they do not necessarily represent optimum operation of the technologies presented.

Source: Table format adapted from WEF and ASCE 1998.

Figure 2-35 shows the best-performing TP removal processes, with the lowest phosphorus concentration in the effluent. All of these plants added chemicals to achieve their respective effluent phosphorus concentrations. Beginning with the lowest, the land application of tertiary effluent in Brighton, Michigan, had a flat line at 0.01 mg/L. The next lowest used MBR in Lone Tree Creek, Colorado, which had an annual average concentration of 0.027 mg/L, and a 5-stage Bardenpho with tertiary clarifier and Trident filter in Pinery, Colorado, which had an annual average of 0.031 mg/L. The annual average and maximum month concentrations are shown below with COVs.

1 = Land application: 0.01 mg/L for annual average and maximum month with COV of 0 percent, in Brighton, Michigan

2 = Biofor, DensaDeg, and MBR: 0.01 mg/L annual average and 0.02 mg/L monthly maximum, in Breckenridge, Colorado

3 = MBR: 0.027 mg/L and 0.038 mg/L, COV of 27 percent, in Lone Tree Creek, Colorado

4 = Five-stage Bardenpho, tertiary clarifier, Trident filter: 0.031 mg/L and 0.061 mg/L, COV of 34 percent, in Pinery, Colorado

5 = Tertiary clarifier/filter: 0.058 mg/L and 0.12 mg/L, COV of 63 percent, in McMinnville, Oregon (seasonal limit)

6 =MBR: 0.07 mg/L and 0.17 mg/L, COV of 107 percent, in Hyrum, Utah

7 = Denite filter: 0.10 mg/L and 0.19 mg/L, COV of 35 percent, in Lee County, Florida

Note that COV did increase to 107 percent and 63 percent for Hyrum and McMinnville. This was caused by changing conditions, especially temperature and water chemistry, through the year, which affected the precipitation/membrane filtration process in Hyrum and the effects of the seasonal limit for McMinnville. Although these two facilities can meet the 0.1 mg/L TP limit on an annual average basis, they might not meet the monthly maximum.

The second group of TP removal processes included EBPR with chemical addition and tertiary filters—Fairfax County, Virginia, and Clark County, Nevada. The annual average concentrations were 0.09 mg/L at both, and both were very reliable with the maximum month concentration below 0.2 mg/L. More description of selected facilities follows.

Achieving TP concentrations of less than 0.1 mg/L usually requires the use of chemicals (alum or ferric chloride) and filtration. Alum has a lower solubility limit than ferric chloride and is usually used in low-effluent phosphorus applications. Filtration can be provided by sand filters, denitrification filters, or membranes. For the phosphorus to be captured by

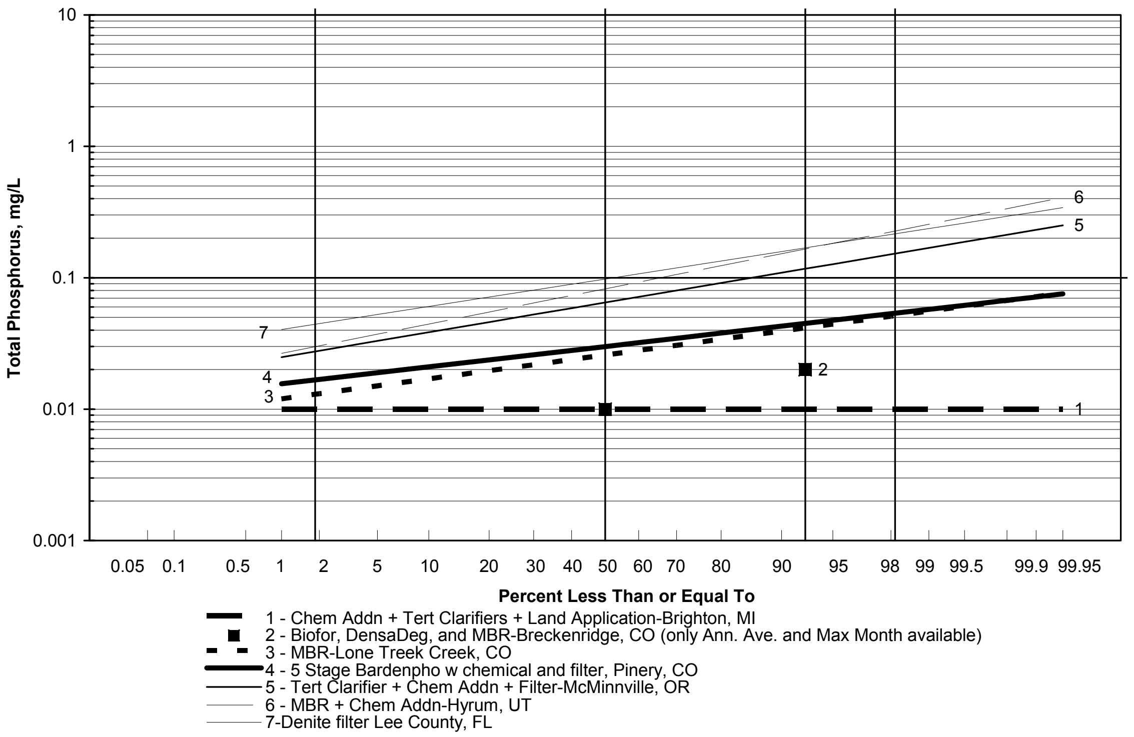


Figure 2-35. Monthly average frequency curves for TP—low-end removal.

Table 2-6. Detailed curve information for Case Study Facility in Figure 2-35

Plant	# Data points	Min temp	Max temp	Ave. temp	Ave. flow MGD	Design flow MGD
Lee Co., FL	261	--	--	30.4 °C*	3.2	5.0

* Only one available in July 2006

either filtration method, a sufficiently sized floc must be formed through coagulation with alum or other chemicals. To achieve effluent concentrations below 0.03 mg/L, more than one filtration step is required. For example, the Iowa Hill Reclamation Facility in Breckenridge, Colorado, has a DensaDeg high-rate solids clarifier (manufactured by Infilco Degremont) that uses sand particles to remove phosphorus floc, followed by additional chemical addition and a deep-bed sand filter. The lowest phosphorus concentrations were observed at the Brighton Environmental Control Facility in Brighton, Michigan, which uses chemical addition, claricones, and land application to an infiltration bed. The mechanism for phosphorus removal in the infiltration bed is adsorption to the soil particles. The water is collected from the infiltration beds by underdrains before discharge to the surface water. The process is very reliable, with a COV of 0 percent. It requires a large area, however, which might not be available in most places, particularly for retrofit applications.

McMinnville, Oregon

Phosphorus is removed through one-point chemical addition using alum and aluminum chlorohydrate upstream of the tertiary clarification, which is then followed by filtration. The seasonal average effluent concentration achieved is 0.058 mg/L. The permit limit applies from May through October. During the rest of the year, no phosphorus limit is in place and effluent concentrations are significantly higher. TP is not measured from November through April. Orthophosphorus concentrations as high as 3 mg/L have been recorded during this period.

Hyrum, Utah

The Hyrum WWTP has an MBR manufactured by Enviroquip/Kubota. Phosphorus is removed by adding alum to the activated-sludge tank. The floc that is formed does not pass through the flat-plate membranes, which take the place of the clarifiers and filtration in a conventional plant. The annual average effluent phosphorus concentration is 0.07 mg/L. The COV is relatively high at 107 percent.

Pinery Water, Colorado

The Pinery Water plant removes phosphorus biologically in the five-stage Bardenpho process, which is followed by a clarification and filtration process called Trident, manufactured by U.S. Filter. The Trident process involves tube clarification, adsorption clarification, and multimedia filtration. Alum is fed upstream of the tube clarifier to promote flocculation. The plant meets a monthly average permit limit of 0.05 mg/L.

Breckenridge, Colorado

The Iowa Hill Water Reclamation Facility achieves low effluent phosphorus concentrations, beginning with an anaerobic selector upstream of the activated-sludge process. A BAF follows. Then the wastewater enters a high-rate solids contact clarifier known as the Densadeg process, manufactured by Infilco-Degremont. The process involves flocculation and plate settling. The sand is recycled and reused in the flocculation process. The wastewater then passes through a Dynasand filter, manufactured by Parkson. The plant has a daily phosphorus limit of 0.05 mg/L and has reported effluent values below 0.01 mg/L.

Brighton, Michigan

The Brighton Environmental Control Facility’s process consists of secondary treatment, claricones, and infiltration basins. The secondary treatment process is an oxidation ditch, which provides little biological phosphorus removal. Ferric chloride is added to provide some chemical removal. Additional ferric chloride and a polymer are added to remove phosphorus in the claricones. The wastewater is then applied to a rapid infiltration bed. The phosphorus is adsorbed to the soil particles in the infiltration bed. The wastewater is then collected in underdrains for discharge to the surface water. Soil adsorption provides a very reliable treatment system, as long as the capacity of the soil is not exceeded. Although the process is capable of meeting a 0.01 mg/L TP effluent concentration, it requires a large amount of space, which might not be available in most areas. Furthermore, the soil in the vicinity of the treatment plant needs to allow water infiltration (sandy soils are needed; clay is not suitable). The land requirement for this facility was 25 acres per million gallons of capacity, which was sufficient to provide treatment for 40 years.

Figure 2-36 shows the mid-level phosphorus removal processes. The Piscataway Advanced Wastewater Treatment Plant (1) in Accokeek, Maryland, feeds alum and polymer to secondary clarification followed by a gravity mixed-media filter to remove phosphorus. The secondary treatment process is a step-feed activated sludge process. The annual average final effluent phosphorus concentration was 0.09 mg/L, and the maximum month was 0.20 mg/L. Lee County, Florida, achieved an annual average concentration of 0.102 mg/L with alum addition through a denitrification filter acting as a filter.

Treatment plants in Kalispell, Montana (2) and Kelowna, British Columbia (5) use EBPR with fermenters, producing annual average concentrations of 0.12 mg/L and 0.139 mg/L, respectively. Neither facility used chemicals, and yet both were very reliable in their performance, with maximum month concentrations of 0.15 mg/L and 0.2 mg/L, respectively. The COVs for Kalispell and Kelowna were very low at 19 percent and 12 percent, respectively.

The next facility removed phosphorus by using EBPR without fermenters with some alum addition. Clearwater, Florida (numbers 3 and 6 in Figure 2-36) added alum to the 5-stage Bardenpho facility and achieved effluent concentrations of 0.132 mg/L and 0.21 mg/L as the annual average and the maximum month, respectively.

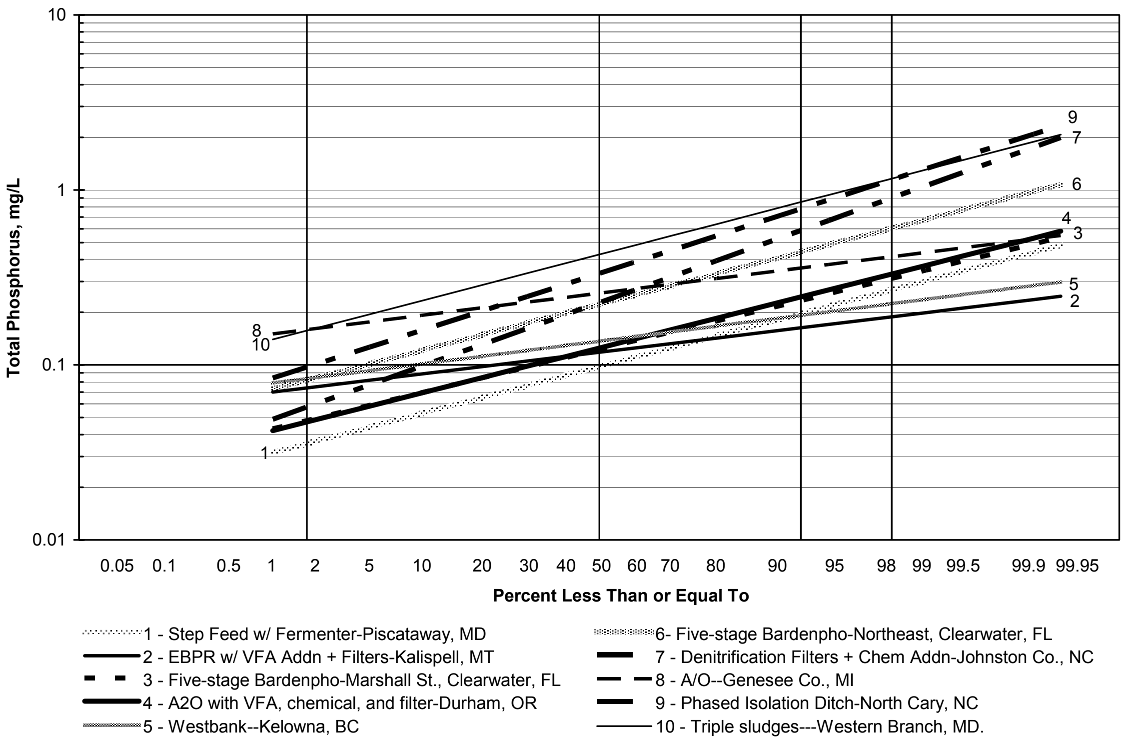


Figure 2-36. Monthly frequency curves for TP removal—mid-range removal.

Table 2-7. Detailed curve information for Case Study Facilities in Figure 2-36

Plant	# Data points	Min temp	Max temp	Ave. temp	Ave. flow MGD	Design flow MGD
Western Branch, MD	285	51 °F	76 °F	64 °F	19.3	30
Central Johnston Co., NC	52	13.1 °C	27.9 °C	20.4 °C	4.1	7.0
Marshall St, Clearwater, FL	260	22.1 °C	31.1 °C	27.5 °C	5.5	10.0
Kalispell, MT	104	8.5 °C	19.6 °C	14.3 °C	2.9	3.1
Kelowna, BC	51	13 °C	22 °C	17.3 °C	8.5	10.6
North Cary, NC	155	16 °C	27 °C	21 °C	7.0	12.0

* Only one available in July 2006

The next group of TP removal processes included EBPR without fermenters or chemical addition—Genesee County, Michigan; North Cary, North Carolina; Central Johnston County, North Carolina, and Western Branch, Maryland. Their annual average effluent concentrations were 0.24 mg/L, 0.38 mg/L, and 0.26 mg/L, respectively. The variability increased to varying degrees without fermenters at these facilities, and thus the maximum month concentrations were 0.36 mg/L, 1.0 mg/L, and 0.64 mg/L, respectively. No chemicals were added at these facilities, and the results were therefore more variable.

In summary, chemical precipitation with alum or ferric chloride has been proven to produce effluent concentrations below 1 mg/L in TP since the 1970s in the Great Lakes region based on the International Joint Commission agreement between the United States and Canada. Chemical precipitation followed by filtration likewise has been proved to produce effluent concentration between 0.5 and 0.1 mg/L in TP. If the permittee desires the lowest possible concentration with low variability, the following processes appear suitable—MBR, land application using infiltration beds, or step-feed activated sludge with fermenters. In addition, chemical addition with special filters is a technology that is emerging, and it is anticipated that full-scale data could be available in the near future. These special filters include the Dynasand D2 advanced filtration process, the Trident process, the Blue PRO filtration system, and the CoMag process. The second group, EBPR with chemical addition, appears suitable for compliance with a 0.5 mg/L limit. If the permittee desires EBPR only, EBPR with fermenters appears to be most suitable. EBPRs without fermenters offer higher variability but will meet the 1.0 mg/L permit limit.

2.5.3 Combined Nitrogen and Phosphorus Removal Matrix and Variability Data

To achieve TN concentrations below 3 mg/L and TP concentrations of less than 0.3 mg/L usually requires chemical addition and filtration for phosphorus removal because few plants are able to achieve effluent concentrations below 0.3 to 0.4 mg/L biologically. Therefore, the options for meeting simultaneous nitrogen and phosphorus removal are the same as those for TN removal, as long as chemical addition and filtration are added for phosphorus removal. In the extreme cases where the phosphorus limit is near 0.01 mg/L, Husband et al. (2005) reported the need to supplement phosphorus in support of full denitrification. A study by deBarbadillo et al. (2006) also recommended a phosphoric acid feed system if the growth rate of denitrifying heterotrophic organisms might be limited during high-nitrate loading periods. During the pilot testing, they observed that a filter influent orthophosphorus-to-nitrate nitrogen ratio of 0.02 or higher is safe.

North Cary, North Carolina

The North Cary WWTP in North Carolina is a PID. The Biodenipho process from Veolia Water/Krueger has two oxidation ditches, which are automatically controlled to operate in alternating aerobic and anoxic modes. An external anaerobic tank for phosphorus removal is configured upstream of the ditches. When aeration is not required, mixers are available to keep the mixed liquor in suspension. The anaerobic tank upstream of the ditch has a 2-hour detention time. The isolation ditch is operated in a 240-minute cycle. For the first 30 minutes, both ditches operate in an aerobic mode. For the next 90 minutes, ditch 1 operates in an anoxic mode while ditch 2 continues in an aerobic zone. A second 30-minute cycle follows with both ditches operating in an aerobic zone. In the last 90 minutes, ditch 1 continues to operate in an aerobic mode, while ditch 2 operates in an anoxic mode. This alternating strategy allows nitrification to occur, followed by denitrification while in the anoxic mode.

The low TN concentrations are achieved with a final anoxic tank that follows the PIDs; it is the second anoxic tank, as in the 5-stage Bardenpho process. The wastewater is then reaerated before the secondary clarifiers to prevent denitrification, causing rising sludge. The RAS rate is 55 percent of the average flow (0.55Q). The RAS enters a holding tank with a 30-minute detention time to minimize the amount of nitrates that enter the anaerobic tank. The sludge age is approximately 16 to 18 days. The secondary effluent passes through a deep-bed sand filter. Methanol can be fed to aid in denitrification, if needed; however, methanol has never been fed. The plant has the ability to add alum or ferric chloride to aid in phosphorus removal, but this has not been necessary for the past 2 years. The annual averages were 3.55 mg/L in TN and 0.38 mg/L in TP in the case study. The maximum month averages were 4.46 mg/L in TN and 1.06 mg/L in TP, without any chemical addition.

McDowell Creek, North Carolina

The McDowell Creek WWTP in North Carolina uses a modified UCT process. Sugar waste is added upstream of the anaerobic zone to provide an additional source of carbon for phosphorus removal. To supplement the influent alkalinity, lime is also added to ensure that nitrification rates will not be limited. The process has two internal recycle streams—from the second anoxic zone to the anaerobic zone and from the aeration zone to the first anoxic zone. The internal recycle from the end of the anoxic zone to the head of the anaerobic zone minimizes the nitrates that are returned. This eliminates competition of the nitrates with the PAOs for the VFAs, thereby improving biological phosphorus removal. The anoxic-to-anaerobic recycle is returned at a rate of 70 percent of the average flow (0.7Q), while the aerobic-to-anoxic recycle has a return rate of 320 percent of average flow (3.2Q). The RAS is returned to the first anoxic zone at a rate of 90 percent of average flow (0.9Q). The RAS enters at the head of the anoxic zone to minimize the presence of nitrates in the anaerobic zone.

A deep-bed sand filter follows the secondary clarifiers; it is designed to provide an opportunity for additional denitrification by microorganisms under anoxic conditions. The water passes through the filter, but no chemicals are added because of the ability to meet the loading limits biologically. Alum is fed to the belt filter press filtrate, which returns to the plant headworks. If the primary influent phosphorus concentration is elevated, additional alum can be fed to the primary clarifiers. A second feed point is available upstream of the secondary clarifiers, but it is not used. The McDowell Creek WWTP is achieving an annual average TP concentration of 0.1 mg/L biologically through VFA addition, with minimal chemical addition. The secondary effluent and final effluent TP values are approximately equal (the filter has little effect on the phosphorus concentration).

Clearwater, Florida

Clearwater, Florida, operates two 5-stage Bardenpho WWTPs, the Marshall Street and Northeast advanced pollution control facilities. The second anoxic zone provides an opportunity to denitrify the nitrates created in the aeration zone, allowing lower TN effluent concentrations to be achieved when compared to a plant with only one anoxic zone. Some 5-stage Bardenpho plants require that a carbon source, such as methanol, be fed to the second anoxic zone (the fourth reactor zone). However, neither of the plants in Clearwater requires methanol addition to achieve average TN concentrations below 3 mg/L. The internal recycle rate is 400 percent of the average flow (4Q), which is relatively high. The RAS rate is approximately 90 percent at the Marshall Street plant and 110 percent at the Northeast plant. The food-to-microorganism ratios for these plants were relatively low—0.024 and 0.04 at the Northeast and Marshall Street plants, respectively.

Phosphorus is removed biologically in the anaerobic zone. Alum is fed to the second anoxic zone (the fourth reaction basin) for the dual purpose of phosphorus polishing and meeting the state's additional requirement to meet low-level trihalomethane concentrations in the reuse water. In addition, a gravity sand filter is used to remove phosphorus tied to suspended solids in the final effluent. The annual average concentrations were 2.32 mg/L in TN and 0.11 mg/L in TP at the Marshall Street plant and 2.04 mg/L in TN and 0.20 mg/L in TP at the Northeast plant. Both plants operated with high efficiency and low variability.

Central Johnston County, North Carolina

The Central Johnston County Regional WWTP in North Carolina employs biological nitrogen removal followed by F.B. Leopold denitrification filters to achieve an annual average TN effluent concentration of 2.14 mg/L. The biological treatment consists of an MLE, with an anoxic zone followed by an aerobic zone. An internal recycle returns some of the nitrates from the aerobic zone to the anoxic zone for denitrification, as in the MLE process. Methanol is fed upstream of the denitrification filters using a methanol-to-nitrates ratio of 4.5 to 1.

No chemicals are added for phosphorus removal, although it is likely that some phosphorus is removed through the denitrification filters. The annual average concentrations were 2.14 mg/L in TN and 0.26 mg/L in TP. The maximum month concentrations were 2.77 mg/L in TN and 0.64 mg/L in TP, a very reliable nitrogen removal but less for TP.

Table 2-3 summarizes technologies to achieve both TN and TP removal. The results are shown in the order of descending effluent phosphorus concentration. COVs for TN and TP removal displayed similar patterns to plants that perform only TN or TP removal. That is, COVs were lower for TN and higher for TP removal. Nitrogen removal performance was efficient and reliable. COVs were below 50 percent for the 5-stage Bardenpho, denitrification filters, and the PID with anoxic tank. COVs for phosphorus, however, went high as the target concentration approached the critical level of 0.1 mg/L or below.

Research Needs

More research is needed to identify the cause of high variability when phosphorus concentrations are low and to develop strategies to improve performance. The solubility products of phosphorus compounds are known to be dependent on other chemical species and the pH in the wastewater. In addition, Banisch et al. (2007) reported nonreactive phosphorus in the effluent in the range of 0.01 to 0.12 mg/L. More research is needed to determine the minimal level of phosphorus needed to support healthy growth of nitrogen-removal organisms in facilities where removal of both nitrogen and phosphorus is required.

2.6 Summary

On the basis of an extensive review of the literature and actual data from 30 full-scale facilities, this chapter has presented a compilation of full plant performance on the same statistical basis. The following paragraphs summarize major findings on nutrient removal technologies.

2.6.1 Performance and Variability

A probability plot of actual data from full-scale operation is a good representation of performance and allows easy comparison of one technology against another on a consistent basis. Full-scale data generally follow a normal distribution. The coefficient of variation, or COV, is a good measure of statistical variability. The lower the COV, the more reliable the performance; conversely, higher COV values indicate more variability in performance.

Table 2-8. Process Performance Data: Nitrogen and Phosphorus Removal—Plant Effluent

	Nitrogen and phosphorus removal								
			Variability (mg/L)						
N (ppm)	Technology	Range of values observed (mg/L)	Std dev./ COV %	Annual average (50%)	Max. month (92%)	Max. week (98%)	Max. day (99.7%)	Reference	P (ppm)
10	Johannesburg	TN: 2.03 to 11.44 TP: 0.19 to 8.3	1.66/21 0.96/145	7.86 0.66	10.41 2.49	11.57 3.9	13.28 8.3	Hagerstown, Maryland	
	UCT	TN: 8.9 to 10[a] TP: 0.3						WEF and ASCE 1998	2 1
5	IFAS	TIN: 5.6 to 11.3[a] TP: 0.2 to 1.7[a]						Broomfield, CO McQuarrie 2004	0.5
	VIP	TN: 3 to 10 TP: 0.19 to 5.75						Rabinowitz 2004	
	VIP with VFA addition	TN: 5 to 10 TP: 0.6 to 0.8						Neethling 2005	
	Step-feed with fermenter	TN: < 5.0; 3 to 13 TP: < 0.3; 0.1 to 5	1.48/57 0.08/89	2.59 0.09	4.30 0.20	5.89 0.31	9.16 0.52	Piscataway, MD	
	Biodenipho/PID	TN: 1.78 to 7.02 TP: 0.09 to 1.99	0.93/14 0.27/64	3.67 0.38	4.46 1.06	5.87 1.45	6.79 1.78	North Cary, NC[b]	
	Modified UCT with VFA addition	TN: 5 to 6[a] TP: 0.1 to 2.7[a]		0.10	0.25	0.75	3.75	McDowell Creek, NC Neethling 2005	0.1

Table 2-8. Process Performance Data: Nitrogen and Phosphorus Removal—Plant Effluent *(continued)*

	Nitrogen and phosphorus removal								
			Variability (mg/L)						
N (ppm)	Technology	Range of values observed (mg/L)	Std dev./ COV %	Annual average (50%)	Max. month (92%)	Max. week (98%)	Max. day (99.7%)	Reference	P (ppm)
	High rate, nitrification, and denitrification activated sludge	TN: 0.4 to 10.4 TP: 0.05 to 1.88	0.59/36 0.27/62	1.63 0.43	2.46 0.89	4.22 0.99		Western Branch[b] Hyattsville, MD	
	5-stage Bardenpho with chemical addition for P removal	TN: 1.24 to 4.29 TP: 0.06 to 0.37	0.36/16 0.07/40	2.32 0.13	3.1 0.21	3.75 0.26	4.29 0.46	Clearwater, FL-MS[b]	
		TN: 0.87 to 5.59 TP: 0.06 to 1.09	0.86/42 0.16/82	2.04 0.20	3.10 0.44	3.90 0.63	5.44 1.07	Clearwater, FL-NE	
	Denitrification filters with chemical addition	TN: 0.13 to 6.50 TP: 0.02 to 1.34	0.56/28 0.05/35	1.71 0.102	2.61 0.19	3.70 0.39		Fiesta Village[b] Lee County, FL	
	Denitrification filters with chemical addition	TN: 0.84 to 3.13 TP: 0.1 to 1.01	0.36/16 0.09/62	2.14 0.26	2.77 0.64	3.13 1.01	-- --	Johnston Co., NC[b,c]	

Notes:

IFAS = integrated fixed-film activated sludge

PID = phased isolation ditch

UCT = University of Cape Town process

VFA = volatile fatty acids

VIP = Virginia Initiative process

Performance periods are listed in table at end of References section

[a] Data obtained from literature; data were not reviewed as part of report.

[b] Subject of case study described in Chapter 3.

[c] Retrofit application.

The data reflect specific operating philosophy, permit limitations, influent conditions, flow conditions and the relative plant loadings compared to their design at these facilities. Thus, they do not necessarily represent optimum operation of the technologies presented.

Source: Table format adapted from WEF and ASCE 1998.

2.6.2 Nitrogen Removal Technologies

Four technologies were identified to meet the low concentration limit with low variability. These are the 4- and 5-stage Bardenpho processes, the step-feed activated sludge process, concentric oxidation ditches, and denitrification filters with carbon sources. COVs for these processes are below 50 percent, and the technologies achieve annual average TN effluent concentrations below 3 mg/L. Seven technologies were identified for mid-level nitrogen removal between concentrations of 3 and 8 mg/L. The key factors that contribute to efficient and reliable nitrogen removal are an adequate supply of a carbon source (internal or external), flexibility in design (such as the number of anoxic/aerobic zones), temperature, alkalinity balance, and sludge age. DON has been reported in varying concentrations. The DON is a critical variable for determining TN standards because the chemicals have limited availability for biological removal.

2.6.3 Phosphorus Removal Technologies

Biological phosphorus removal (without filters or chemical addition) achieved an annual average effluent concentration of 0.26 mg/L with a COV of 35 percent. Excellent performance with low variability was reported for biological phosphorus removal processes, which achieved mean effluent concentrations down to 0.12 mg/L with a COV of 19 percent and 0.14 mg/L with a COV of 12 percent in another plant. The key performance factors included a sufficient supply of VFAs from an on-site fermenter, temperature, control of secondary phosphorus release, and good filtration. The practical TP limit of 0.1 mg/L is reached consistently with chemical addition and filtration. However, variability increases significantly at the target limit of 0.1 mg/L and below. The COVs increase to 93 percent for a plant achieving an annual average of 0.058 mg/L and to 107 percent for a facility meeting an annual average mean 0.07 mg/L. Special filters have proved effective in achieving low concentrations below 0.03 mg/L. They include the Trident filter from U.S. Filter, the Dynasand D2 advanced filtration system from Parkson, and membrane filtration processes from various manufacturers. The only technology evaluated that meets a TP effluent concentration of 0.01 mg/L at all times is land application of tertiary effluent through soil. The COV at the Brighton, Michigan, plant employing this technology is 0 percent in this case.

2.6.4 Combined Nitrogen and Phosphorus Removal Technologies

The TN and TP removal data show the same trends as the plants that performed only TN or TP removal. The nitrogen removal is as reliable as that described earlier, whereas the phosphorus removal shows higher COVs, indicating potential interaction between the solubility product at pH and other chemical species that occur throughout the year. Another factor is the nonreactive form of phosphorus in the wastewater, which might vary during the year. More research is needed to determine these chemical interactions. More research is also needed to establish the minimum phosphorus concentration needed to support nitrogen removal in the same process.

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Zimmerman, R.A., D. Richard, and J.M. Costello. 2004. Design, Construction, Start-Up, and Operation of a Full-Scale Separate Stage Moving Bed Biofilm Reactor Nitrification Process. In *Proceedings of the Water Environment Federation's 77th Annual Technical and Educational Conference*, New Orleans, LA, October 2–6, 2004.

Attachment 1: Locations Providing Data

Location	Data performance period
Brighton, Michigan	January through December 2005
Chelsea, Michigan	October 2005 through September 2006
Clark County, Nevada	January through December 2006
Clearwater, Florida: Northeast	October 2005 through September 2006
Clearwater, Florida: Marshall Street	October 2005 through September 2006
Cumberland, Maryland	January through December 2005
Cheshire, Connecticut	April through November 2006
Durham, Oregon	January through December 2004
Enfield, Connecticut	September 2005 through August 2006
Jewett City, Connecticut	September 2005 through August 2006
New Haven, Connecticut	September 2005 through August 2006
Ridgefield, Connecticut	September 2005 through August 2006
Thomaston, Connecticut	September 2005 through August 2006
Hagerstown, Maryland	January through December 2005
Genesee County, Michigan	October 1999 through September 2000
Hammonton, New Jersey	January through December 2005
Hyrum, Utah	November 2005 through September 2006
Central Johnston County, Smithfield, North Carolina	October 2005 through September 2006
Kalispell, Montana	July 2005 through June 2006
Fairfax County, Virginia	January through December 2006
Piscataway, Maryland	January through December 2005
McMinnville, Oregon	January 2005 through October 2006
Kelowna, British Columbia	January 2005 through December 2005
North Cary, North Carolina	October 2005 through September 2006
Pinery Water, Colorado	January through December 2004
Western Branch, Marlboro, Maryland	January through December 2006
Westminster, Maryland	October 2005 through September 2006
Lee County, Florida	January through December 2006
Breckenridge, Colorado	January through December 2003
Lone Pine Tree, Colorado	January through December 2006

CHAPTER 3: Case Studies and Reliability Factors

3.1 Introduction and Overview

The objective of the case studies was threefold: to study selected technologies that remove nitrogen, phosphorus, or both to low concentrations; to identify factors that contribute to reliability; and to identify factors that contribute to costs. Nine facilities were selected as case studies, and the results are summarized in this chapter. (The complete case studies are presented in Volume II, Appendix A).

In selecting the case study locations, consideration was given to ensuring that there was a variety of technologies that achieved low effluent concentration in either ammonia nitrogen and phosphorus or both total nitrogen (TN) and total phosphorus (TP), a variety of locations in cold- and warm-weather regions, and a variety of sludge-handling processes. The chapter is organized to present the specific nutrient removal technologies selected, followed by factors that influence the reliability of performance and costs, in both capital and operation and maintenance (O&M), at these facilities.

To establish a simple and yet sound statistical method by which plant performance and data can be presented and compared, the coefficient of variation (COV) was used in this study. The statistical background is included in Volume II, Appendix B.

3.1.1 Permit Limits for the Case Study Facilities

Table 3-1 summarizes the nutrient permit requirements for all the case study facilities. For the U.S. facilities, these values come from the National Pollutant Discharge Elimination System (NPDES) permits issued by the facility’s state regulatory authority, and they were current as of the writing of this manual in summer 2007. For the facility in Kelowna, British Columbia, the relevant limits were as given by Environment Canada.

Table 3-1 demonstrates that there was a wide variety in discharge seasons and numeric limits for the case study facilities. The facilities have standards ranging from annual averages to daily maximums. Total maximum daily loads (TMDLs) were used in some locations to develop load limits, while straight concentration limits were used as the basis for technologies in Florida. It is notable that limits for maximum week and maximum month were specified in one case, while the limits for maximum day and maximum month were specified in another. In another case, limits were set for maximum week, maximum month, and entire quarter. Clearwater, Florida, has limits for maximum week and maximum month and an annual limit.

Table 3-1. Discharge permit limits and performance data summary

Plant and location	Nutrient removal processes	Permit limits and performance results(mg/L)									
				Quarterly or annual		Month		Week		Max day	
		Annual average	COV for annual average	Avg permit limits	Max (q) Avg (a) result	Avg permit limits	Max month result	Avg permit limits	Max week result	Permit limit	Max day
Noman M. Cole Pollution Control Plant; Fairfax County, Virginia	Step-feed AS with dual media & deep bed filter + ferric chloride	TP = 0.09 NH_4-N = 0.12 TN = 5.25	TP: 21% NH_4-N: 14% TN: 12%	No quarterly or annual limits	--	TP = 0.18 Sum NH_4-N = 1.0 Win NH_4-N = 2.2 TN = report	TP = 0.12 NH_4-N = 0.135 TN = 6.0	TP = 0.27 Sum NH_4-N = 1.5 Win NH_4-N = 2.7 TN = report	TP = 0.16 NH_4-N =0.29 TN = 8.01	No daily limits	--
Kalispell Advanced Wastewater Treatment, Kalispell, Montana	Mod. UTC with fermenter and up flow sand filter (no chemical for P)	TP = 0.121 NH_4-N = 0.07	TP: 19 % NH_4-N: 0%	No quarterly or annual limits	--	TP=1.0 NH_4-N =1.4	TP=0.15 NH_4-N = 0.07	No weekly limits	--	No daily limits	--
Clark County Water Reclamation District, Nevada	A/O process with active primary for VFA with dual media filters + alum	TP = < 0.09 NH_4-N = 0.12	TP: 30% NH_4-N: 14%	No quarterly or annual limits	--	No monthly average limits	TP = 0.12 NH_4-N = 0.15	No weekly limits	--	TP= 0.22[a] NH_4-N = 0.64[a]	TP = 0.22 NH_4-N = 0.57
Central Johnston County Smithfield, North Carolina	Plug flow AS (anoxic, aeration) with denitrification filter and methanol (No chem. for P)	TP = 0.26 NH_4-N = 0.44 TN = 2.14	TP: 62% NH_4-N: 12% TN: 16 %	TP = 2.0 (Q) TN = 3.7**	TP = 0.47 (Q Max) TN = 2.14 (Av)	TP=1.0 Sum NH_4-N = 2.0 Win NH_4-N = 4.0	TP = 0.64 NH_4-N = 0.54	Sum NH_4-N = 6.0 Win NH_4-N = 12.0	NH_4-N = 0.86	No daily limits	--
North Cary, North Carolina	Oxidation ditch (Biodenipho) with upflow sand filter (No chemical for P)	TP=0.38 Ammonia N = 0.08 TN=3.7	TP: 64% Ammonia N: 102% TN: 14 %	TP = 2.0 (Q) TN = 3.94[b] (A)	TP = 0.57 (Q max) TN = 3.7 (Av)	Sum NH_4-N = 0.5 Win NH_4-N = 1.0	NH_4-N = 0.34	No weekly limits	--	No daily limits	--

Table 3-1. Discharge permit limits and performance data summary *(continued)*

		Permit limits and performance results(mg/L)									
				Quarterly or annual		Month		Week		Max day	
Plant and location	Nutrient removal processes	Annual average	COV for annual average	Avg permit limits	Max (q) Avg (a) result	Avg permit limits	Max month result	Avg permit limits	Max week result	Permit limit	Max day
Kelowna, British Columbia, Canada	3-stage Westbank with fermenters (No chemical for P)	TP=0.139 TN=4.38	TP: 21% TN: 12%	TP = 0.25 (A)	TP = 0.139 (Av)	No monthly average limit	TP = 0.20 NH_4-N =1.01 TN = 4.9	No weekly limits	--	TP = 2.0 TN =6.0 (daily max)	TP = 0.25 TN = 5.8
Marshall Street Water Reclamation Facility, Clearwater, Florida	5-stage Bardenpho with sand filter + alum	TP=0.132 NH_4-N=0.038 TN=2.32	TP: 40% NH_4-N: 18% TN: 16%	TP = 1.0 (A) TN = 3.0 (A)	TP = 0.132 (Av) TN = 2.32 (Av)	TP = 1.25 TN = 3.75	TP = 0.21 TN = 3.1	TP = 1.5 TN = 4.5	TP = 0.26 TN = 3.75	TP = 2.0 TN = 6 (daily max)	TP = 0.37 TN = 4.29
Lee County, Florida	AS with denitrification filter and methanol addition + alum	TP=0.102 TN=1.57	TP: 35% TN: 28%	TP = 0.5 (A) TN = 3.0 (A)	TP = 0.08 (Av) TN = 1.57 (Av)	TP = 0.5 TN=3	TP = 0.19 TN = 2.61	TP = 0.75 TN=4.5	TP = 0.39 TN = 3.70	TP = 1.0 TN=6 (daily max)	TP = 0.27 TN = 5.98
Western Branch, Upper Marlboro, Maryland	Three AS with methanol addition + alum	TP = 0.47 NH_4-N = 0.22 TN = 1.70	TP: 62% NH_4-N: 163% TN: 36%	TP = 0.3^c (A) TN = 4.0^c (A)	--	TP = 1.0 TN = 3.0 (Apr-Oct) NH_4-N = 1.5 (Apr-Oct), NH_4-N = 5.5 (Nov-Mar)	--	TN = 4.5 (Apr-Oct)	--	--	--

Notes:
A/O = anoxic/oxic
AS = activated sludge
COV = coefficient of variation: one standard deviation divided by the mean
NH4-N = ammonia nitrogen
NL = No limit
TP = total phosphorus
TN = total nitrogen
Sum = Summer
Win = Winter
Chem. = chemical
a Daily maximum permit limits were derived from a wasteload allocation on the basis of daily maxima at design flow
b Annual average permit limit derived from a wasteload allocation on the basis of annual average at design flow
c Annual average limits based on 30 MGD annual load following installation of Maryland-funded upgrades

The permit for Kelowna, British Columbia, also includes reliability criteria for phosphorus removal, which specified limits for the maximum day, 99th percentile, 90th percentile, and annual average, as follows:

Maximum day	2.0 mg/L TP
99th percentile	1.5 mg/L
90th percentile	1.0 mg/L
Annual average	0.25 mg/L

These Canadian limits are comparable to statistics as applied at U.S. facilities, as shown in Table 3-2. For example, the Canadian 90th percentile limit is slightly less stringent than the U.S. maximum month limit for data collected on a daily basis.

Table 3-2. Statistical comparison of Canadian and U.S. permit limits

Statistical percentile	U.S. term—averaging or maximum period	Canadian term—percent less than value
50	Annual average	50th percentile
80	Maximum quarter	--
90	--	90th percentile
92.3	Maximum month	--
98.1	Maximum week	--
99	--	99th percentile
99.7	Maximum day	--

3.2 *Summary of Case Studies*

Table 3-1 above shows the location of and technologies used in each of the case studies.

The highlights of each case study are presented in order, from facilities with the lowest permit limit to those with the highest permit limit. All facilities were required to remove varying levels of phosphorus. Two facilities had ammonia nitrogen permit limits but no TN limits, while the other seven facilities had TN limits, as shown in Table 3-1. This chapter presents the nitrogen and phosphorus removal facilities first, followed by the facilities required to remove ammonia nitrogen and phosphorus.

3.2.1 Total Nitrogen and Phosphorus Removal at Low Concentration Limits (3 mg/L or less in TN)

Four case studies fell into this group, and they are characterized as having multiple, distinct, anoxic zones with sufficient carbon supply.

Case Study No. 1: Western Branch, WSSC, Maryland

This facility has a three-stage activated-sludge system followed by tertiary filtration. The third activated-sludge stage was added to provide denitrification using methanol as the carbon source. The permit includes a monthly average ammonia limit of 1.5 milligrams per liter (mg/L) in summer (April–October) and 5.5 mg/L in winter (November–March). The TN limits are 3 mg/L as a monthly average and 4.5 mg/L as a weekly average, which both apply during the summer (April–October). An annual average TN limit of 4.0 mg/L is expected to go into effect in the future. The phosphorus limit for the future is 0.3 mg/L at the design flow of 30 million gallons per day (MGD).

The Western Branch Wastewater Treatment Plant (WWTP) is part of the Washington Suburban Sanitary Commission (WSSC) system, and it is in Upper Marlboro, Maryland. The plant employs three separate activated-sludge systems in series to accomplish biochemical oxygen demand (BOD) removal (high-rate activated sludge, or HRAS), nitrification (nitrification activated sludge, or NAS), and denitrification (denitrification activated sludge, or DNAS). The return activated sludges (RAS) are kept separate to facilitate the growth of specific types of microorganisms in each system by allowing for differing sludge residence times. After grit removal and screening, the wastewater goes directly to the HRAS. The effluent is filtered prior to ultraviolet (UV) disinfection. Waste activated sludges (WASs) from the three systems are combined, thickened by dissolved air flotation (DAF), dewatered by centrifuge, and burned in two multiple-hearth incinerators.

The plant does not have primary settling. Alum was added to remove phosphorus. The process recycle water from the DAF, centrifuge, and incinerator is returned to the headworks. The plant treated an annual average flow of 23.0 MGD, and the maximum month flow was 26.6 MGD.

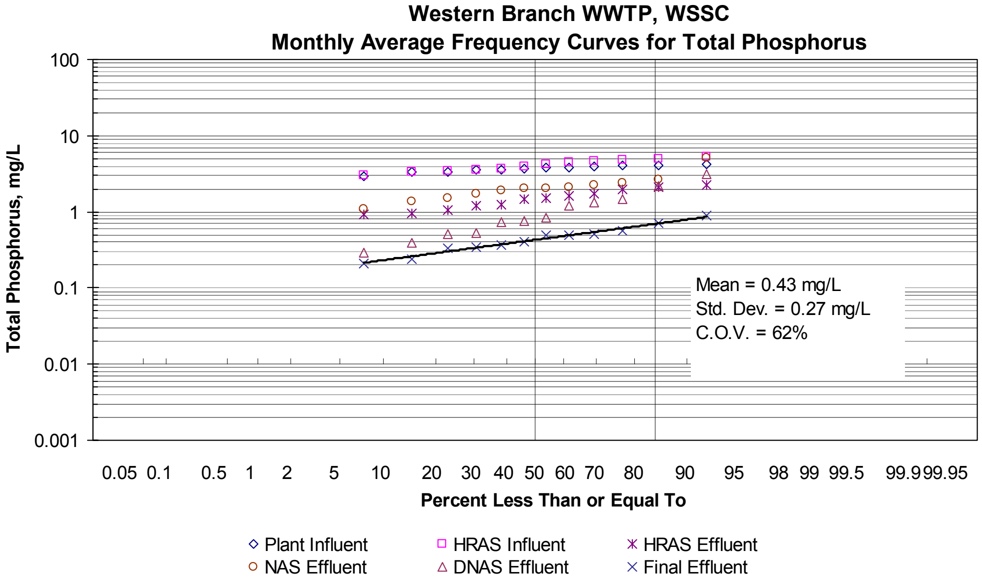

Figure 3-1. Western Branch WWTP: Monthly frequency curves for TP.

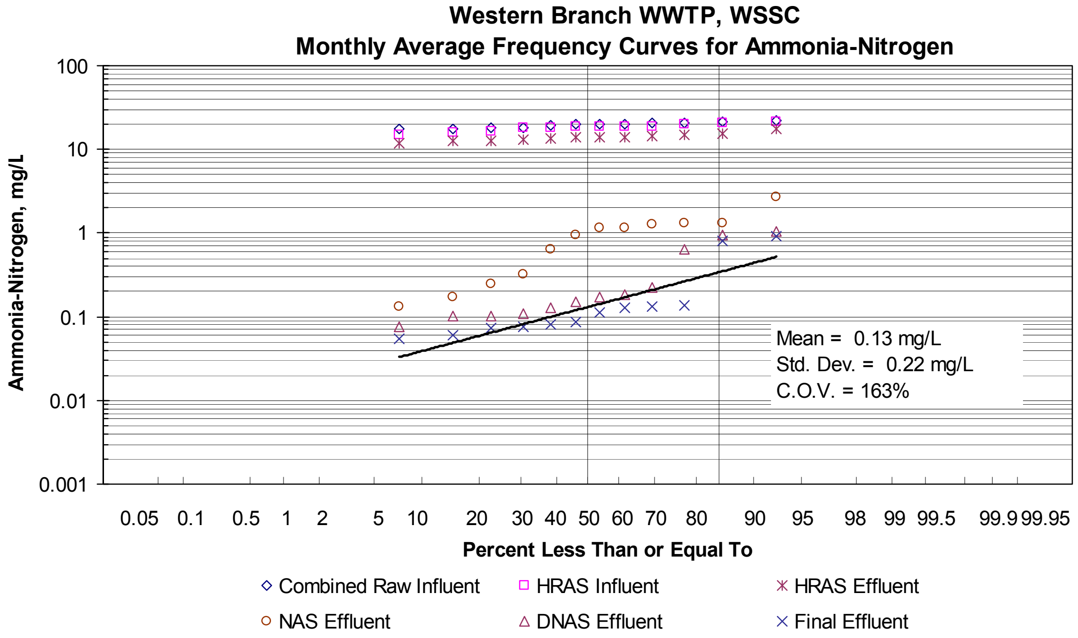

Figure 3-2. Western Branch WWTP: Monthly frequency curves for ammonia nitrogen.

The performance was efficient and reliable during the evaluation period for TN but moderate for ammonia nitrogen and TP. The average concentrations were 1.63 mg/L in TN with a COV of 36 percent, 0.22 mg/L in ammonia nitrogen with a COV of 163 percent, and 0.43 mg/L in TP with a COV of 62 percent.

Several factors contributed to this performance at Western Branch, including the following:

1. Wastewater characteristics. Because this facility uses a separate stage for denitrification with methanol and alum for phosphorus removal, the wastewater characteristics were not a concern.

2. Three activated-sludge systems in series provided redundancy and thus added reliability for TN removal, but a large footprint was required for the bioreactors and clarifiers.

3. The first two stages provided BOD removal and nitrification in two separate stages in series. The nitrate in the effluent from the second stage (NAS) averaged 16.5 mg/L with a COV of 12 percent. Note that this is high because denitrification was not designed to occur in this stage. The methanol dosage averaged 2.5 lb/nitrate nitrogen entering the DNAS tank.

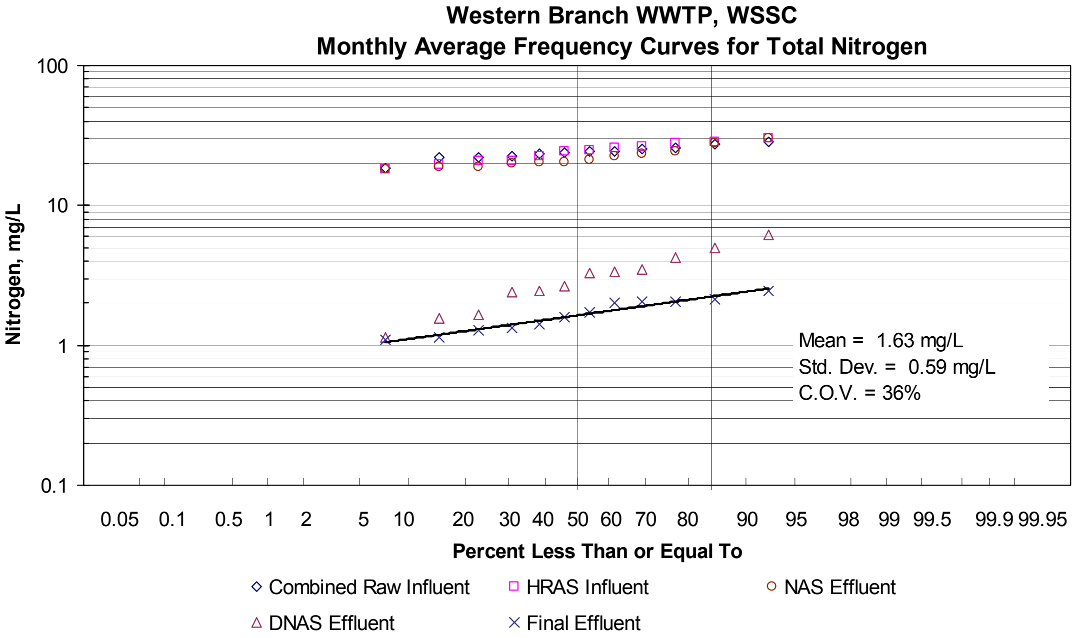

Figure 3-3. Western Branch WWTP: Monthly frequency curves for TN.

4. The control strategy included daily testing of key parameters and adjustment of the methanol dosage, when needed.

5. Alum was fed to the DNAS tanks for phosphorus removal at an average dosage of 10 mg/L with good results.

6. Dissolved oxygen (DO) probes were installed for continuous monitoring in the first two stages. The signals from the probes were used to control the air valves and thus control the air flow to the basins. The plant also has online sensors for suspended solids and sludge blanket levels in the secondary clarifiers.

7. Recycling of phosphorus has been minimized at the facility. The sludge is thickened under aerobic conditions, which reduces phosphorus release. In addition, the biosolids are incinerated rather than digested, which also reduces phosphorus recycle because most of it stays with the ash.

8. No special procedures are in place for managing wet-weather flows.

Case Study No. 2: Fiesta Village WWTP in Lee County, Florida

This facility has an oxidation ditch followed by denitrification filters with a methanol feed system. The permit contains an annual average TN limit of 3 mg/L and TP limit of 0.5 mg/L, with higher values for monthly and weekly averages. In addition, the permit specifies a daily limit of 5 mg/L in total suspended solids (TSS) for water reuse.

The facility was an extended-aeration oxidation ditch, and four denitrification filters were added along with three screw pumps. Alum was used for the TSS control required for water reuse, which aided phosphorus removal at the same time. The sludge was digested aerobically, stored, and hauled away for processing at another county facility.

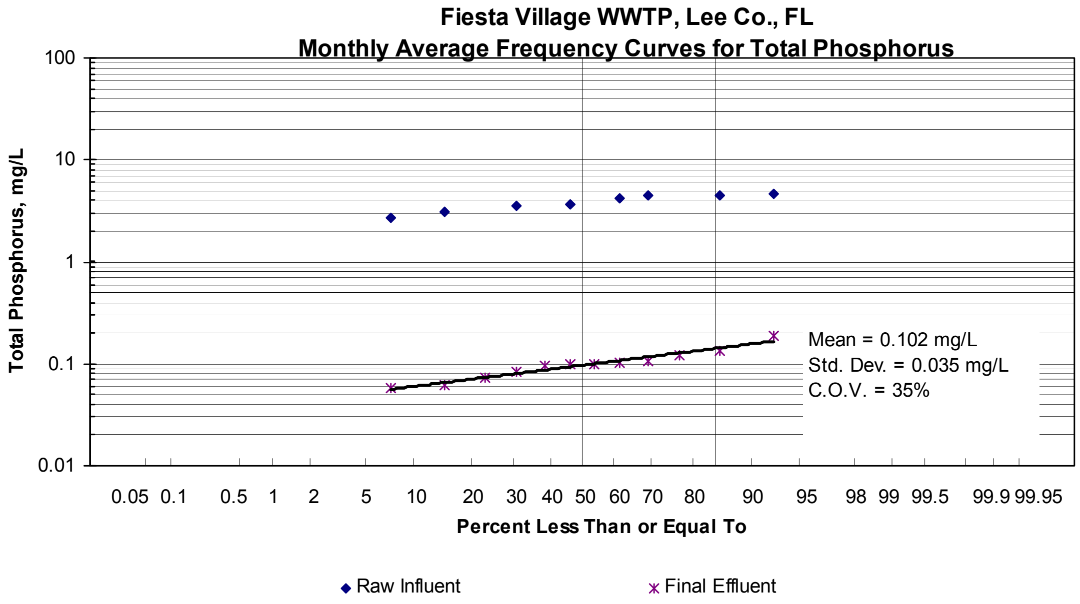

Figure 3-4. Lee County, Florida: Monthly frequency curves for TP.

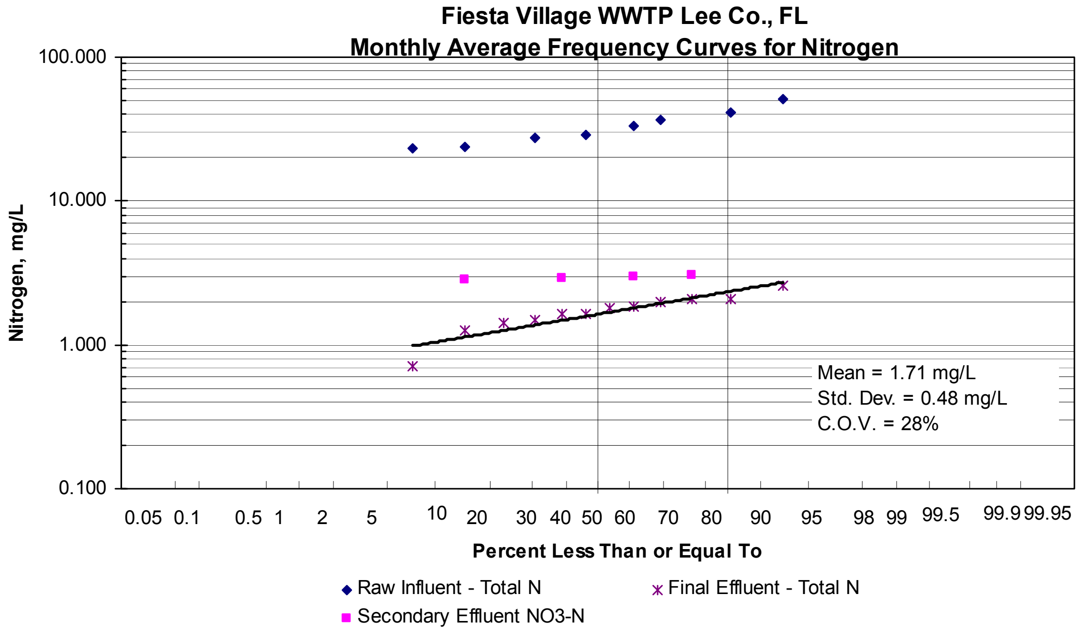

Figure 3-5. Lee County, Florida: Monthly frequency curves for TN.

The performance was efficient and reliable. The plant achieved a TN concentration of 1.71 mg/L with a COV of 28 percent and TP concentration of 0.1 mg/L with a COV of 35 percent.

A number of key factors contributed to this high performance:

1. Denitrification is accomplished in two processes in series: first in the oxidation ditch, which has a target nitrate nitrogen concentration of 3 to 3.5 mg/L, followed by the denitrification filter, with a minimal dose of methanol feed to remove additional nitrate nitrogen.

2. The high level of denitrification in the oxidation ditch is a unique accomplishment at this plant, achieved by carefully controlling the brush aerators. The standard procedure calls for turning off one brush aerator during the day and two during the night, which provides anoxic zone(s) within the ditch comprising approximately 25 to 50 percent of the tank volume. The location of these aerators varies, depending on the season. In addition, the clarifiers operate with a denitrifying sludge blanket depth between 2.5 ft and 3.5 ft.

3. The denitrifying filters then bring the nitrate nitrogen below 3 mg/L with a methanol feed rate of 1.9 lb/lb nitrate nitrogen, which is low. The dosage is determined on the basis of daily performance. This process is compact with a small footprint and, thus, is preferred at a location with limited space.

4. Alum was fed at an average dosage of 8.9 mg/L at a mass basin Al/TP ratio of 2.32 to reach an effluent concentration of 0.1 mg/L

5. Recycle loads are minimal primarily due to the fact that aerobically digested sludge is hauled off-site for processing at another facility.

6. During wet-weather periods, a normal mode of operation is maintained. Under extreme peak flow conditions, however, the biomass inventory is protected from surges by shutting off a number of aerators.

Case Study No. 3: Central Johnston County, North Carolina

This facility has a plug-flow activated-sludge system followed by a denitrification filter with a methanol feed. The permit includes a monthly ammonia limit of 2 mg/L in summer and 4 mg/L in winter, and an annual average TP limit of 1 mg/L. In addition, the TN limit is set on the basis of a load limit of 56,200 lb per year, which is equivalent to 3.7 mg/L on an annual average basis at the design flow of 7 MGD.

This facility was retrofitted from conventional activated sludge to enhanced biological phosphorus removal (EBPR), followed by a separate-stage denitrification filter for nitrogen removal. The plant treated 5.17 MGD during the maximum month and had an annual average of 4.12 MGD during the evaluation period. There is no primary settling, and the WAS is aerobically digested before dewatering by a belt filter press off-site. After UV disinfection, the reclaimed water is stored for reuse.

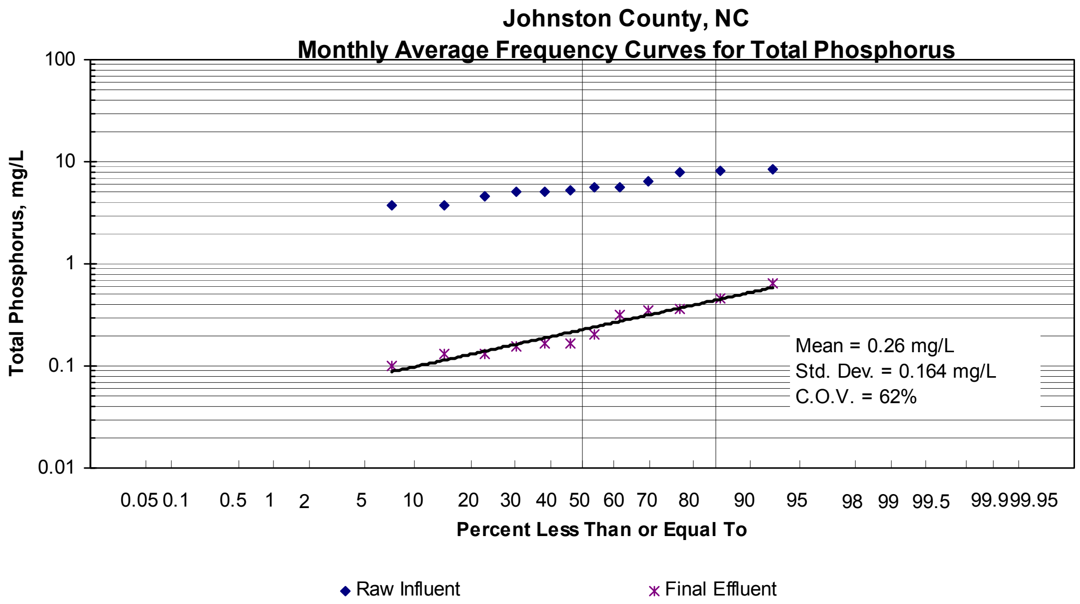

Figure 3-6. Johnston County, North Carolina: Monthly average frequency curves for TP.

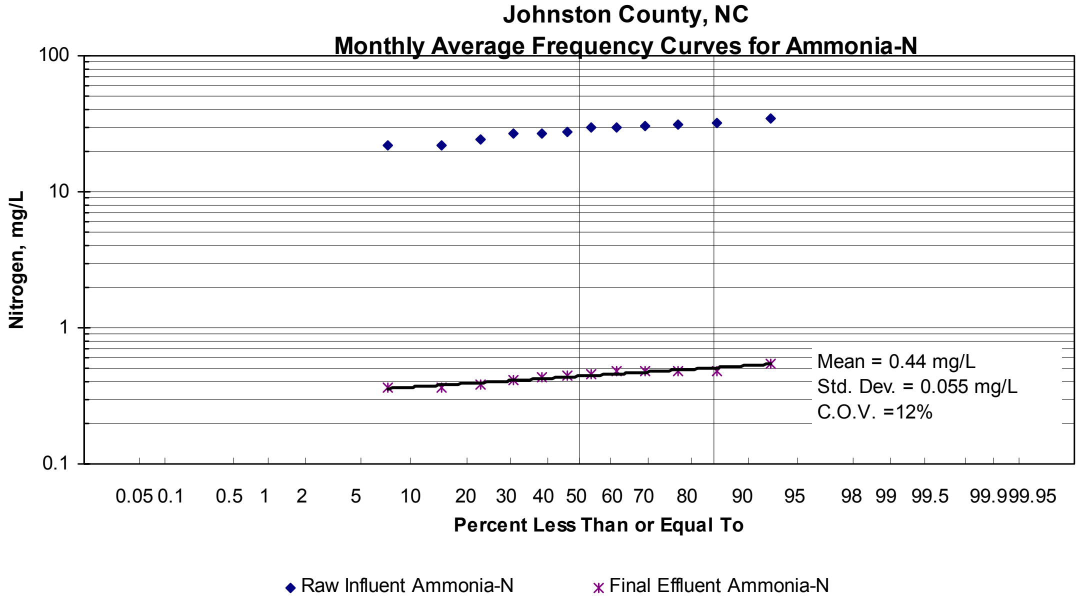

Figure 3-7. Johnston County, North Carolina: Monthly average frequency curves for ammonia nitrogen.

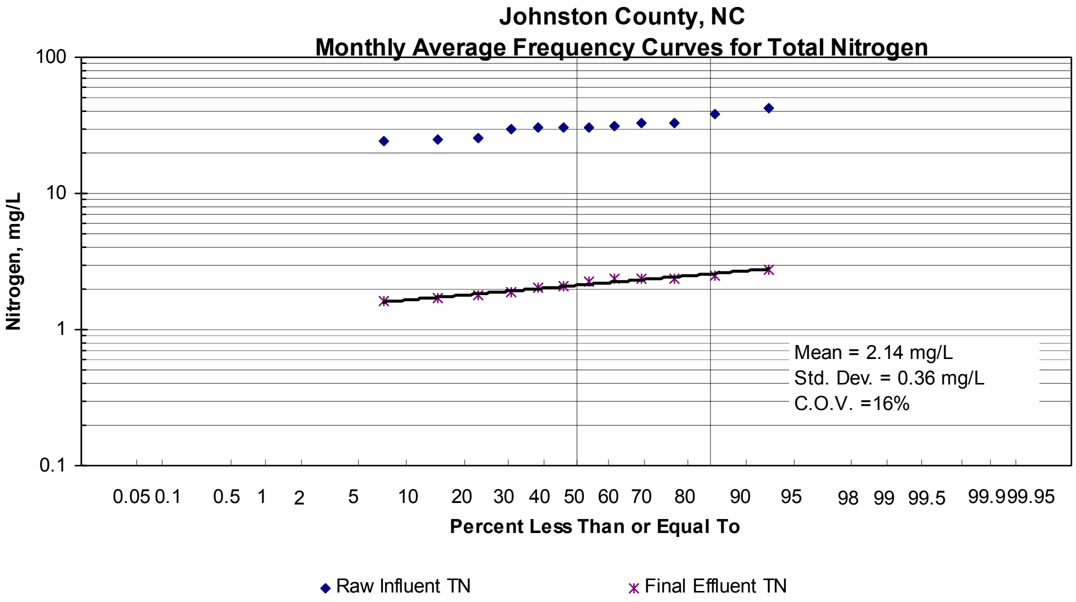

Figure 3-8. Johnston County, North Carolina: Monthly average frequency curves for TN.

The performance was highly efficient and reliable for nitrogen removal and biological phosphorus removal. The TN COV for the monthly average was very reliable at 16 percent at a mean concentration of 2.14 mg/L. The COV for ammonia nitrogen was also very reliable at 12 percent at a mean concentration of 0.44 mg/L. The COV for TP was 62 percent at an average of 0.26 mg/L on a monthly basis. TP during the maximum month was 0.64 mg/L at this COV, meeting the permitted limit of 1 mg/L.

Several distinct factors were noted at this facility:

1. The wastewater characteristics are favorable for nutrient removal. The BOD-to-TP ratio and BOD-to-total Kjeldahl nitrogen (TKN) ratio were 55.1 and 10, respectively, where 25 and 4 are reported as adequate in the literature (Lindeke et al. 2005; Neethling et al. 2005; WEF and ASCE 1998).
2. A significant amount of denitrification was accomplished in the clarifier, with a deep sludge blanket of 3 to 4 feet. (This can be referred to as a *denitrification blanket*.) As a result, the RAS flow rate was reduced significantly. Note that all these procedures were developed and optimized by plant personnel.
3. Biological phosphorus removal was successfully achieved in the retrofitted activated sludge without any chemical addition.
4. Methanol was added before the denitrification filters. The methanol was fed at a controlled dosage with online monitoring of nitrate, by a Hach probe. The average dosage of methanol for the year was relatively low at 14 mg/L.

5. This process is compact with a small footprint and, thus, is preferred at a location with limited space.
6. The recycle loads are minimized because only aerobic digestion is available on-site. The sludge is dewatered off-site, so there is no return flow from the dewatering process.
7. Wet-weather flows are managed without major difficulties. Even during Tropical Storm Alberto in June 2006, the plant operated normally. As a result of Alberto, flow increased from 3.4 to 10.5 MGD, following 10 inches of precipitation in a 24-hour period. Under extreme weather conditions like a hurricane, the plant would shut down part of the aeration basins and protect the sludge inventory. This could last for up to a day without causing adverse effects on the biomass at the plant.

Case Study No. 4: Marshall Street Advanced WWTP, Clearwater, Florida

The Clearwater, Florida, facility uses a 5-stage Bardenpho process followed by a sand filter with alum feed. This facility was retrofitted from conventional activated sludge to the 5-stage Bardenpho process and tertiary filtration.

The NPDES permit includes annual limits of 3 mg/L for TN and 1 mg/L for TP. Monthly average limits are 3.75 mg/L for TN and 1.25 mg/L for TP. In addition, dichlorobromomethane and dibromochloromethane are limited to 24 and 46 µg/L, respectively, on an annual average basis.

The plant was designed to treat 10 MGD. It treated 6.85 MGD during the maximum month with an annual average of 5.45 MGD for the evaluation year. The wastewater is settled in the primary tanks, treated at the Bardenpho process, and filtered through rapid sand filters before chlorination. The WAS is thickened by rotary-drum thickeners and then digested along with the primary sludge in anaerobic digesters before dewatering by a belt filter press. The biological treatment process selected is a classic 5-stage Bardenpho design with a long hydraulic retention time (HRT, 20 hours) and a long sludge retention time (SRT, 25 days) with an anaerobic zone for denitrification followed by two separate alternative aerobic and anoxic zones for nitrification and phosphorus removal. No external carbon source is used.

The performance was highly efficient and reliable for nitrogen and phosphorus removal. The COVs were 16 percent for the monthly averages at a mean concentration of 2.32 mg/L TN. The COV for phosphorus was 40 percent at an average concentration of 0.13 mg/L. This process has a large footprint and thus requires a large site.

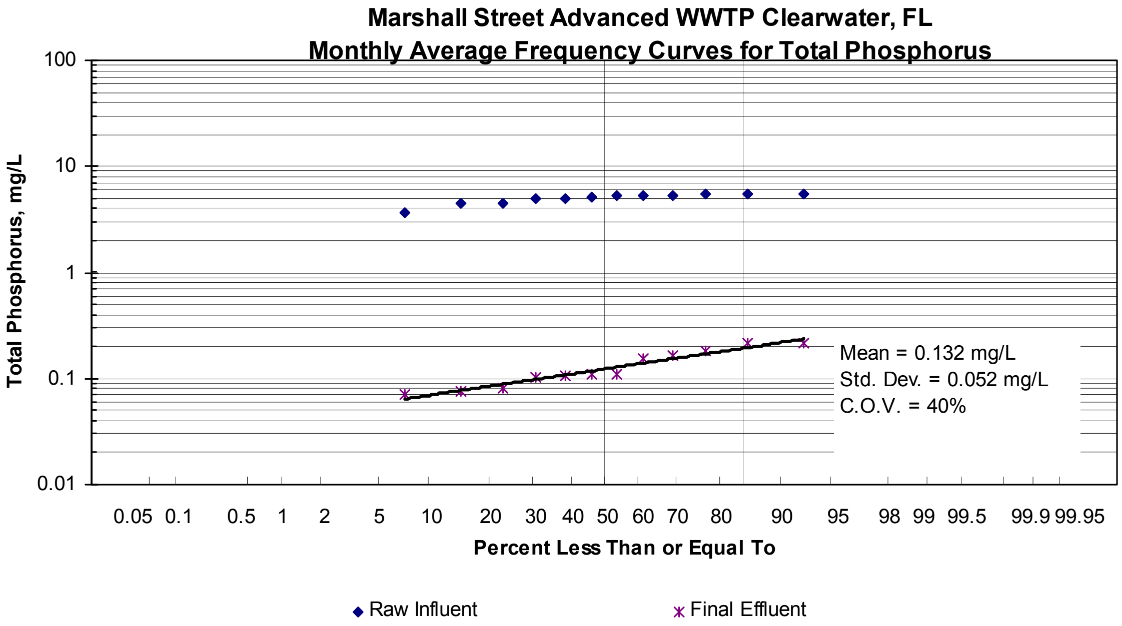

Figure 3-9. Marshall Street Advanced WWTP, Clearwater, Florida: Monthly average frequency curves for TP.

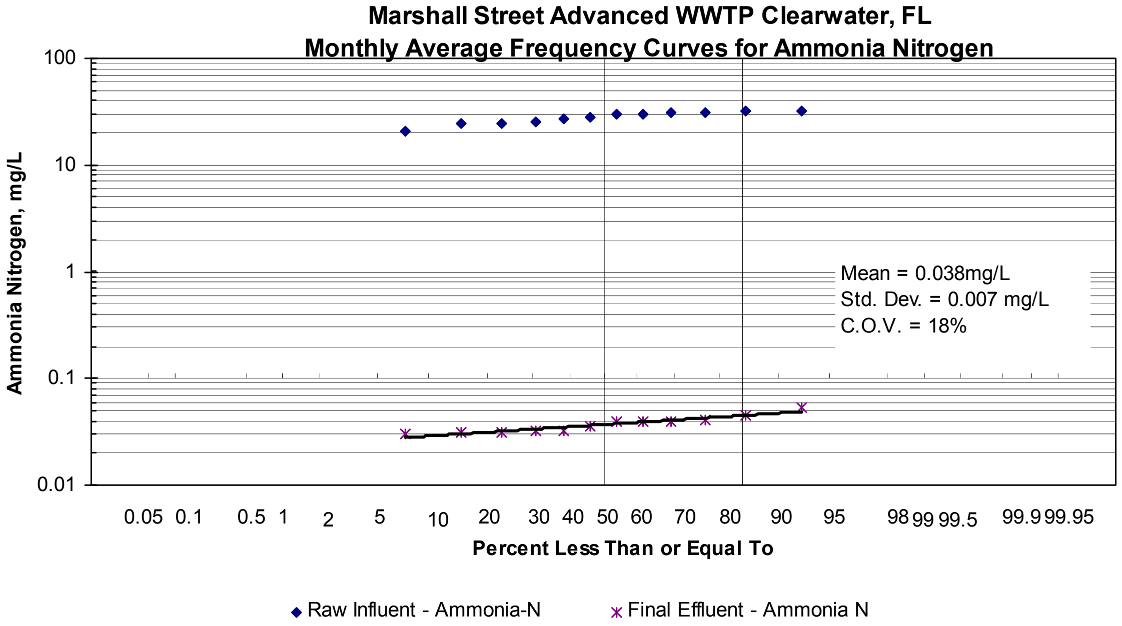

Figure 3-10. Marshall Street Advanced WWTP, Clearwater, Florida: Monthly average frequency curves for ammonia nitrogen.

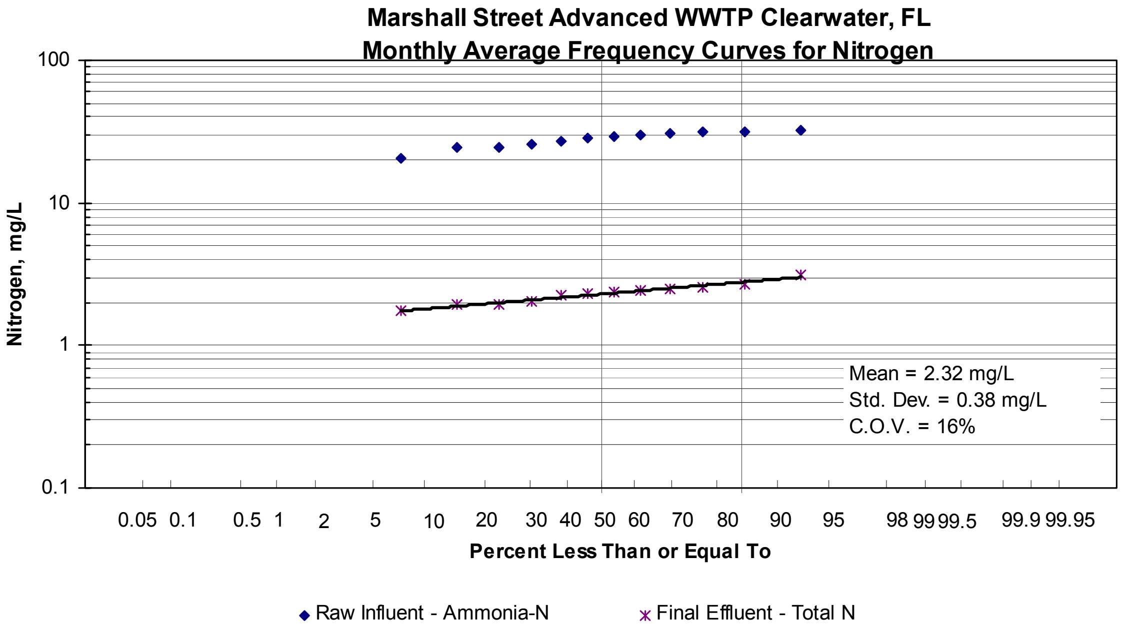

Figure 3-11. Marshall Street Advanced WWTP, Clearwater, Florida: Monthly average frequency curves for TN.

Contributing factors at this facility include the following:

1. Thorough operating guidelines were developed and followed by the plant personnel.
2. This facility had favorable wastewater characteristics in BOD, TKN, and TP. The BOD-to-TP ratio was 37.5 and the BOD-to-TKN ratio was 6.7. Both ratios are higher than what is considered adequate, 25 and 4, respectively.
3. Process controls were automated with online probes for DO, nitrate, ortho-phosphate, and oxidation-reduction potential (ORP). The controls were based on readings in the second anoxic zone for nitrate, DO, and ORP. The strategy developed by plant personnel includes a minimum target ORP at –60 millivolts and nitrate nitrogen at 0.5 mg/L. DO levels are then adjusted by automatic controls to stay in this region. In addition, plant personnel use turbidity and conductivity meters for effluent monitoring. Turbidity is for reuse monitoring, as required by Florida, and conductivity is used as an early warning indicator for potential seawater intrusion to the sewer system to protect the water reuse customers.
4. Alum is added to meet the trihalomehthane requirements of Florida for water reuse. The average dosage was 27 mg/L as alum, or at the aluminum-to-phosphorus ratio of 1.6 on a molar basis.
5. Recycle flows from dewatering are mixed with the plant influent and treated together. The recycle loadings from dewatering were moderate at 30 percent for phosphorus

and 14 percent for nitrogen. Thickening by mechanical equipment helped in minimizing recycle loads in nitrogen and phosphorus. Anaerobic digestion also contributed to the increase in recycle loads of these nutrients to moderate levels.

6. Wet-weather flows are handled with the normal mode of operation.

3.2.2 Total Nitrogen and Phosphorus Removal at Mid-Level Concentration Limits (3 to 6 mg/L)

Three case studies fell into this group, and they are characterized as having one anoxic zone with or without external carbon sources.

Case Study No. 5: North Cary, North Carolina

The North Cary, North Carolina, facility uses a phased isolation oxidation ditch (PID) (Biodenipho) with an upflow sand filter. The NPDES permit requires ammonia limits of 0.5 mg/L for summer and 1.0 mg/L in winter, 3.94 mg/L for TN on an annual average, and 2 mg/L on a quarterly basis.

This facility was expanded, replacing the Schreiber process with a PID, in 1997. The design flow was 12 MGD and treated 8.7 MGD in the maximum month and 7 MGD as the annual average of this evaluation period. The wastewater is treated at this PID, filtered by an upflow Dynasand filter, and disinfected by UV irradiation before discharge. The facility has a 2–million-gallon (MG) equalization basin online and has additional storage capacity of 7 MG in the old facility on-site. The sludge is aerobically digested and shipped to another facility for dewatering and drying.

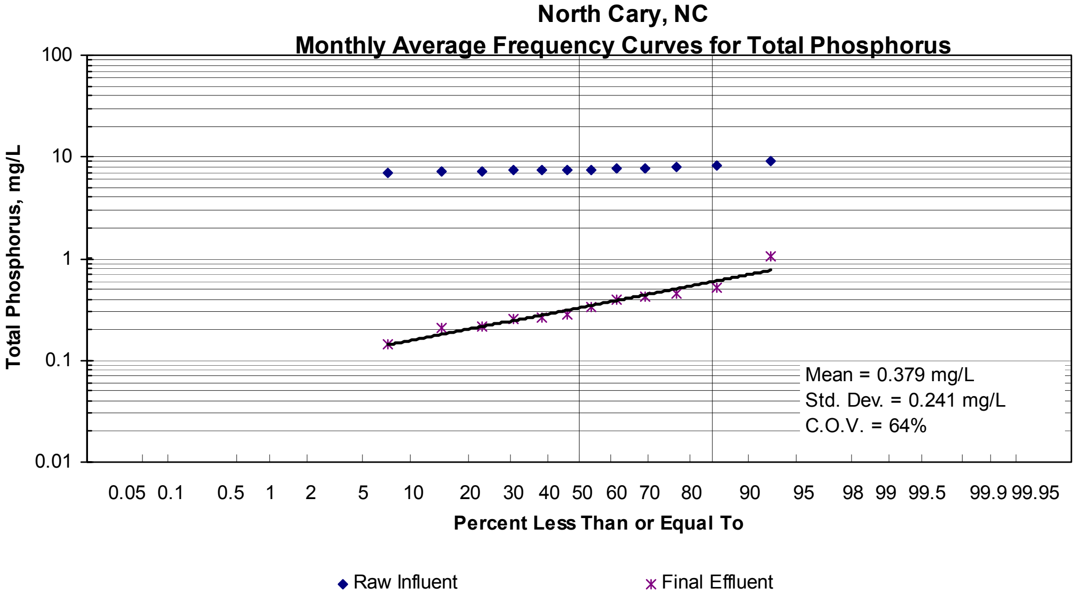

Figure 3-12. North Cary, North Carolina: Monthly average frequency curves for TP.

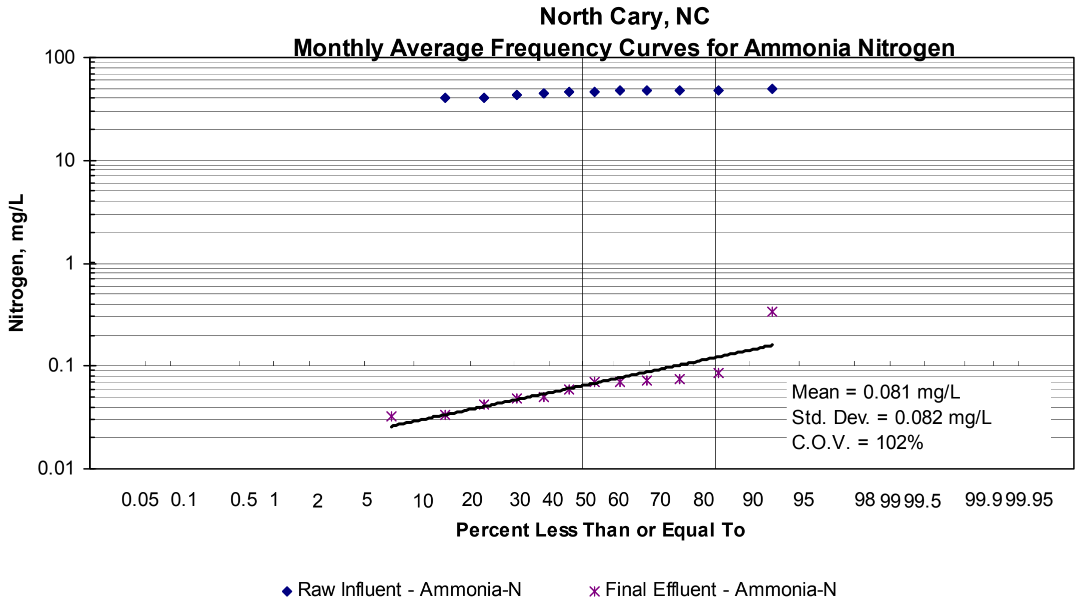

Figure 3-13. North Cary, North Carolina: Monthly average frequency curves for ammonia nitrogen.

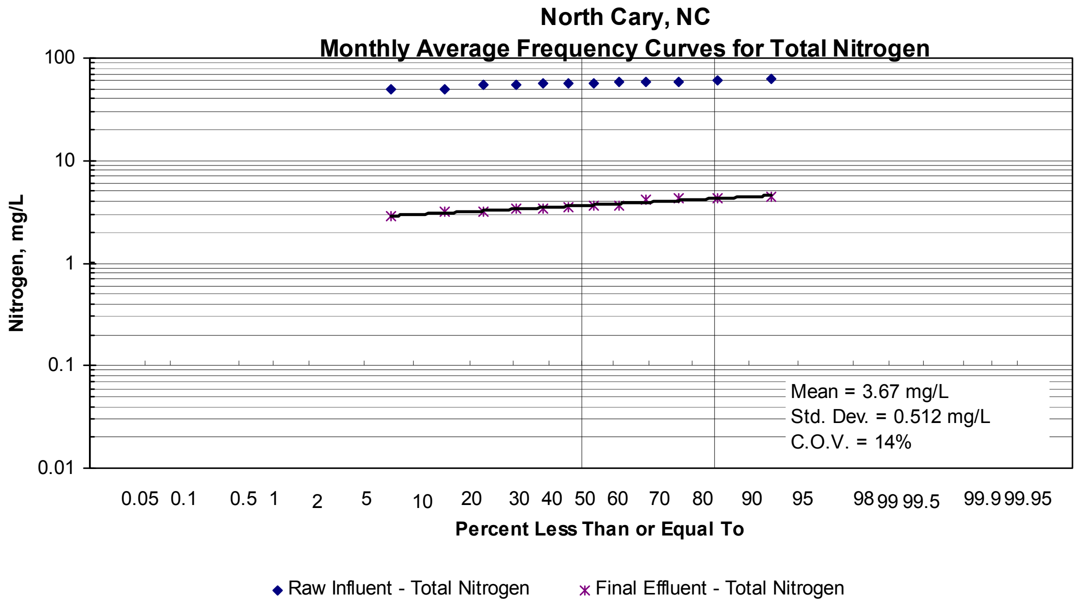

Figure 3-14. North Cary, North Carolina: Monthly average frequency curves for TN.

The performance was efficient and reliable. The COVs were 102 percent for ammonia at a mean concentration of 0.08 mg/L, 64 percent for TP at a mean concentration of 0.38 mg/L, and 14 percent for TN at a mean concentration of 3.67 mg/L. This was achieved without any chemical addition. The quarterly average TP was 0.57 mg/L at this COV and met the limit of

2 mg/L. The maximum month ammonia nitrogen concentration was 0.34 mg/L, well within the permit limit of 0.5 mg/L in summer.

This process provides two anaerobic selector zones ahead of the oxidation ditch, which alternate between anoxic and aerobic cycles to maximize nitrification and denitrification using the carbon present in the wastewater. The oxidation ditch is followed by two anoxic zones and one reaeration zone.

Several key factors were noted at this facility:

1. Wastewater characteristics were favorable. The BOD-to-TP ratio and BOD-to-TKN ratio were 31.6 and 4.3, respectively. Both were adequate in accordance with the literature.
2. Reliability is ensured with two anaerobic zones ahead of and two anoxic zones after the ditch. The automatic process logic control of cycle times is based on online DO measurements.
3. Separate means of providing aeration and mixing in the ditch allow optimal control of aerobic and anoxic cycles for best performance.
4. Recycle loads are minimized by having only aerobic digestion on-site; there are no anaerobic processes. The digested sludge is transported off-site for processing, and therefore no recycled loads go back to the facility.
5. Wet-weather operation follows the normal mode of operation. The PID process design allows the cycle time to be adjusted based on incoming flow. When the system is in *storm mode* the plant is switched to sedimentation, thereby preventing solids washout.
6. Equalization basins with a total volume of 9 MG are available. During Tropical Storm Alberto in June 2006, all the equalization basins were filled, the PID operated in storm mode for a short duration, and the facility was still able to comply with all NPDES permit limits.

Case Study No. 6: Kelowna, British Columbia

The Kelowna, British Columbia, facility uses a 3-stage Westbank process with a fermenter and a dual-media filter. The Canadian Ministry of Environment permit requires a TN limit of 6 mg/L and 2.0 mg/L on a daily basis, or 1.5 mg/L on the 99th percentile, or 1 mg/L on the 90th percentile, and 0.25 mg/L on an annual basis for TP.

This facility was retrofitted from the existing 5-stage Bardenpho to the 3-stage Westbank process. The design capacity is 10.5 MGD, and the facility treated 7 MGD as the annual average for the evaluation year. The wastewater is settled in primary tanks, treated at the bioreactor, filtered through dual-media filters, and disinfected by UV. The primary sludge is fermented and stored before dewatering and composting off-site. The secondary sludge is thickened by DAF and stored. Combined primary and secondary sludge is then dewatered by centrifuge for off-site composting. The primary effluent can be sent to the equalization basin, which has a capacity equivalent to approximately 7.5 percent of the average design flow.

The performance was efficient and reliable. The COV was 12 percent for the monthly average data at a mean concentration of 4.38 mg/L TN. The COV for TP was 21 percent at the mean concentration of 0.139 mg/L. The maximum month average was 0.20 mg/L TP and 4.9 mg/L TN at these low COVs.

Some of the unique features of this facility are as follows:

1. The wastewater characteristics were favorable but not sufficient year-round. The BOD-to-TP ratio was 26.6 and found to be somewhat low for a cold-region facility. The BOD-to-TKN ratio was 5.3, which was adequate for what is recommended in the literature.
2. The unique features of this facility include the fermenter for volatile fatty acid (VFA) production, which ensured a reliable and efficient operation of the EBPR. The design and operation of fermenter technology evolved from this facility. Current operation is based on 5-day sludge age year-round, and the predominant VFA species were acetic acid and propionic acid, both of which are desirable for a good EBPR.

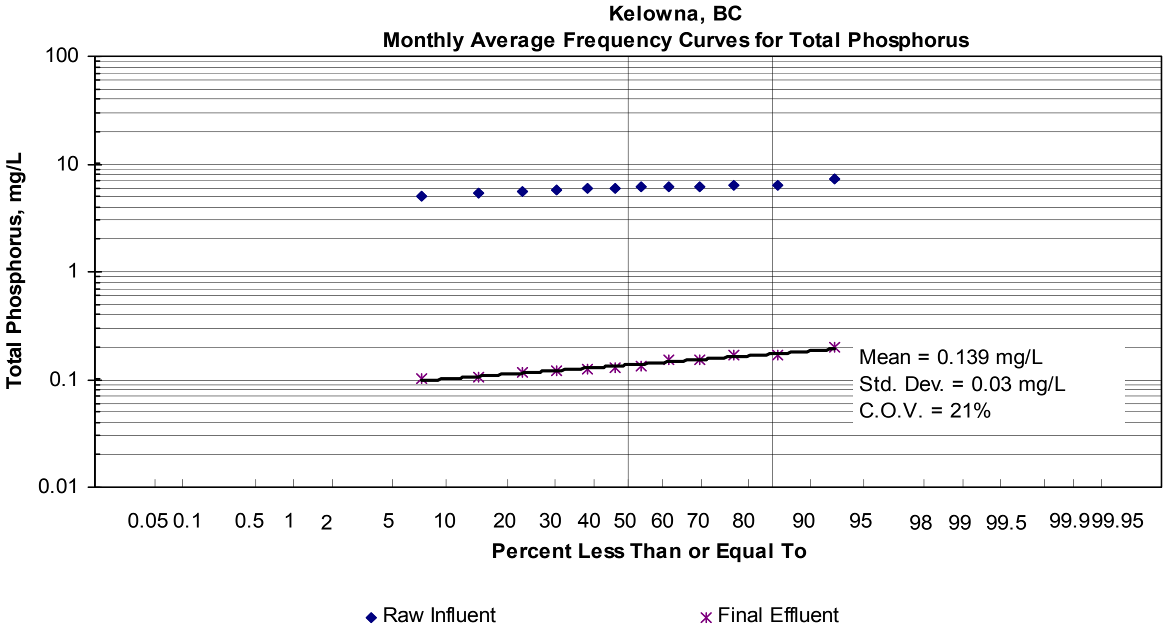

Figure 3-15. Kelowna, British Columbia: Monthly average frequency curves for TP.

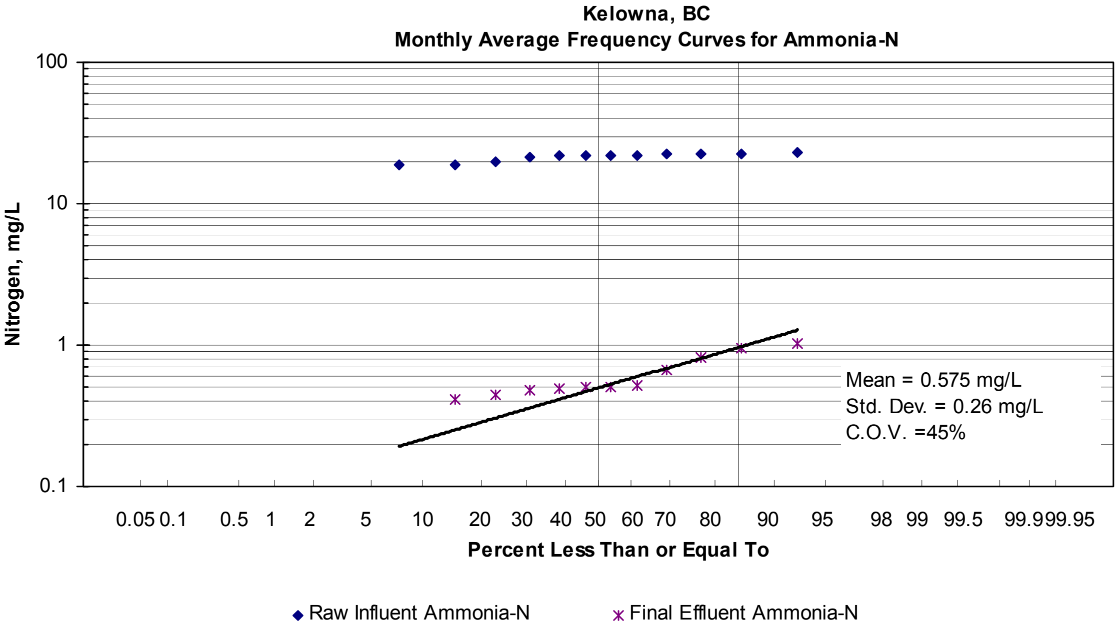

Figure 3-16. Kelowna, British Columbia: Monthly average frequency curves for ammonia nitrogen.

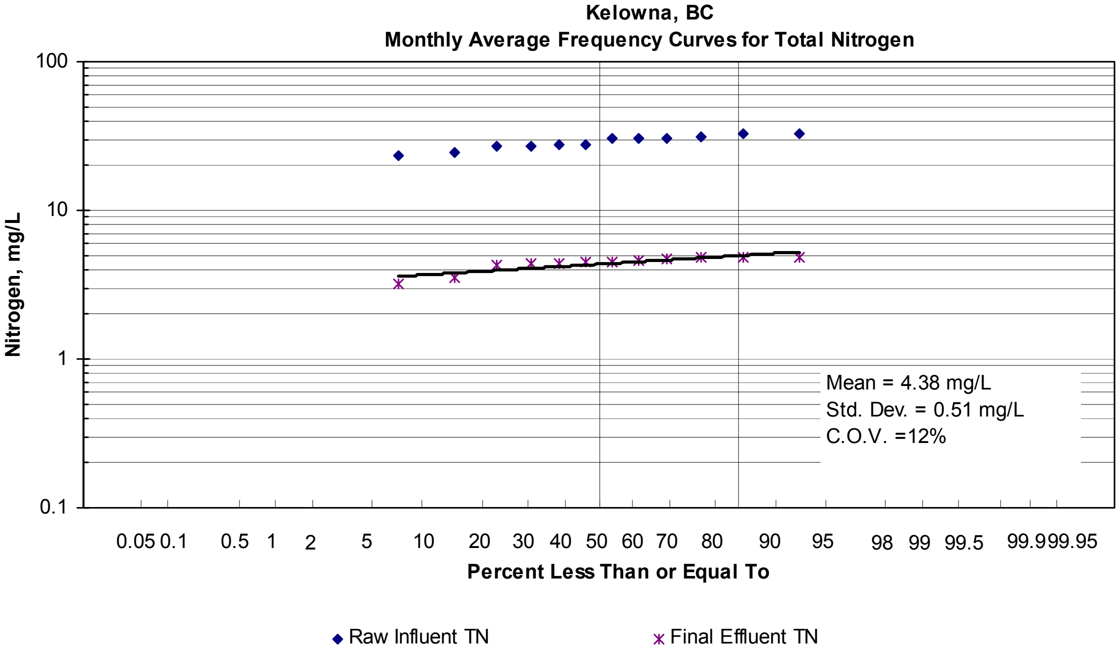

Figure 3-17. Kelowna, British Columbia: Monthly average frequency curves for TN.

The annual production of VFA is equivalent to 10 mg/L in the influent to the bioreactor from the fermenters. Other sources of VFA in the plant included centrate from dewatering, primary effluent, and influent, all of which added up to the equivalent of 7 mg/L as an average. The target VFA was 4–8 mg VFA/mg for soluble phosphorus removed. The VFA-rich fermenter supernatant (17 mg/L) was directly discharged to the anaerobic zone, ensuring a steady feed of VFA to the phosphorus-accumulating organisms (PAOs).

3. A step-feed of influent is split equally for both the anaerobic and anoxic zones. This process has evolved from the 5-stage Bardenpho, with the modifications resulting in shorter HRT (10 hours) and SRT (10 days). The HRT of the anaerobic zones was reduced from 3 hours to 1 hour with the direct addition of VFA and primary effluent.

4. Like the Central Johnston County, North Carolina, and Lee County, Florida, facilities, this facility used a denitrification blanket in the clarifier. The average depth ranged between 2 and 3 feet. Nitrate reduction in the RAS blanket up to 6 mg/L did not create rising sludge concerns. This allowed minimal potential for nitrate return to the anaerobic zone.

5. The denitrification was controlled by continuously monitoring ORP at the end of the anoxic zone. These data were fed into the computer system to ensure that a sufficient amount of primary effluent was diverted to the anoxic zone to meet the nitrate load from the nitrified internal recycle flow. The average was approximately 50 percent of the primary effluent.

6. The Kelowna plant operates under extremely low temperatures. A monthly average low temperature of 13 degrees Celsius (°C) was observed during the months of January and February. A high temperature of 22 °C was recorded in August.

7. The facility has an especially flexible design. Of the 21 bioreactor cells, 17 have dual equipment—mixers and diffusers. This allows each cell to have the ability to be operated as a swing zone, which was expensive but offers great flexibility for changes in wastewater and permit requirements.

8. Recycle loads from dewatering were minimized by maintaining separate processes for secondary sludge and primary sludge. No sludge digestion was practiced. The total recycle loads from dewatering were only 13 percent in TP and 0.1 percent in TN.

9. Wet-weather flows were managed under the normal mode of operation, using the equalization basin. The collection system was separated, and the seasonal variation was not very significant. The maximum month flow was 10 percent higher than the average flow. The total basin capacity was equivalent to 7.5 percent of the influent design flow rate.

Case Study No. 7: Fairfax County, Virginia

The Noman M. Cole facility in Fairfax County, Virginia, uses a step-feed activated-sludge process with a tertiary filter with ferric chloride addition. The NPDES permit limits are 1.0 mg/L for ammonia nitrogen in summer and 2.2 mg/L in winter, and 0.18 mg/L for TP on a monthly basis. The facility is not required to report TN, but it started a voluntary program in anticipation of the future limit as a part of the Chesapeake Bay Initiative.

This facility was expanded from the existing step-feed activated-sludge plant in 2002. The new design capacity of the plant is 67 MGD, and the annual average flow was 47 MGD during the evaluation year. The wastewater is settled in the primary tanks, treated by activated sludge with three to four feed points, settled again in the tertiary clarifier, and filtered before disinfection by chlorination and dechlorination. The secondary effluent is equalized up to 13.2 MG. The tertiary filters consist of the old, dual-media units and the new, deep mono-media filters. The headworks area also has retention basins with a capacity of 5.7 MG and an equalization basin with 4 MG, which together are equivalent to 15 percent of the average flow. The secondary sludge is thickened by DAF. The primary sludge is fermented in the gravity thickeners, which operate at an HRT of less than a day and a sludge age of 3 days. The fermented primary sludge and thickened secondary sludge were mixed together for dewatering by centrifuge, followed by incineration.

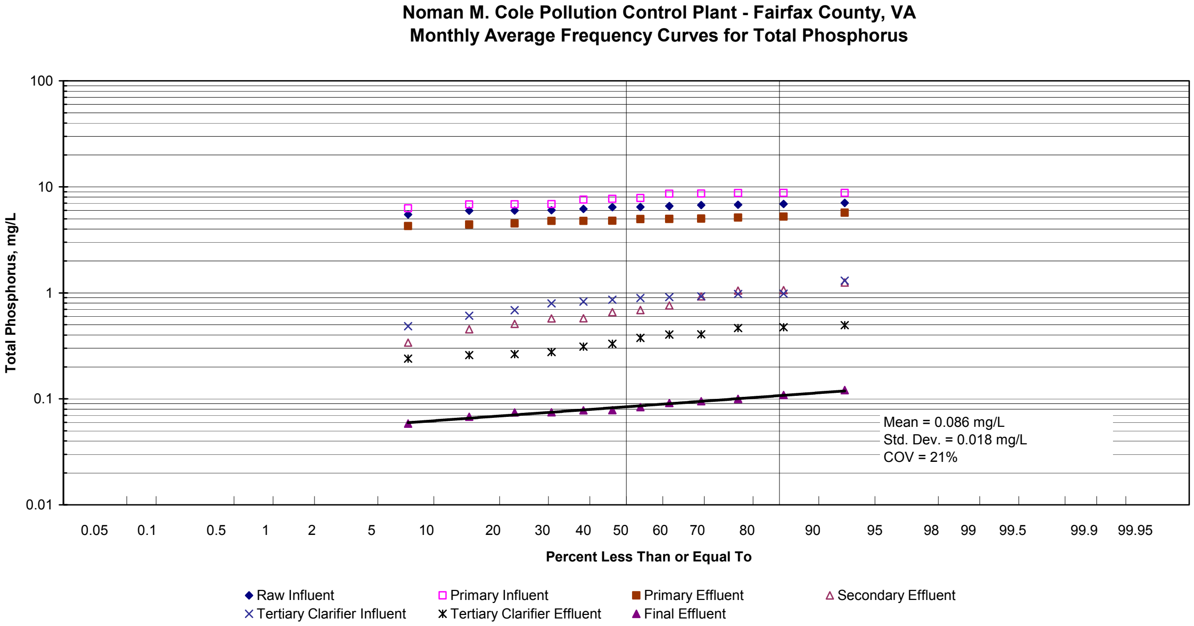

Figure 3-18. Noman M. Cole Pollution Control Plant, Fairfax County, Virginia: Monthly average frequency curve for TP.

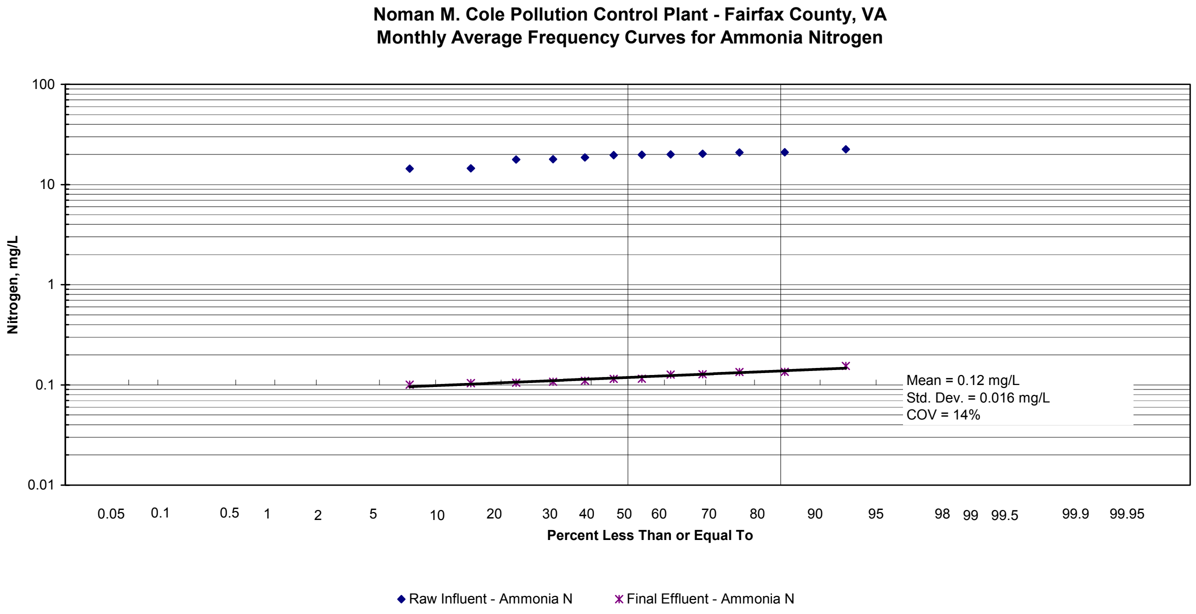

Figure 3-19. Noman M. Cole Pollution Control Plant, Fairfax County, Virginia: Monthly average frequency curve for ammonia nitrogen.

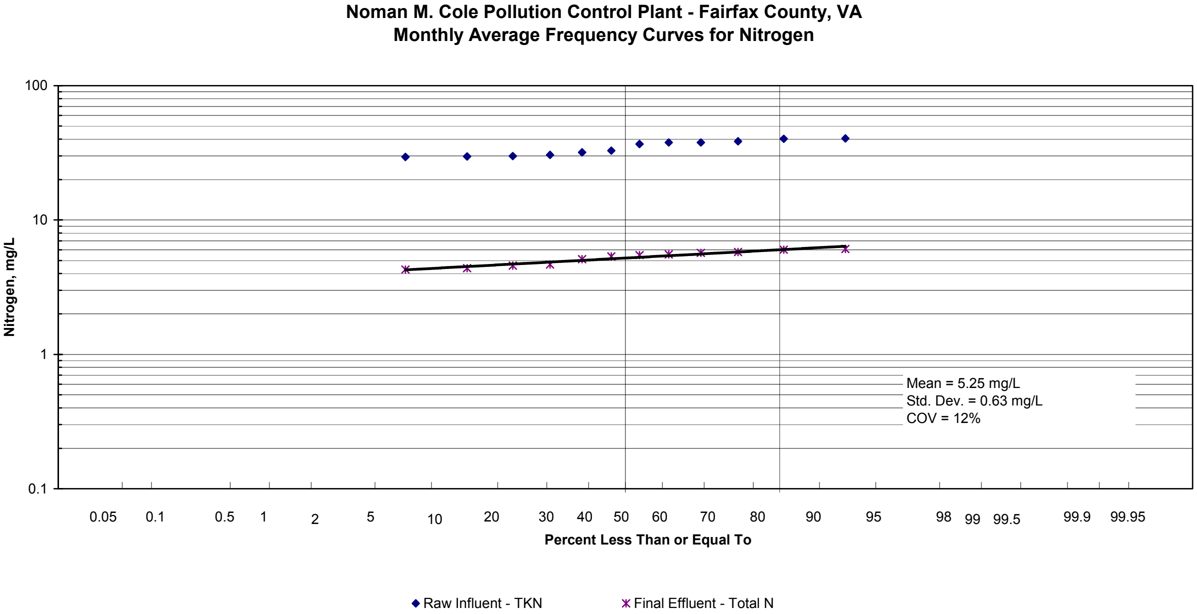

Figure 3-20. Noman M. Cole Pollution Control Plant, Fairfax County, Virginia: Monthly average frequency curve for TN.

The plant performed well with good reliability. The COV was 21 percent for TP on a monthly basis at a mean concentration of 0.09 mg/L. The COV for ammonia nitrogen was 14 percent at a mean concentration of 0.12 mg/L N. The COV was 12 percent for TN on a monthly average at a mean concentration of 5.25 mg/L. This facility included three feed locations with a 40, 40, and 20 percent flow split in the old plant and four feed points with a 25 percent split in the new plant, with an anoxic zone in each feed point, which provides an opportunity to denitrify rapidly with carbon present in wastewater.

Contributing factors for this group of facilities include the following:

1. The wastewater characteristics were favorable. The BOD-to-TP ratio and BOD-to-TKN ratio were 29.5 and 5.4, respectively.
2. The step feed is unique in providing a dedicated carbon supply and three anoxic cycles for better nitrogen removal. Caustic soda is fed to supplement the alkalinity deficiency in the wastewater.
3. Phosphorus removal is achieved in three steps: biological removal in activated sludge, chemical removal in tertiary clarifier, and then tertiary filters.
4. Primary sludge is fermented in gravity thickeners with an SRT of 3 days and HRT of approximately 1 day. The units are operated between the hours of 8 a.m. and 2 p.m., when the greatest amount of ammonia reaches the plant. The VFA production was equivalent to 10 mg/L by chemical oxygen demand in the primary effluent. The VFAs consist of 33 percent acetic acid, 49 percent propionic acid, and 18 percent others. VFAs produced in this way are fed to the anoxic zones of the activated-sludge process.
5. Secondary sludge is thickened by DAF, thus minimizing the release of phosphorus and ammonia.
6. The plant design already includes recycle flows and loadings. They include 10 percent in BOD, 19 percent in TSS and 23 percent in TP. All tanks and processes are sized to include these loadings, which is a conservative approach. Lime is added to the cake up to 13 percent by weight to minimize these recycle loads back to the plant.
7. Wet-weather operation follows the normal mode of operation. The facility manages the wet-weather flow in four distinct steps: at the retention basins first, then equalization at the headworks, step-feed activated sludge, and finally equalization of the secondary effluent. Because the facility is a step-feed facility, the process is more stable than that at other plants. The equalization basins were designed to divert flow at 1.6 times the average flow and then again in the secondary effluent before the tertiary clarifiers.

3.2.3 Phosphorus Removal Low Level (less than 0.1 mg/L)

There are two case studies for this level of performance, one with both biological phosphorus removal and chemical phosphorus removal (Clark County, Nevada) and the other characterized as having biological phosphorus removal (Kalispell, Montana).

Case Study No. 8: Clark County, Nevada

The Clark County, Nevada, facility uses an anoxic and then oxygenated (anoxic/oxic, or A/O) process with primary sludge thickening for VFA production and tertiary filters with alum feed. This facility was expanded from 88 MGD to 110 MGD in 1995 for the A/O process to meet daily load limits on nutrients. The limits are equivalent to an ammonia limit of 0.56 mg/L and a phosphorus limit of 0.19 mg/L at 110 MGD. The facility treated 95 MGD during the evaluation period. Clark County made these voluntary efforts to reduce phosphorus during the evaluation period to the lowest possible concentration.

The facility has two plants—the Central Plant (CP) and the Advanced Waste Treatment Plant (AWT). The wastewater is settled at primary tanks, treated in the A/O process, filtered, and disinfected by UV irradiation and sent either to reclaimed-water users or to Las Vegas Wash and eventually to Lake Meade. The AWT has tertiary clarifier ahead of its filter, whereas the CP does not. The tertiary filter at the AWT does not have air scour, whereas the CP filter has air scour capability. Ferric chloride is added into the primary clarifiers for odor control, while alum is added to both the tertiary clarifiers and tertiary filters. The secondary sludge is thickened by DAF, stored together with primary sludge, and dewatered by a belt filter press. The cake is sent to a landfill.

The facility produced low concentration phosphorus and ammonia in the effluent with good reliability. The COV was 30 percent on the average concentration of 0.099 mg/L in TP and 14 percent on the average ammonia concentration of 0.12 mg/L. The daily limits were met in accordance with the permit, at 0.22 mg/L for TP and 0.57 mg/L for ammonia nitrogen.

Contributing factors for this facility include the following:

1. The wastewater characteristics were favorable. The BOD-to-TP ratio was 29.8 for the year and ranged from 26.5 to 34.2 on a monthly basis.
2. VFA generation was achieved in the primary settling tanks. The practice was based on thickening the sludge up to 5 percent total solids (TS). The upper limit was 6 percent; and higher percentages should be avoided.
3. The A/O process parameters were optimized for the SRT and RAS flow rate seasonally. The clarifier operated with a minimal sludge blanket—less than 6 inches. The automation and computer controls include DO control at 0.5 mg/L using multiple probes on all nine basins with WAS management on a daily basis. The critical time of

the year is when the temperature rises above 110 degrees Fahrenheit (°F) and oxygen transfer becomes less efficient.

4. Biological phosphorus removal by the A/O process followed by chemical addition reduced overall chemical sludge generation.
5. Waste secondary sludge was thickened in the DAF unit, thereby minimizing phosphorus release because of anaerobic conditions.
6. Alum was added to the tertiary clarifier in the AWT and alum was added again to the tertiary filter to minimize secondary release of phosphorus in the filters.
7. Recycle loads were minimized by daily processing of all sludge (a no storage policy), ferric addition to the filtrate from dewatering, DAF thickening of the WAS, and not digesting sludge on-site.
8. The plant follows the normal operating procedures during wet-weather events in August and September.

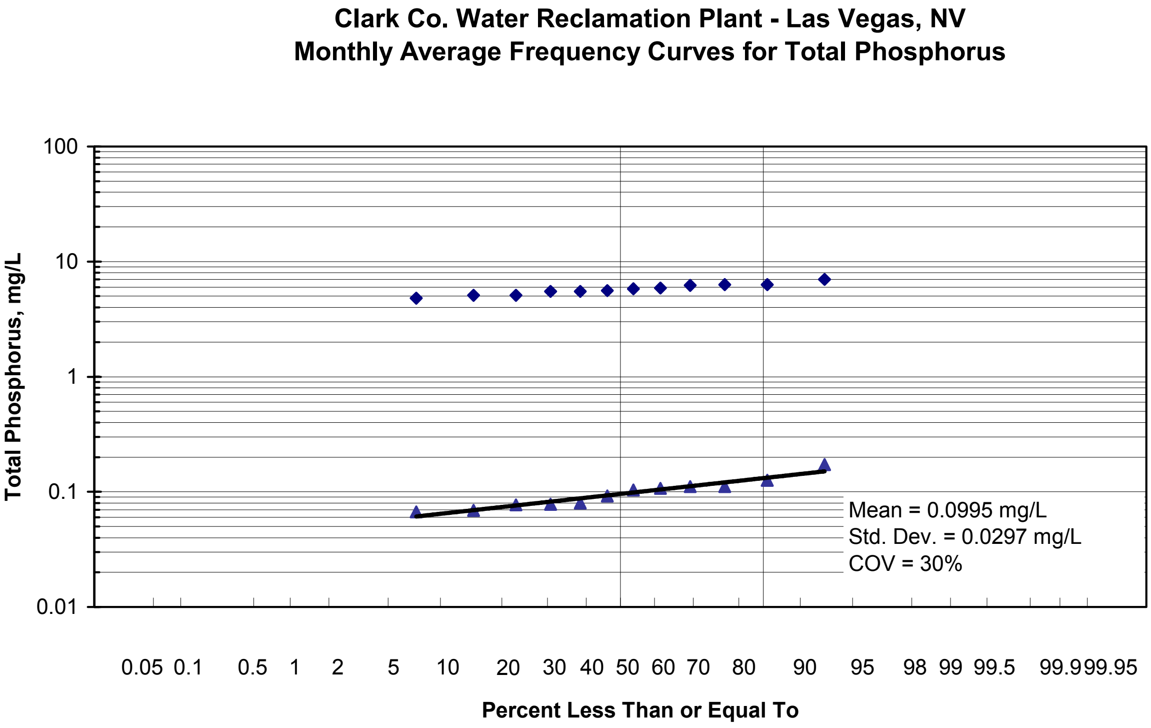

Figure 3-21. Clark County Water Reclamation Plant, Las Vegas, Nevada: Monthly average frequency curve for TP.

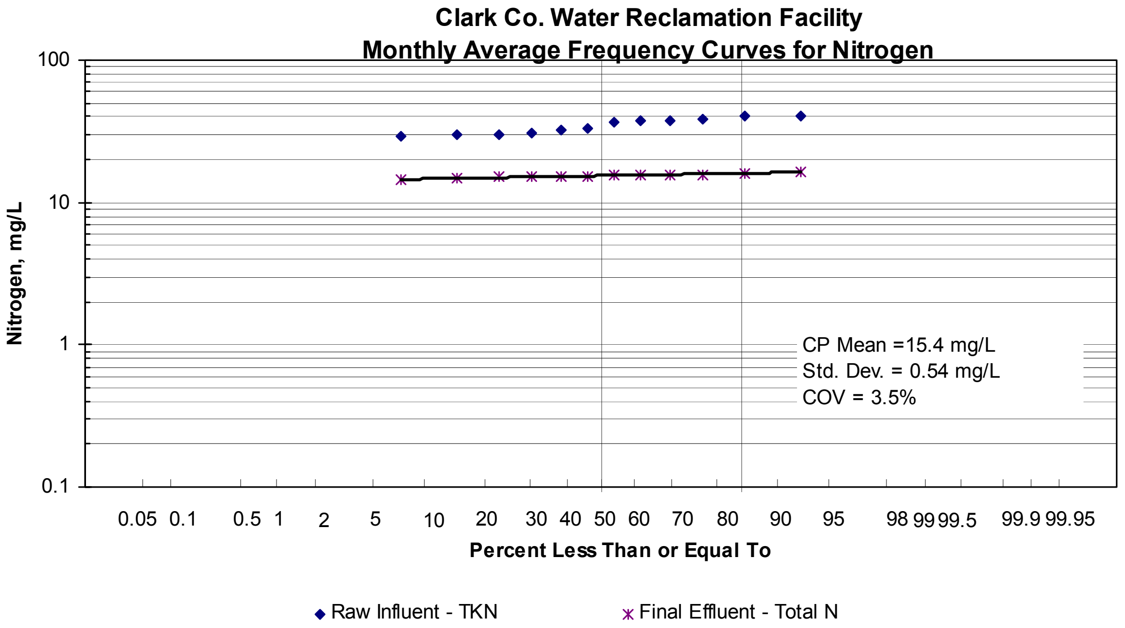

Figure 3-22. Clark County Water Reclamation Plant, Las Vegas, Nevada: Monthly average frequency curve for TN.

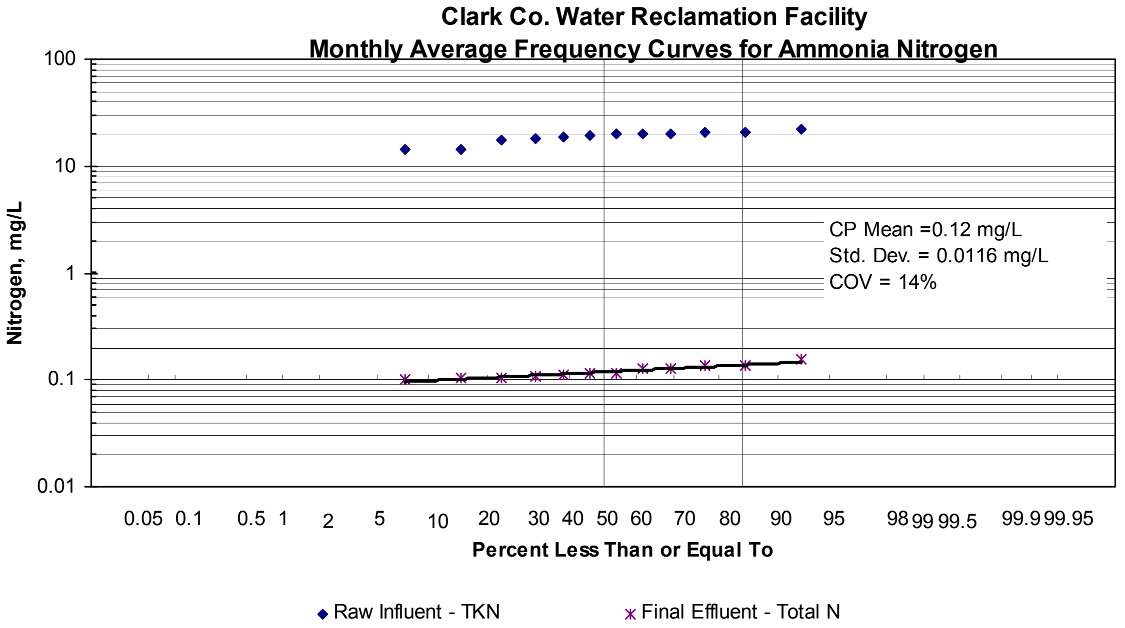

Figure 3-23. Clark County Water Reclamation Plant, Las Vegas, Nevada: Monthly average frequency curve for ammonia nitrogen.

Case Study No. 9: Kalispell, Montana

The Kalispell, Montana, facility uses a modified University Cape Town (UCT) process with a fermenter and an upflow sand filter. The NPDES permit specifies a monthly limit of 1 mg/L in TP and an ammonia limit of 0.14 mg/L N. The facility was designed for 3 MGD and treated 2.8 MGD during this evaluation year. The wastewater is settled in the primary tanks, treated in the modified UCT process, filtered, and then disinfected using ultraviolet radiation before discharge. The primary sludge is fermented before anaerobic digestion. The secondary sludge is thickened by DAF. The thickened secondary sludge and digested primary sludge are dewatered together by a belt filter press and then trucked off-site for composting.

The plant operated very efficiently and reliably. The COV was 19 percent on a monthly average basis at the mean concentration of 0.12 mg/L TP. The COV for ammonia nitrogen was 0 percent, below detection at all periods. The effluent concentrations in the maximum month were 0.15 mg/L for TP and 0.07 mg/L for ammonia nitrogen.

The unique features of this facility are as follows:

1. The wastewater characteristics were favorable. The BOD-to-TP ratio was 55, much higher than the value of 25 reported favorable in the literature. No nitrogen removal is required at this facility.

2. A 2-stage fermenter was installed to ensure a definite supply of VFAs in anticipation of low winter temperatures and increased flows during the spring snowmelts and rainfalls. The fermenter took the primary sludge and operated at an SRT of 5 days and an HRT of 7 to 21 hours. The unique system allows independent control of HRT and SRT. The HRT is controlled by adjusting fermenter volume, while the SRT is controlled by a function of the mass of solids lost from the primary clarifier and the volume of fermented sludge wasted daily from the fermenter. The VFA production was estimated to be equivalent to 18 mg/L at 20 °C and 13 mg/L at 13 °C.

3. The bioreactor has been optimized for SRT and HRT, and the RAS is now designed to maintain a target mixed liquor suspended solids level in the aeration basin. A no-blanket policy has been applied to the clarifier operation.

4. The WAS is thickened in DAF units and thus kept aerobic to minimize release of phosphorus, before it is dewatered by belt filter press. Although this facility nitrified fully down to the detection limit (at 0.07 mg/L), denitrification was not required and thus was not practiced. The COV for ammonia nitrogen was 0 percent at the mean concentration of 0.07 mg/L as nitrogen. The COV was 31 percent at the mean concentration of TN of 10.6 mg/L.

5. Both digester supernatant and filtrate from dewatering returned to the plant's headworks. The ortho-phosphorus in these streams averaged 6 percent of the plant's influent TP.

6. The equalization basin was sized to store 12.5 percent of the plant influent. The normal mode of operation is maintained during wet-weather periods.

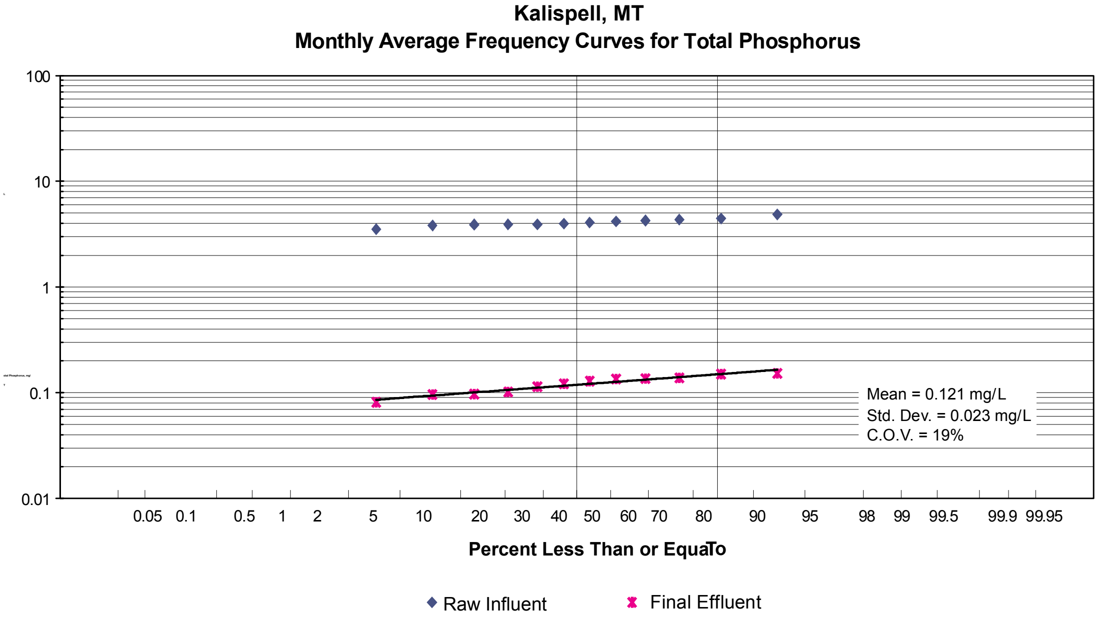

Figure 3-24. Kalispell, Montana: Monthly average frequency curve for TP.

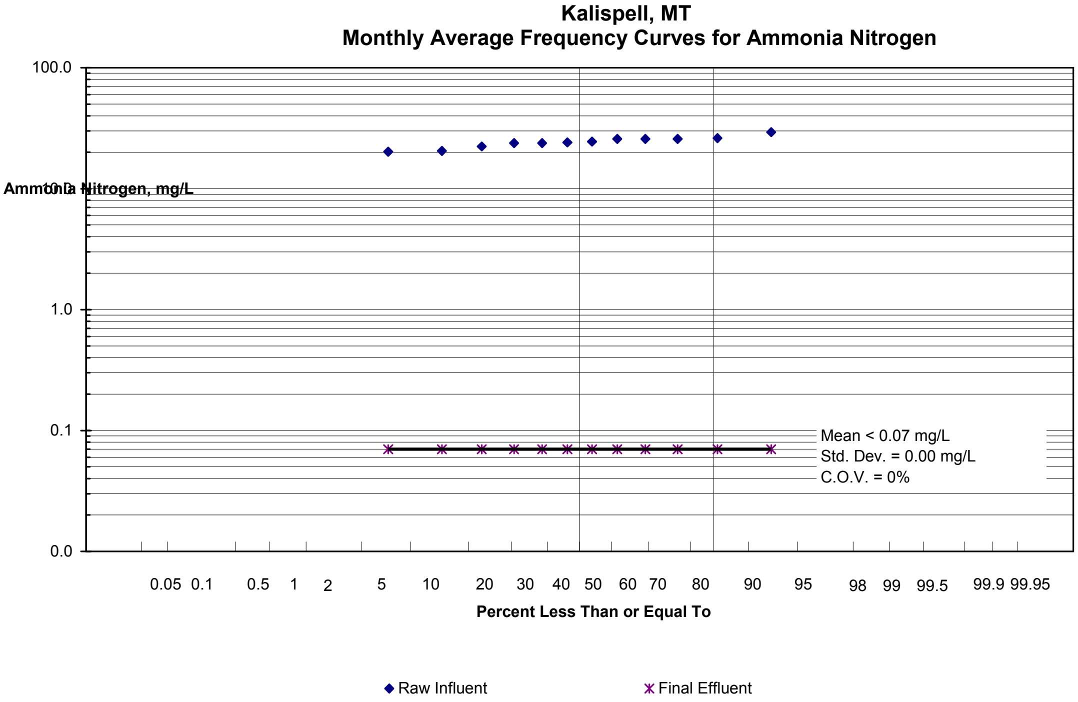

Figure 3-25. Kalispell, Montana: Monthly average frequency curve for ammonia nitrogen.

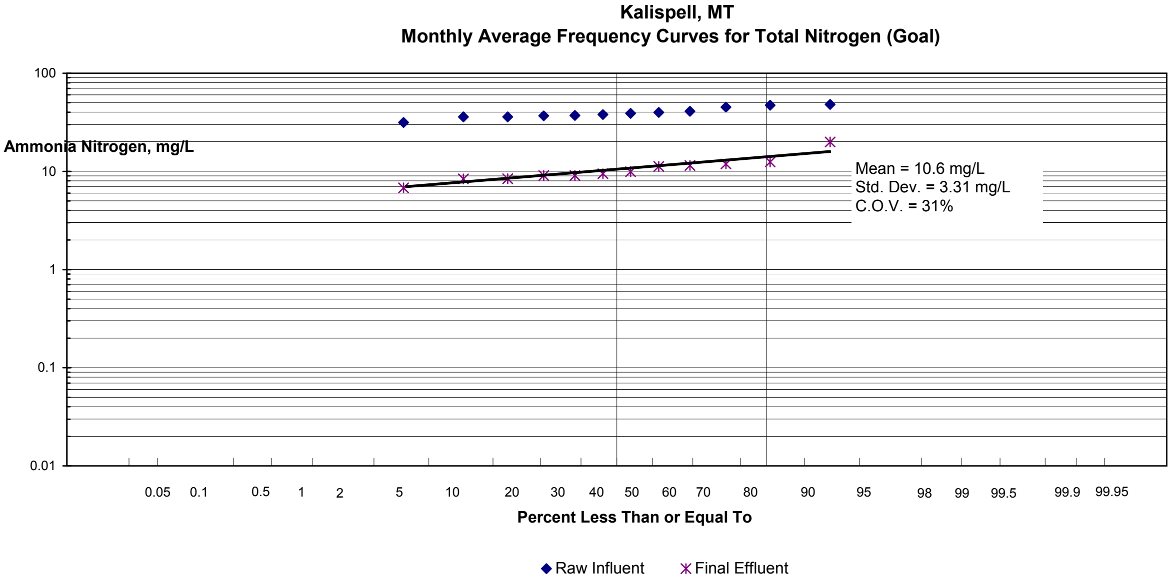

Figure 3-26. Kalispell, Montana: Monthly average frequency curve for TN.

3.3 Reliability Factors

3.3.1 Wastewater Characteristics

BOD-to-TP and BOD-to-TN Ratios

The BOD-to-TP ratios ranged between a low of 26.6 at Kelowna and a high of more than 50 at Kalispell on average for the year. There were monthly variations in these ratios at each facility because of weather conditions and the sewer system. The monthly variations in Fairfax County, Virginia, are shown in Table 3-3. To make the extent of variations clearer, the BOD-to-TP (or BOD-to-TKN) ratios were normalized by dividing the monthly values by the average of those values. This shows that the relative variation in values was small, between 90 percent and 110 percent of the average. The BOD-to-TP ratio remained between 28 and 33.5 in the plant influent and 24.4 and 33.8 in the primary effluent.

The BOD-to-TKN ratios ranged between a low of 4.3 at North Cary to a high of 10 at the Central Johnston County, North Carolina. The BOD-to-TKN ratio varied between 4.6 and 6.1 at Fairfax, Virginia, as shown below.

Table 3-3 shows that the BOD-to-TP ratio in the primary effluent went down as low as 24 in some months at the Fairfax, Virginia. The North Cary and Central Johnston plants, as well as the Fairfax County plant, decided to use EBPR and installed fermenters to ensure reliable operation. At the Clark County, Nevada, facility, the BOD-to-TP ratio was noted as a low 29 for the annual average and the primary settling tanks were converted to a thickener/fermenter for VFA generation.

Table 3-3. Monthly variation of wastewater characteristics at Fairfax County, Virginia

Date	INF BOD-to-TP	Normalized by avg	PE BOD-to-TP	Normalized by avg	INF BOD/TKN	Normalized by avg
Jan-06	33.1	1.1	29.8	1.0	6.1	1.1
Feb-06	33.7	1.1	29.5	1.0	5.4	1.0
Mar-06	28.3	0.9	27.4	1.0	5.3	1.0
Apr-06	28.2	0.9	27.1	0.9	5.5	1.0
May-06	27.2	0.9	26.8	0.9	4.7	0.9
Jun-06	28.9	1.0	24.4	0.9	5.5	1.0
Jul-06	28.8	1.0	26.1	0.9	5.9	1.1
Aug-06	29.4	1.0	28.1	1.0	4.6	0.9
Sep-06	31.5	1.0	32.4	1.1	5.0	0.9
Oct-06	33.5	1.1	33.8	1.2	4.9	0.9
Nov-06	32.2	1.1	32.0	1.1	5.4	1.0
Dec-06	28.2	0.9	26.3	0.9	5.4	1.0
Avg	30.3		28.7		5.3	

The low temperatures encountered by the case study facilities during the study period were 10 °C and 13 °C at the Kalispell, Montana, and Kelowna, British Columbia, facilities, respectively. Much warmer conditions occurred at plants in Florida.

At those facilities that depend on methanol feed for nitrogen removal or chemical feed for phosphorus removal, however, these characteristics in BOD (i.e., BOD-to-TP or BOD-to-TKN ratio) were not as critical in design or operation as at the other facilities for reliable performance.

3.3.2 Fermenter and VFA Generation

Three facilities had dedicated fermenters; one used primary settling tanks for VFA generation. The reliability of EBPR was ensured by having a fermenter in the facility.

- Kalispell, Montana, had a 2-stage fermenter with a variable SRT and HRT up to an SRT of 3–5 days with an HRT of 24 hours or less.
- Kelowna, British Columbia, and Fairfax County, Virginia, had gravity thickeners converted with an SRT of 3 to 5 days. The HRT averaged approximately 1 day (a 24-hour period).
- Clark County, Nevada, had primary settling tanks successfully serving as thickeners and fermenters at total solids concentrations of 5 percent.

The other facilities relied on VFAs found in the influent or generated from in-plant recycles.

The goal was to produce VFA of 18 mg/L in the influent to bioreactors so that a VFA-to-phosphorus ratio of 4 or higher could be maintained. Among these facilities, Kalispell and

Kelowna had biological phosphorus removal (no chemicals) and produced an annual average of 0.12 and 0.13 mg/L with COVs of 19 and 21 percent, respectively. Other facilities achieved lower concentrations but required chemical addition, filtration, or both.

3.3.3 Bioreactor Design and Process Parameters

The case studies demonstrated a range of process parameters that were designed in accordance with industry standards or were uniquely developed by plant personnel. These parameters are summarized below.

1. *SRT and HRT*: The bioreactor systems for the case study plants have a wide range of SRTs, from 5 days and more than 40 days in warm months to between 9 and 60 days in cold months. The Kelowna system was optimized for nutrient removal through the use of a denitrifying sludge blanket; when the system was operated in step-feed mode, shorter SRTs and HRTs were possible than in other systems. The HRTs ranged from 10 hours in Kelowna to 20 hours in Clearwater, Florida.

2. *Denitrifying sludge blanket in secondary clarifier*: A denitrifying blanket is maintained in three facilities. Denitrifying blankets ensured good phosphorus removal, alkalinity recovery, and shortening the anoxic zone in the bioreactor. Two facilities, however, maintain a no-blanket policy; the concern is secondary release of phosphorus (Kalispell and Clark County).

3. *Step-feed mode*: For both nitrogen and phosphorus removal, primary effluent was a good source of carbon, and thus the step-feeding of primary effluent accelerated the biological activities and reduced the size of the anoxic zone requirement. The feed locations varied from two at Kelowna to four at Fairfax, Virginia.

4. *Flexible design/swing zones*: The reliability of the biological process increases significantly in facilities with built-in swing zones. Kelowna and Clearwater have swing zones, and they allow adjustment on the basis of changing wastewater characteristics as well as the treatment objective. Kelowna evolved from the classic 5-stage Bardenpho to a 3-stage Westbank process using flexible design. It treats 70 percent more flow in the same original tanks. Clearwater has the flexibility to operate in either 5-stage Bardenpho mode for nitrogen and phosphorus removal or 4-stage Bardenpho mode for nitrogen removal. North Cary has the flexibility of separate mixing and aeration in the oxidation channel and thus can operate in accordance with wastewater flow and demand (summer vs. winter, or wet vs. dry days).

5. *Carbon source*: All the facilities use the internal carbon present in the wastewater, although some facilities have fermenters to increase VFA production. Three facilities use external carbon sources to support denitrification: two (Central Johnston County

and Lee County) use denitrifying filters and one (Western Branch) uses denitrifying activated sludge.

6. *Automatic controls and sensors*: In all facilities, DO control is universally practiced. All activated sludge plants have a network of DO probes and use the resulting data in making daily decisions on operation and control. Nitrate and ORP sensors are used in the facilities that remove nitrogen. Nitrate probes are in used in Central Johnston County, Clearwater, and Kelowna. An ORP probe is used in Clearwater and Kelowna. Standard procedures include target values for ORP, nitrate nitrogen, and DO for best nitrogen removal in these facilities.
7. *WAS management*: Sludge blanket meters are used in Clearwater, Florida, and Clark County, Nevada. At the Clark County facility, the plant computer (following pre-programmed operating criteria) makes daily WAS decisions for each of the nine activated-sludge units. At other facilities, samples are analyzed and operating decisions are made accordingly, with excellent results.
8. *Denitrification filter*: Central Johnston County, North Carolina, and Lee County, Florida, both employ downflow denitrification filters with supplemental carbon supplied by an automated methanol feed system. Both facilities pump the filter effluent to the next treatment process. Both systems have good reliability for both nitrogen removal and phosphorus removal to low levels via filtration.

3.3.4 Secondary Sludge Thickening

Secondary sludge thickening is one of the key factors in maintaining EBPR and minimizing the negative effect of recycle loads on the facility. For those facilities with EBPR, all but one uses DAF or a rotary-drum thickener and avoid phosphorus release.

3.3.5 Sludge Digestion

Sludge digestion is a critical element in reducing the recycle loads to the facility. Three facilities digest sludge aerobically: Central Johnston County, North Carolina; Lee County, Florida; and North Cary, North Carolina. The plants thereby minimize phosphorus and nitrogen release that would otherwise occur under anaerobic conditions. The Kalispell, Montana, facility digests primary sludge, but not secondary sludge, anaerobically. The remaining facilities digest sludge anaerobically and thus increase the recycle loads to the main plant. Two of these facilities, Kelowna and Fairfax County, have the capability to add lime to the cake and thus reduce the recycle of phosphorus. Kelowna eventually ceased lime addition when the recycle loads become low enough to manage.

3.3.6 Recycle Flows and Loads

All facilities with tertiary filters have provisions to equalize backwash water through existing or new tanks. The filtrate from dewatering is sent back to the main plant and could impose significant nutrient loads. Recycle loads were observed to range between 13 and 30 percent of TP and TKN, respectively.

The best cases for recycle loads were found at the Central Johnston County, North Carolina, and Lee County, Florida, facilities, where the secondary sludge is aerobically digested and sent away for off-site processing. These facilities do not practice primary settling.

The next-best case was found at the Kelowna, British Columbia, facility, where the secondary sludge is thickened aerobically with DAF and stored separately from the primary sludge. The primary sludge is fermented and stored for joint dewatering with the thickened secondary sludge. The final cake is sent away for composting. The recycle loads at Kelowna were the lowest of all the case study facilities at 13 percent of the influent TP load and 0.1 percent of the influent TN load.

The next best case was Western Branch, Maryland, where all the sludge is thickened aerobically by DAF, dewatered by centrifuge, and burned in incinerators. It is notable that recycle loads are received from dewatering back to the main plant but not from secondary sludge handling.

The next best case was found at the North Cary, North Carolina, facility, where the secondary sludge is digested aerobically and dewatered.

The next best case was found at Fairfax County, Virginia, where the secondary sludge is thickened aerobically while the primary sludge is fermented; the fermented primary sludge and the thickened secondary sludge are combined for dewatering. Lime can be added to reduce the phosphorus recycle load. The plant was designed to handle recycle loads of 13 percent of influent TP and BOD and 30 percent of influent TSS.

The next best case was found at the Clark County, Nevada, facility, where the secondary sludge is thickened aerobically and stored with the fermented primary sludge; they are subsequently dewatered together. Ferric chloride is added to the sludge feed.

Two facilities operate anaerobic sludge digestion and thus increase the opportunity for recycling ammonia nitrogen and phosphorous to the plant headworks. The Kalispell, Montana, facility digests primary sludge only before dewatering, and the Clearwater, Florida, facility digests both primary and thickened secondary sludge.

3.3.7 Wet-Weather Flow Management

Equalization Basin

Three facilities have some form of flow equalization. Fairfax County, Virginia, has two basins: one at the headworks area and one at the secondary effluent. They have capacities representing 11 percent and 20 percent of influent daily flow, respectively. North Cary, North

Carolina, has a large-capacity basin at the headworks. All told, the plant could equalize 28 percent of the influent daily flow by using the basin and existing unused tanks. Kelowna, British Columbia, has a basin with a capacity of 7.5 percent of the daily primary effluent flow. These basins ensure steady operation of the facility under high-flow conditions by reducing hydraulic surges in the settling tanks, providing stable chemical feed, and maintaining reliable process controls, where automated.

Process Design

Three plants have processes that have inherent advantages during high-flow periods. Fairfax County, Virginia, and Kelowna, British Columbia, both have step-feed activated-sludge systems, which means that high flows can be distributed through the aeration basins better than with a conventional feed plant. North Cary, North Carolina, has a PID, which offers the operators the ability to adjust the cycle time during wet-weather flow conditions.

Mode of Operation

Under emergency conditions, plant personnel can shut down portions of the aeration basins and thereby prevent solids from overloading the clarifiers, potentially causing a loss of biomass. The recovery back to normal operation is thereby ensured with the secured biomass inventory. The shutdown can be accomplished manually or by preprogrammed modes of operation.

3.3.8 Tertiary Filters

All facilities have tertiary filters, which help the facilities meet low TP concentration limits. Central Johnston County and Lee County, Florida, have denitrification filters, which differ from the traditional filters used at other locations. The filters at the other locations include rapid sand filters (two locations); dual-media filters with anthracite and sand (four locations); a deep mono-media filter (one location), and a dual sand filter (one location). More details can be found in Chapter 2 of this manual.

3.3.9 Tertiary Clarifier

Two facilities have tertiary clarifiers. Fairfax County, Virginia, has a tertiary clarifier before the tertiary filters. Ferric chloride is added to the clarifier to aid settling. The sludge from this process is recycled to the plant headworks. At Clark County, Nevada, the AWT plant operates a tertiary clarifier ahead of the sand filter. Alum is fed to the clarifier to aid settling. The sludge from this clarifier is thickened before being recycled to the headworks. For both facilities, the use of tertiary clarification provides added reliability for the subsequent tertiary filter operations.

Table 3-4 presents the wastewater characteristics and treatment processes of the case study facilities.

Table 3-4. Case study facilities' treatment processes

Facility	Fairfax, VA	Kalispell, MT	Clark Co., NV	Central Johnston, NC	N. Cary, NC	Kelowna, BC	Clearwater, FL	Lee Co. FL	Western Branch, MD
Wastewater characteristics									
BOD-to-TP ratio	29.5	54.9	29.	55.1	31.6	26.6	37.5	34.8	89.7 (COD/TP)
BOD-to-TKN ratio	5.4	5.7	5.5	10.	4.3	5.3	6.7	4.0	13.9 (COD/TN)
Temperature in °C	15–25	10–20	20–29	14–27	16–27	13–22	23–31	30	13–23
Equalization basin– percent of influent flow	11.5–20				58	7.5			
Equalization basin location	Raw/SE				Raw	Raw			
Bioreactors									
Bioreactor type	Step-feed AS	Mod. UCT	A/O	AS denit. filter	Oxid. ditch	Westbank	Bardenpho	AS/ denit. filter	Triple AS
SRT in bioreactor, days	16–19	8–12	5–9	7-9	11–14	6–9	25–44	24–59	6–60
Fermenter SRT, days	3	4–5	primary	No	No	3–6	No	No	
Swing zones	yes	yes				yes	yes	yes	
Mixing/aeration separated in bioreactor					yes	yes			
Denitrification blanket				yes		yes	yes	yes	yes
Denitrification filter				yes				yes	
External carbon source				Methanol				Methanol	Methanol
Alkalinity supplement	NaOH								
Online DO probe	yes	yes	yes	yes	yes	yes	yes	yes	yes
Online NO$_3$ probe				yes		yes	yes		
Online ORP probe						yes	yes		
Online ortho-P probe						yes	yes		
Tertiary Treatment									
Tertiary clarifier	yes		yes						
Tertiary filters									
Tertiary conventional sand filter		yes					yes		
Tertiary dual-media filter	yes		yes			yes			yes
Tertiary deep mono-media filter	yes								
Tertiary dual sand filter					yes				

Table 3-4. Case study facilities' treatment processes *(continued)*

Facility	Fairfax, VA	Kalispell, MT	Clark Co., NV	Central Johnston, NC	N. Cary, NC	Kelowna, BC	Clearwater, FL	Lee Co., FL	Western Branch, MD
Sludge Handling									
Primary settling / yes or no	yes	yes	yes	NO	NO	yes	yes	NO	NO
Secondary sludge thickening – aerobic	DAF	DAF	DAF			DAF	Rotary drum		DAF
Sec sludge gravity thickening/holding			yes						
Separate secondary/primary sludge storage						yes			
No dewatering on-site					yes				
On-site aerobic sludge digestion				yes	yes			Yes	
On-site anaerobic primary sludge digestion		Yes							
On-site anaerobic prim+sec. sludge dig.	Yes						Yes		
No on-site sludge digestion		yes	yes			yes			yes
Recycle Streams									
chemical treatment of filtrate/centrate	lime		$FeCl_3$			lime			
Filtrate/centrate recycle point									HDWK
Filter Backwash recycle point	primary			HDWK	HDWK			HDWK	HDWK

Notes:
BOD-to-TP = biochemical oxygen demand-to-total phosphorus ratio
BOD-to-TKN = biochemical oxygen demand-to-total Kjeldahl nitrogen ratio
DAF = dissolved-air flotation unit
DO = dissolved oxygen
HDWK = headworks
ORP = oxidation-reduction potential
SE = Secondary effluent
SRT = solids retention time

3.4 Cost Factors

The costs incurred for the studied facilities were analyzed to allow comparison on a consistent basis. Both capital and O&M costs were considered.

3.4.1 Capital Costs

Capital costs incurred for nutrient removal upgrades were provided by plant personnel at the studied facilities. If the upgrades were part of an overall program of upgrades at the plant (e.g., expansion, installation of UV disinfection), the best estimates were obtained of what was done for secondary treatment, tertiary filtration, and applicable sludge handling. The costs incurred in the past were updated on the basis of the *Engineering News-Record*'s Construction Cost Index, as provided in the U.S. Department of Agriculture Web site (USDA 2007). The capital costs for the plants are shown in Table 3-5. The land cost was not included in this evaluation.

The capital costs incurred for the biological nutrient removal (BNR) projects were allocated among phosphorus removal, nitrogen removal, and BOD removal. The allocation was done as follows:

1. If costs for particular pieces of equipment specifically for one nutrient could be obtained, that cost was allocated entirely to that nutrient. For example, the cost of the fermenter used at Kalispell was known and attributed entirely to phosphorus removal. Another example is denitrificaiton filters entirely for nitrogen removal.

2. If particular pieces of equipment could be allocated to two nutrients, that was done with an even split. For example, the mixers used at Kalispell were just for anoxic tanks, where nitrogen and BOD would be removed. In other cases, the installation was clearly for both phosphorus and nitrogen removal.

3. For all instances where no equipment breakout was available or the equipment would go toward all treatment, the costs were allocated by the percentages 12 percent to phosphorus, 48 percent to nitrogen, and 40 percent to BOD. This allocation was based on an estimate-of-cost breakdown for Kelowna and provided a consistent basis for comparing relative costs. For denitrifying filters, all the BOD percentage was attributed to nitrogen, making the split 12 percent phosphorus and 88 percent nitrogen.

4. For Clearwater (Marshall Street), the cost fractions were 17, 63, and 20 percent for phosphorus, nitrogen, and BOD, respectively. In that instance, the total cost of the BNR upgrades was known, as were the volumes of the various vessels. Those volumes were attributed to phosphorus or nitrogen removal (fermenter to phosphorus, anoxic basins to nitrogen), while the aeration basin was apportioned 10 percent for phosphorus, 50 percent for nitrogen, and 40 percent for BOD. The fraction of the total volume for each nutrient was used for the cost apportionment.

Table 3-5. Case study facilities' capital costs

Facility	Fairfax, VA	Kalispell, MT	Clark County, NV	Central Johnston, NC	North Cary, NC	Kelowna, BC	Clearwater, FL	Lee County, FL	Western Branch, MD
Design flow MGD	67	3	110	7	12	10.5	10	5	30
Influent P (mg/L)	6.4	4.1	5.8	5.8	7.7	6.0	5	3.85	3.7
Influent N (mg/L)	34.6	39.6	30.3	31.2	56.4	28.8	44.8	33.2	23.9
Influent NH_3-N (mg/L)		24.3	26.8	28	45.5	21.3	28	27.2	19.6
Capital attributed to P (2007 dollars)	$0	$1,190,000	$65,450,000	$889,000	$4,089,600	$6,818,000	$5,019,000	$480,000	$2,464,000
Capital attributed to N (2007 dollars)	$71,600,000	$4,310,000	$56,980,000	$2,400,000	$16,358,000	$27,273,000	$18,600,000	$13,460,000	$49,583,000
Capital attributed to other (2007 dollars)	$0	$3,600,000	$98,570,000	$0	$13,632,000	$22,728,000	$5,905,000	$11,630,000	$84,906,000
Electrical attributed to P (kWh/yr)	3,360,000	389,000	*	1,843,000	377,400	884,000	931,000	62,700	262,400
Electrical attributed to P (dollars/yr)	$185,000	$17,500	*	$103,200	$17,400	$41,500	$102,400	$7,500	$26,200
Electrical attributed to N (kWh/yr)	18,060,000	1,077,000	*	4,169,000	2,558,000	4,100,000	4,623,000	1,911,500	8,865,000
Electrical attributed to N (dollars/yr)	$993,000	$48,500	*	$233,500	$118,000	$193,000	$509,000	$192,000	$886,500
Electrical attributed to other (kWh/yr)	0	1,045,600	*	588,000	0	919,000	2,075,000	2,162,000	16,066,000
Electrical attributed to other (dollars/yr)	$0	$47,100	*	$33,000	$0	$43,200	$230,000	$259,000	$1,607,000
Chemical attributed to P (dollars/yr) (alum, ferric)	$393,000	$5400	*	$0	$0	$0	$7,400	$34,600	$107,000
Chemical attributed to N (dollars/yr) (Methanol, pH control caustic as needed)	$250,000	$0	*	$53,100	$0	$0	$0	$83,900	$425,000
Sludge attributed to P (tons/yr) (alum/ferric or biological uptake sludge)	730	0	*	0	90	0	5.8	37.5	72.9

Table 3-5. Case study facilities' capital costs *(continued)*

Facility	Fairfax, VA	Kalispell, MT	Clark County, NV	Central Johnston, NC	North Cary, NC	Kelowna, BC	Clearwater, FL	Lee County, FL	Western Branch, MD
Sludge attributed to P (dollars/yr)	$393,000	$0	*	$0	$17,900	$0	$1,500	$21,900	$32,400
Sludge attributed to N (methanol conversion) (tons/yr)	0	0	*	60	1702	0	0	14.1	839
Sludge attributed to N (dollars/yr)	$0	$0	*	$15,100	$341,000	$0	$0	$8,300	$372,000
Total O&M attributed to P (dollars/yr)	$970,000	$17,500	$3,004,000	$103,200	$35,300	$40,300	$109,800	$64,000	$165,000
Total O&M attributed to N (dollars/yr)	$1,243,000	$57,800	$3,852,000	$301,700	$230,000	$109,000	$509,000	$284,200	$1,311,000
Annualized capital for P (6%, 20 years)	$0	$168,100	$5,710,000	$51,000	$357,000	$595,000	$438,000	$41,900	$215,000
Annualized capital for N (6%, 20 years)	$6,240,000	$323,600	$4,970,000	$302,700	$1,426,000	$2,378,000	$1,620,000	$1,174,000	$4,324,000
Unit O&M for TP ($/lb)	$1.07	$0.49	$1.81	$1.48	$0.23	$0.27	$1.37	$1.77	$0.78
Unit capital For TP ($/lb)	$0.00	$2.84	$3.43	$0.73	$2.28	$3.97	$5.38	$1.16	$1.01
Unit total for TP ($/lb)	$1.07	$3.33	$5.24	$2.21	$2.51	$4.24	$6.76	$2.93	$1.79
Unit O&M for TN ($/lb)	$0.29	$0.19	$0.43	$0.49	$0.41	$0.14	$0.75	$0.91	$0.99
Unit capital for TN ($/lb)	$1.47	$1.46	$0.55	$0.49	$2.54	$3.05	$2.40	$3.87	$3.27
Unit total for TN ($/lb)	$1.76	$1.65	$0.98	$0.98	$3.36	$3.19	$3.15	$4.78	$4.26
Unit capital cost ($/gal/day)	$1.07	$3.03	$2.01	$0.58	$2.84	$3.25	$2.95	$2.79	$1.73

Note: Cost data available per unit operation; boldface type indicates actual dollar amounts.

The apportioned costs are detailed in each case study and summarized in Table 3-5. Table 3-5 also presents the annualized capital cost for phosphorus and nitrogen, which was done at a 6 percent interest rate for 20 years.

3.4.2 Operation and Maintenance Costs

The case studies considered only three categories of O&M costs: power/electrical usage, chemical usage, and the cost of disposing of extra sludge generated by the nutrient removal processes. All other costs, including labor, were excluded from consideration. Labor was excluded for three reasons:

1. Labor and energy costs are highly sensitive to local conditions, such as the prevailing wage rate, the relative strength of the local economy, the presence of unions, and so on; thus, they would only confound comparison of the inherent cost of various technologies.
2. For most processes, the incremental extra labor involved in carrying out nutrient removal is recognized but not significant, in view of the automatic controls and supervisory control and data acquisition (SCADA) system that came with the upgrades. For example, if an activated-sludge system is now to be operated to remove nutrients, the amount of monitoring, checking of basins, and making process decisions is very similar. Adding an entirely new process could require additional labor hours per week, but that is very site-specific and difficult to determine and compare. With nutrient removal upgrades, many facilities added automation features along with SCADA system and thus the need for additional labor is minimal.
3. Additional labor-hour determination is difficult because the plants were largely unable to break down which extra personnel were employed because of nutrient removal and related overtime costs.

Electrical costs were estimated on the basis of direct plant data or the power draw of the blowers/aerators, mixers, and pumps used for treatment. The power draw, in kilowatt-hours per year (kWh/year), was allocated among phosphorus, nitrogen, and BOD removal. The allocation was done in the same manner as capital allocation:

1. If the piece of equipment was for one nutrient, those kWh/yr were allocated entirely to that nutrient. That was the case for mixers in an anaerobic tank or a fermenter.
2. For aeration blowers, the allocation of kWh was based on the oxygen demand by PAOs, nitrifiers, and standard heterotrophic organisms. In this manner, the allocation 10 percent for phosphorus, 50 percent for nitrogen, and 40 percent for BOD was developed.

For other pieces of equipment, the distribution of kWh was the same as the main capital distribution: 12 percent for phosphorus, 48 percent for nitrogen, and 40 percent for BOD.

The electrical cost for each nutrient was determined by multiplying the kWh/year by the average electrical cost, as supplied by plant personnel. This average cost covers the actual consumption charge (kWh) and the peak demand charge imposed by the local utility.

If a plant had to add chemicals for nutrient removal, the cost and amount of the chemicals were supplied for the case study. The chemicals used by these municipalities were alum and ferric chloride for phosphorus removal and methanol for denitrification. One plant (Noman M. Cole in Fairfax County, Virginia) also used sodium hydroxide as needed to control the pH following nitrification.

If a plant generated additional sludge because of nutrient removal, the cost of disposing of that additional sludge was included. In general, if an anaerobic or anoxic process is used for nutrient removal, the overall amount of sludge should go down compared to a fully aerobic process, and those processes inherently generate less sludge. Additional sludge would be generated if phosphorus was removed chemically or if methanol (or other external carbon source) was added for denitrification. The sludge yield of 0.2 lb/lb BOD was used for methanol feed, where applicable.

Table 3-5 presents the annual O&M estimates for nutrient removal at the case study facilities.

3.4.3 Unit Costs

The annualized capital costs and annual O&M costs were normalized by dividing by the mass of nutrient removed during the study period. These normalized costs became the Unit O&M and Unit capital costs presented in Table 3-5, with the cost expressed as dollars per pound of nutrient removed. The calculation was done for TP and TN. As a comparison, the unit capital cost for the BNR expansion was calculated as the dollars per gallon per day capacity. The following observations can be made about the results in the table:

1. The unit O&M cost for TP ranged from \$0.23 per pound for North Cary, \$0.27 per pound in Kelowna, and \$0.49 per pound in Kalispell to \$1.81 for Clark County at the high end. The low cost in North Cary is because of fully functioning biological phosphorus removal. The low costs for Kelowna and Kalispell are because of those plants having fully functioning biological phosphorus removal aided by a fermenter (so chemicals are not needed). The trade-off for the low operating costs was higher capital costs (\$3.05 per pound phosphorus and \$1.48 per pound phosphorus for Kelowna and Kalispell, respectively). The capital cost at North Cary was moderate at \$2.35 per pound.
2. The capital cost range for TP was \$0 for the Noman M. Cole plant in Fairfax County to high-end costs of \$3.05 per pound for Kelowna, \$3.27 per pound in Western

Branch, and $3.87 per pound in Lee County. The costs for installing phosphorus removal processes at Noman M. Cole, an older plant, were fully depreciated and were not available.

3. The unit O&M for nitrogen removal is low, with costs ranging from $0.14 to $0.99 per pound removed. Higher costs are associated with having to add chemical, while the plants with the lowest costs for nitrogen removal (Kelowna and Kalispell) are able to effect total nitrification and denitrification without chemical addition.
4. The capital cost for nitrogen removal is low, ranging from $0.49 to $3.87 per pound, with the lowest capital cost for TN removal at Central Johnston County, where the retrofit was based substantially on existing facilities with addition of denitrification filters. High-value locations involved major additions, and they included Lee County, Western Branch, and Kelowna, at $3.87, $3.27, and $3.05, respectively.
5. The overall unit capital cost for these plants on a flow basis ranged from $0.58/gallon per day (gpd) for Central Johnston County to $3.25/gpd for Kelowna.

The factors affecting capital costs are as follows:

1. *The treatment to be added*: The major factor is the availability of existing facilities. In those facilities with available site and tanks, much can be retrofitted without significant cost. If a new biological treatment is added, however, it adds to higher capital cost. Chemical treatment requires less capital.
2. *Built-in flexibility in the design*: Some plants include fermenters for EBPR or swing zones with two types of equipment and two modes of operation for nitrogen removal, such as the 4-stage and 5-stage Bardenpho process. The added flexibility increases capital costs but allows operators to potentially reduce O&M costs through reductions in energy or chemical usage.
3. *Equalization basin*: Basins are expensive capital items but could reduce O&M costs because of lower peak demands for power and chemical.
4. *Treatment of recycle flows*: Equipment and buildings for lime or ferric addition are expensive.
5. *The amount of automation*: Depending on the size of the facility, this expense could be expensive but could be beneficial in saving O&M costs.
6. *Separate storage tanks for the secondary sludge from the primary sludge*: This is an added cost at EBPR facilities to consider for control of recycle loads.

The factors affecting the O&M costs are the following:

1. *Use of external supplemental carbon*: If methanol must be purchased, that will tend to push the costs up, compared to use of in-stream carbon generated by a fermenter or fed to the anoxic zone by a step-feed mechanism.
2. *Use of chemical for phosphorus removal*: As with methanol addition, if the plant has to use chemical phosphorus removal to meet the discharge limit, it will tend to have a higher cost than if biological phosphorus removal can be used. Ideally, the biological phosphorus removal will use site-generated VFAs for the carbon source in the anaerobic zone; if such are not available, chemical addition will be required, at greater cost.
3. *Alkalinity Addition for nitrification*. Caustic soda is added to supplement alkalinity in those areas with soft water (low alkalinity).
4. *Use of a fermenter*: In some cases, sufficient VFAs exist in the influent and in-plant recycle stream so that the EBPR does not need additional VFAs. If some must be added, they are typically generated by fermenting primary solids. This fermentation can take place in a dedicated unit that, therefore, has associated operating costs, or in some cases, the fermentation can occur in a sludge thickener, with less electrical use than a dedicated unit.
5. *Automation*: Automation adds cost but could save O&M costs by optimizing power usage and chemical usage.

3.5 Summary

The performance and costs of the treatment processes at the nine case study facilities are summarized in the sections that follow.

3.5.1 Discharge Permits

The discharge permits are diverse in the number of discharge seasons, allowable nutrient discharge limits, and averaging period. Some of these diverse permit requirements are based on TMDLs; others are based on a state policy for water reuse (such as Florida’s), or a regional goal (such as the goals for the Chesapeake Bay and the Long Island Sound).

An in-depth evaluation was made of full-scale plant performance, and the performance was presented using a simple statistical basis for comparison and interpretation. The coefficient of variation, or COV, represents one standard deviation divided by the mean. With this, all performance data for one year were presented for these selected facilities.

All nine facilities provided one year of data, and the monthly average values were used in the summary as the base. Weekly averages were also used where a permit limit was specified.

3.5.2 Phosphorus Removal

The discussion below summarizes the highlights of efficiency and reliability related to phosphorus removal that was accomplished at the facilities. Four facilities removed phosphorus biologically, without chemical addition. Three facilities used chemicals to supplement biological phosphorus removal. Two facilities removed phosphorus primarily with chemical addition. The first four facilities relied on biological phosphorus removal followed by tertiary filters. They met the permit limits and performed efficiently and reliably.

- Kalispell, Montana: 0.12 mg/L TP at a COV of 19 percent
- Kelowna, British Columbia: 0.14 mg/L TP at a COV of 21 percent
- Central Johnston County, North Carolina: 0.26 mg/L TP at a COV of 62 percent
- North Cary, North Carolina: 0.38 mg/L TP at a COV of 64 percent

Both Kalispell and Kelowna have fermenters, operate year-round, and show very high reliability. Two facilities without fermenters produced good effluent but at an increased COV, or reduced reliability compared to those with fermenters.

With small amounts of chemical addition, biological phosphorus removal was taken to even lower effluent concentrations and increased reliability, as follows:

- Clark County, Nevada: 0.09 mg/L TP at a COV of 30 percent
- Fairfax, Virginia: 0.09 mg/L TP at a COV of 21 percent
- Clearwater, Florida, Marshall Street: 0.13 mg/L TP at a COV of 40 percent

Clark County, Nevada, used primary settling tanks as VFA generators, and Fairfax County, Virginia, had a fermenter online. Both showed lower COVs than the others at varying chemical dosages. Clearwater, Florida, fed alum to remove trihalomethanes for water reuse purposes, and Clark County, Nevada, added alum to push the limit of the existing facility.

Two facilities used chemicals for phosphorus removal:

- Lee County, Florida: 0.10 mg/L TP at a COV of 35 percent
- Western Branch, Maryland: 0.43 mg/L TP at a COV of 62 percent

These facilities remove nitrogen in an add-on process with methanol, and thus a decision had been made to remove phosphorus with chemical addition instead of retrofitting the existing processes for biological phosphorus removal.

The factors for reliable operation include the following:

1. *Favorable wastewater characteristics*: The BOD-to-TP ratio in the influent and primary effluent.
2. *Fermenters that ensured an adequate supply of carbon*: Ensured reliability during cold months and wet-weather periods. Kalispell, Montana, and Kelowna, British Columbia, achieved the highest reliability, followed by Fairfax County, Virginia, and Clark County, Nevada, which use active primary settling for sludge thickening and VFA production, both with chemical addition.
3. *Flexible design*: Flexible design with swing zones in anticipation of changing conditions in weather and wastewater characteristics, including
   - Equipping aeration tanks with both mixers and aerators for separate control, like North Cary, North Carolina
   - Compartmentalizing the basin and creating swing zones, like Kalispell, Montana, and Kelowna, British Columbia
   - Adding multiple feed points for carbon feed, like Fairfax County, Virginia, for step-feed
   - Placing a tertiary clarifier ahead of the tertiary filter to reach a low concentration limit
   - Having multiple chemical feed points
4. *Flow equalization for stable operation*: The facilities have equalization available, with the amount of storage varying based on their sewer systems. Kelowna has an equalization basin with volume 7.5 percent of the design flow, based on a relatively *tight* sewer system that has low infiltration. Fairfax County, Virginia, has two basins with a total volume of 31 percent of the design flow.
5. *Wet weather*: Storm flows were handled favorably with step-feed activated sludge at Fairfax County, Virginia, and the PID at North Cary, North Carolina. In addition, North Cary had the largest basins to store wet-weather flows—a 7-MG retention basin and a 2-MG equalization basin for the 12-MGD facility). The operational controls in many facilities included shutting off a section of the aeration tanks during peak flow periods and protecting the biomass inventory to prevent solids washout. This emergency shutoff mode of operation lasted up to 24 hours but was followed by a successful restart, meeting all permit limits.
6. *Recycle loads were minimized by*
   - Keeping secondary sludge aerobic until dewatering; thickening, storing, and aerobic digestion—Central Johnston County, North Carolina; Lee County, Florida; North Cary, North Carolina; and Kalispell, Montana.

  - Treating filtrate with lime or ferric chloride—Fairfax County, Virginia, and Clark County, Nevada. Kelowna, British Columbia, no longer adds lime.
  - No sludge digestion on-site—Kelowna, British Columbia, and Clark County, Nevada.

7. *Operating denitrification blanket in secondary clarifiers*: Four facilities have developed operating procedures to maximize the denitrification of RAS in the clarifier and have made it a permanent practice for successful biological phosphorus removal: Kelowna, British Columbia; Central Johnston County, North Carolina; Lee County, Florida; and Clearwater, Florida.

8. *Automation of process controls with sensors*: Some facilities have multiple sensors, with automated process controls using the resulting signals. Commonly monitored parameters were DO, ORP, and nitrate in the wastewater and the sludge blanket. Clark County, Nevada, used the daily results to control the blowers and the sludge age. Other facilities rely on a daily check of critical parameters and take steps to optimize the operation with efficiency and achieve a high degree of reliability.

3.5.3 Nitrogen Removal

For nitrogen removal, the following paragraphs summarize the efficiency and reliability accomplished at the case study facilities.

Two facilities that are required to remove only ammonia nitrogen met the permit limits and performed efficiently and reliably:

- Kalispell, Montana: 0.07 mg/L ammonia nitrogen at a COV of 0 percent
- Clark County, Nevada: 0.12 mg/L ammonia nitrogen at a COV of 14 percent

Seven facilities are required to remove TN. These facilities met the permit limits and performed efficiently and very reliably:

- Central Johnston County, North Carolina: 2.14 mg/L TN at a COV of 16 percent
- Lee County, Florida: 1.71 mg/L TN at a COV of 28 percent
- Clearwater, Florida, Marshall Street: 2.32 mg/L TN at a COV of 16 percent
- North Cary, North Carolina: 3.7 mg/L TN at a COV of 14 percent
- Kelowna, British Columbia: 4.38 mg/L TN at a COV of 12 percent
- Western Branch, Maryland: 1.63 mg/L TN at a COV of 36 percent
- Noman Cole, Fairfax County, Virginia: 5.25 mg/L TN at a COV of 12 percent

Reliability factors for these performances included the following:

1. *Favorable wastewater characteristics*: BOD-to-TKN ratios of above 4 in all cases. For add-on processes with methanol addition, however, this ratio was not a concern. Methanol dosage at three such facilities depended directly on the nitrate nitrogen coming into the add-on process.

2 *Adequate supply of carbon*: Central Johnston County, North Carolina; Western Branch, Maryland; and Lee County, Florida, need a separate stage for denitrification and all feed methanol. Other plants use carbon present in their wastewater.

3. *Flexible design*: This ensures added reliability in anticipation of variations in the weather and in wastewater characteristics.
   - Swing zones at Kelowna, British Columbia
   - Separate control of mixing and aeration at North Cary, North Carolina
   - Multiple carbon feed points (step-feed) at Fairfax County, Virginia; Kelowna, British Columbia; and Clearwater, Florida
4. *Flow equalization*: The same principles that apply to phosphorus removal also apply to nitrogen removal. The backwash water from the denitrification filter is sent to a basin for equalization. In Central Johnston County, North Carolina, the backwash water is stored in filter backwash reclaim tanks before it is pumped to the headworks. In Lee County, the backwash goes to a mud well before it is returned to the headworks.

5. *Recycle loads*: The best strategy is to minimize recycle loads:
   - Keep sludge handling aerobic in thickening, storing, and stabilization
   - Process all sludge each day: Clark County, Nevada
   - Provide no sludge digestion on-site: Kelowna, British Columbia; Clark County, Nevada; and Western Branch, Maryland
   - Treat the recycle loads: lime addition at Fairfax County, Virginia, and Kelowna, British Columbia, and ferric chloride at Clark County, Nevada
   - It is important to note that Fairfax County, Virginia, accounted for the recycle loads in their retrofit or expansion design
6. *Wet-weather flows*: The step-feed and PID offer the best process protection against wet-weather flows. North Cary, North Carolina, has the largest storage capability, as described above. The operational controls in many facilities included shutting off a section of aeration tanks during the peak flow periods and protecting the biomass inventory to prevent solids washout. This emergency shutoff mode of operation lasted up to 24 hours but was followed by a successful restart in meeting all permit limits.

7. *Denitrification blanket in secondary clarifier*: A denitrification blanket ensures good nitrate removal and alkalinity recovery; it also optimizes the anoxic zone in the bioreactor, allowing the anoxic basin size to be reduced.

8. *Automatic controls with multiple sensors*: Some facilities have multiple sensors, and process controls were based on them. They include DO, ORP, and nitrate in the wastewater and sludge blanket. Clark County, Nevada, uses these data to control the blowers and the sludge age. Clearwater, Florida, uses these data on DO, ORP, and nitrate. The target limits are set for these three parameters at the anoxic zone for reliable nitrogen removal.

3.5.4 Costs for Capital and O&M

Capital Cost

The capital cost at the case study facilities for nutrient removal was estimated on the basis of consultation with the owners because all the upgrades were a part of an overall expansion. The estimate was obtained for secondary treatment, tertiary clarification and filtration, and applicable sludge handling. The capital costs incurred for nutrient removal were then allocated among phosphorus removal, nitrogen removal, and BOD removal. There is a wide variation in costs because of the type and age of the existing facility.

1. *Unit cost for treatment capacity*: This cost varied from a low of $0.58 to a high of $3.25/gpd capacity according to the existing facility and the age of construction. The variation was based on the capability of the existing facility, the desired level of treatment, and when the plant modifications were carried out.

2. *Unit cost for TP removal*: This cost varied from zero to a high of $5.38 per pound of TP removed.

3. *Unit cost for TN removed*: This cost varied from a low of $0.49 to a high of $387 per pound TN removed.

O&M Costs

This study included three categories of O&M costs: power/electrical usage, chemical usage, and cost of disposing of extra sludge generated by nutrient removal processes. Labor was excluded because of the high variation in geographic location, difficulty in allocating the fraction for nutrient removal, and an understanding that the staff size did not change after upgrade.

1. *Unit cost for phosphorus*: This cost ranged from a low of $0.23 to $0.1.81 per pound TP removed.

2. *Unit cost for TN removal*: This cost ranged from a low of $0.14 to a high of $0.99 per pound TN removed.

Contributing factors include the following:

1. The biological phosphorus removal facilities added capital cost, especially where fermenters were installed, but ensured a high degree of reliability in performance. The O&M cost was lower than that for the other processes.
2. Flexibility in design, such as the swing zone and alternate mode of operation, added costs (and redundant equipment and structures) but provided added capability to handle changing wastewater characteristics and new regulations.
3. Equalization basins added costs but provided a higher degree of treatment for captured flows in the basin.
4. Separate storage tanks for secondary sludge and primary sludge added costs but ensured more reliable performance, especially at EBPR facilities.
5. Treatment of recycle flows with lime or ferric chloride added costs but resulted in more reliable performance.

Total Annual Cost

The annual costs were estimated on the basis of a 6 percent interest rate for 20 years.

1. The unit cost for phosphorus removal ranged from a low of $1.07 to a high of $6.76 per pound TP removed.
2. The unit cost for TN removed ranged from a low of $0.98 to a high of $4.78 per pound TN removed.

The contributing factors described in the above section on Capital and O&M costs all apply to the total cost.

3.6 References

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CHAPTER 4: Cost Factors

This chapter summarizes the cost factors to be considered in evaluating alternative technologies for upgrades at existing facilities and, more broadly, provides context on cost for the entire document. Costs vary widely, depending on a number of factors, including the target concentrations for nitrogen and phosphorus, as well as the existing facilities' suitability to be used for upgrades.

As also discussed in Chapter 5, there are three general types of possible plant upgrades to be considered for costs: retrofitting existing facilities with additional piping and equipment; adding a new process or technology to an existing treatment train; or expanding the existing plant, possibly with an entirely different technology. Section 4.1 describes costs reported at selected facilities in the United States, as well as costs incurred by facilities included in the cases studies provided in Chapter 3. Section 4.2 describes the results of estimating costs for 12 retrofit alternatives for nutrient removal, with the estimates generated using CAPDETWorks software. Finally, Section 4.3 describes the estimated costs for 20 expansion alternatives for nutrient removal, again with the estimates generated using CAPDETWorks. In all cases, capital and operation and maintenance (O&M) costs are provided as available. In addition, Section 4.3 provides a breakdown of the relative fraction of various components of O&M costs, i.e., labor, chemical, and energy costs.

4.1 Modifying Existing Facilities

4.1.1 Literature Review

Modifying or retrofitting existing facilities is often the least costly and most environmentally favorable approach for wastewater treatment facilities that are required to implement nutrient removal. Maryland (MDE 2004) and Connecticut (CTDEP 2006) have both implemented programs to assist treatment plants in upgrading processes for nutrient removal. Note that the construction industry indices of *Engineering News-Record* were 7,298.25 for September 2004 and 7,700 for July 2006 for comparative purposes. Table 4-1 shows the costs for various technologies used in the upgrades. For each technology, the following are shown: the size of the facility (in million gallons per day [MGD]), the capital cost (in dollars per gallon per day [gpd] capacity), the O&M cost per MGD day treated, and the O&M unit cost for removal of 1 pound of nitrogen. The upgrades cost between \$0.22 and \$5.20 per gpd capacity. The Maryland data consist of projected costs to upgrade from the current treatment process. The O&M costs from Maryland, where available, are based on only the projected increase in electricity and chemical usage. Specifically, labor costs and any anticipated increases in sludge-handling or disposal costs are not included in the Maryland data.

Table 4-1. Upgrade costs for Maryland, Connecticut, and others

Treatment processes	Location	Actual effl. TN (ppm)	Initial TN conc. (ppm)	New TN goal (ppm)	Flow (MGD)	Capital Cost $/gpd capacity	O&M Cost $/MG treated	O&M Cost $/lb TN removed
Cyclic on/off	Ridgefield, CT	--	9.6	5.1	1	$0.20	$111	$0.60
Added upflow denitrification filter	Cheshire, CT	6	7	5	3.5	$1.65	$136	$1.05
MLE → 4-stage Bardenpho	Seneca, MD	16	8	3	20	$0.21	$63	$1.51
MLE → 4-stage Bardenpho	Freedom, MD	8	8	3	3.5	$0.99	--	--
Step-feed AS → 4-stage Bardenpho	Cumberland, MD	7.3	8	3	15	$1.10	$122	$2.94
MLE → added denite. filter	Baltimore, MD	9	8	3	180	$1.39	--	--
MLE → added denitrification filter	Cox Creek, MD	16	8	3	15	$1.74	$104	$2.50
A^2O → 5-stage Bardenpho + denitrification filter	Frederick, MD	23	6.5	3	7	$1.41	--	--
Post-MBBR for TN removal[a]	Broomfield, CO	8	8	3	8	$1.70	--	--
On/off aeration[a]	Broomfield, CO	--	8	3	8	$1.00	--	--
IFAS[a]	Broomfield, CO	--	8	3	8	$0.85	--	--
Lagoon → 4-stage Bardenpho	Hurlock, MD	--	15	3	1.5	$4.12	--	--
MLE → MBR/5-stage Bardenpho[a]	Las Virgenes Calabasas, CA	--	42	1	16	$5.20	--	--

Notes:

A^2O = anaerobic/anoxic/oxic
AS = activated sludge
IFAS = integrated fixed-film activated sludge
MG = million gallons
[a]Stephenson and Mohr 2005

MBBR = moving-bed biofilm reactor
MBR = membrane biological reactor
MLE = modified Ludzack-Ettinger
TN = total nitrogen

O&M cost data are presented for facilities that had such information included in the studies reviewed.

The Broomfield, Colorado, and Las Virgenes Municipal Water District (in California) capital costs are also based on projections. The Connecticut data are based on actual costs to upgrade the plant. The O&M costs for the Connecticut plants are an estimate of the actual costs associated with nitrogen removal. The O&M costs represent the total O&M costs for nitrogen removal, not the incremental increase from the upgrades. Specifically, the Ridgefield, Connecticut, O&M costs include electricity, sludge-handling and disposal, labor, maintenance, and administrative costs associated with nitrogen removal at the plant. No chemicals are added at this facility. O&M costs for Cheshire, Connecticut, include electricity, chemicals, and laboratory costs associated with nitrogen removal at the plant.

4.1.2 Case Studies

Table 4-2 presents the cost results for the upgrades done at the facilities examined in the case studies found in Chapter 3 and Appendix A (in Volume II) of this document. The table shows the flow, the capital cost in dollars per gallon of treatment capacity, the O&M cost in dollars per million gallons (MG) treated, and the unit O&M costs for removal of a pound of nitrogen, phosphorus, or ammonia nitrogen, as applicable. The capital costs were determined by totaling the capital expenditures for nutrient removal technologies (costs brought to present worth by using cost indices) and then dividing that total by the gpd capacity of the plant. The construction index of the *Engineering News-Record* was 7,959.17 in July 2007. The O&M costs for the case studies included power/electricity usage, chemical usage, and sludge disposal costs associated with the nutrient being removed (total nitrogen [TN], total phosphorus [TP], or ammonia nitrogen). Labor costs were not included in the O&M costs. The O&M costs for nitrogen, ammonia, and phosphorus for a year were divided by the pounds of those substances removed over the year to obtain the unit costs. Similarly, the total O&M dollars expended over the year were divided by the total flow processed at the plant over the year to obtain the dollars per MG treated.

Table 4-2. Upgrade costs for case studies

Process	Location	Design flow (MGD)	Target Concentration (annual average)	Initial Concentration (annual average)	$/gpd capacity	$/MG	O&M $/lb TN	O&M $/lb TP	O&M $/lb NH_3
Step-feed AS with tertiary clarifier and filter	Fairfax County, VA	67	Ammonia-N+TP, 1 mg/L & 0.18 mg/L (TN 5.2 mg/L voluntary)	18.9 mg/L ammonia-N, 6.4 mg/L TP	$1.07	$106	$0.46	$1.07	--
Modified UCT with fermenter + tertiary filter	Kalispell, MT	3	Ammonia-N+TP, 1.4 mg/L & 1 mg/L	24 mg/L ammonia-N, 4 mg/L TP	$3.03	$108	--	$0.49	$0.22
A/O with tertiary clarifier + tertiary filter	Clark County, NV	100	Ammonia-N+TP, 0.6 mg/L & 0.2 mg/L	27 mg/L Ammonia-N, 5.8 mg/L TP	$2.01	$183	--	$1.81	$0.50

Table 4-2. Upgrade costs for case studies *(continued)*

Process	Location	Design flow (MGD)	Target Concentration (annual average)	Initial Concentration (annual average)	$/gpd capacity	$/MG	O&M $/lb TN	O&M $/lb TP	O&M $/lb NH_3
Plug flow AS with denitrification filter	Johnston County, NC	7	TN+TP, 3.7 mg/L & 1 mg/L	31.2 mg/L TN, 5.8 mg/L TP	$0.58	$221	$0.48	$1.48	--

Key factors affecting the costs presented in Tables 4-1 and 4-2 are summarized below. Capital costs depend largely on the facilities available to meet the new requirements. Making a simple change in mode of operation from continuous aeration to the cyclic on-off mode of activated sludge was the lowest-cost conversion (Ridgefield, Connecticut). Conversion of the Freedom, Maryland, modified Ludzack-Ettinger (MLE) to a 4-stage Bardenpho required a mid-level expense of approximately $1.00/gpd capacity. Adding denitrification filters at Cox Creek, Maryland, was more expensive at $1.30 to $1.74/gpd capacity, but the upgrade had a small footprint. Adding fixed-film media at Broomfield, Colorado (Stephenson and Mohr 2005) was an option at a similar cost, also with a small footprint. Adding a fermenter for biological phosphorus removal at Kalispell, Montana, brought the cost above $3.00/gpd capacity, but the benefits were reliable operation and low O&M costs. Flexible design also tended to increase the capital costs. An example of this is the Marshall Street plant in Clearwater, Florida, which included having the flexibility of two modes of operation—operating as a 4-stage or a 5-stage Bardenpho—at the same facility. Another example of designed flexibility is North Cary, North Carolina, where mixers and aerators are in the same basin for better control in the phased isolation ditch (PID).

Costs for flow equalization basins were included for Fairfax, North Cary, and Kelowna, and could not readily be separated from other plant costs. Typically, equalization basins add $1.00 to $2.00 per gallon capacity in capital costs, depending on location, hydraulics, and control devices. Equalization basins give the plant operators capabilities to better manage and treat recycle flow streams, as well as to handle wet-weather flows.

4.2 Retrofit Process Cost Models

As discussed in detail in Chapter 5, readers of this document seeking to select a retrofit process technology to achieve nutrient removal requirements should first determine which processes are technically suited for the specific objectives at their facility. After candidate technologies have been identified, the readers should assess the estimated capital and O&M costs. They should use this cost information as a factor in selecting the process, along with other factors such as reliability, sustainability, and environmental considerations discussed in Chapter 5.

This section provides planning-level cost estimates for 12 retrofit treatment alternatives, including several alternatives involving the anoxic/oxic (A/O) process, also known as the Phoredox process. These alternatives were deemed the most likely retrofit options to be used at existing plants because of footprint size, ease of installation in an operating facility, ability to meet treatment objectives, and overall costs. Each alternative was evaluated at three different flow rates—1 MGD, 5 MGD, and 10 MGD. This range was deemed to cover the flow rate range of most readers of this manual. Note that the cost rankings represented in the cost curves will likely hold for larger and smaller flows. Ultimately, readers should find the process scenarios that best match their candidate technologies to obtain a cost estimate. Three alternatives not included in these estimates are membrane biological reactors, land application for nutrient removal, and *special filters* like the Blue PRO; these technologies are new and thus have no software-based model for estimating costs. A reader seeking cost estimates for these processes should contact the process vendors.

4.2.1 CAPDETWorks

The cost estimates for the scenarios were developed using CAPDETWorks software, version 2.1. The U.S. Environmental Protection Agency (EPA) and U.S. Army Corps of Engineers originally developed this software as a planning tool; Hydromantis Corporation now maintains and updates (2006). The software works as follows. The user generates a process layout involving a number of unit operations. The user can also define input variables, including wastewater flow rate, wastewater influent quality, and desired effluent quality or other performance coefficients. Alternatively, the user can choose to use default values developed by Hydromantis. The software then calculates the required sizes of the unit operations and uses cost-curve models from the software's database to estimate the capital, labor, chemical, and energy costs that would be incurred.

The cost functions included in software version 2.1 (the most recent available during document preparation) were updated in 2000. The model uses several standard indices to update costs to current dollars: the *Engineering News-Record* (ENR) Construction Cost Index, the Marshall & Swift Index, and the Pipe Index. Values were obtained from a U.S. Department of Agriculture Web site (USDA 2007) that transcribes historical values of these indices. The values used for the indices are shown in Table 4-3, along with other cost factors used in all CAPDETWorks runs.

A number of authors have used the CAPDET model to provide planning-level costs. Wright et al. (1988) found that the program provides construction cost estimates that are within plus or minus 20 percent of the actual costs. Overall, they showed that feasible alternatives can be compared using estimations from CAPDETWorks because the software includes all the items needed for process assessment except land and building costs. In general, the rankings of alternatives by cost will not change significantly for plants of different sizes.

To further verify the validity of CAPDETWorks, examples were run with the model to match what was installed at two of the case study locations discussed in Chapter 3, the Marshall Street facility in Clearwater, Florida, and the Kalispell Advanced Wastewater Treatment facility in Kalispell, Montana. The model was used to cost the biological treatment systems, tertiary filters, and any fermenters/digesters used to provide volatile acids for phosphorus treatment. Primary treatment, preliminary treatment, and sludge handling were not included in the costing. Because CAPDETWorks does not include a fermenter unit operation, two thickeners were used as an approximation.

Table 4-3. Cost factors used in CAPDETWorks estimates

Indices used to update costs	Value	Date
ENR	7,940	May 2007
Marshall & Swift	1,360	2007
Pipe Index	732	2007
Cost factors	**Value**	**Unit**
Electricity	$0.08	kW-hr
Labor	$25.00	Person-hour
Alum	$0.41	lb as Al2(SO4)3-14H_2O

The design and actual influent flow rate, influent constituents, and other basic plant parameters were inputs for the software, along with the known plant layouts. The model was then run to obtain estimates of capital cost for installing the plant, as well as O&M estimates for power consumption, chemical usage (if any), and additional sludge generation (if any). Those were the only components of O&M included in the case studies. In the event that current operating flow was well below the design flow, as was the case for Clearwater, a second run was done with the current flow to obtain operating costs.

Life-cycle costs were calculated for the CAPDETWorks results by first annualizing the capital cost at 20 years at 6 percent interest. The annualized capital cost was then added to the annual O&M cost to obtain a total annual cost. This cost was then divided by the annual flow to get the life-cycle cost per MG treated. The model results were then compared with the results presented in the case studies developed for this manual. All capital costs for both the software estimates and the actual capital costs were updated to 2007 dollars.

For electrical costs, the total electricity used in the unit operations cited was used rather than the breakdowns for phosphorus and nitrogen used in the case studies; this is because the CAPDETWorks software provides total electricity, including that attributable to biochemical oxygen demand (BOD) removal. Chemical costs in such cases were both nil.

Table 4-4 presents the results of the modeling versus case study results comparison for both Clearwater and Kalispell.

For Clearwater, the estimated capital cost from CAPDETWorks was close to the actual capital cost, within 17 percent. The O&M costs are similarly close, within 6 percent.

For Kalispell, the estimated capital cost is within 2 percent of the actual cost. The O&M costs are 29 percent off. The difference here might be that CAPDETWorks designed a smaller anoxic zone relative to the rest of the system than Kalispell actually uses; that would account for higher blower usage and, thus, the higher electrical costs.

Overall, these results indicate that CAPDETWorks would provide reasonably accurate cost estimates for comparative purposes during planning.

Table 4-4. Comparison of CAPDETWorks and actual costs for Clearwater, Florida, and Kalispell, Montana

Cost	Clearwater: 5-stage Bardenpho			Kalispell: University of Cape Town		
	CAPDETWorks	Actual	% Difference	CAPDETWorks	Actual	% Difference
Total capital	$35,541,000	$29,500,000	17	$8,903,000	$9,100,000	2
O&M electrical	$640,000	$610,000	5	$159,000	$113,000	29
O&M chemical	$0	$74,000	--	$0	$0	--
O&M sludge	$0	$0	--	$0	$0	--
Total O&M	$640,000	$684,000	6	$159,000	$113,000	29

Certain input assumptions were common to all model runs, and they are presented in Table 4-5. Beyond these values, various model components had default parameters, which are shown as the scenarios are discussed below. All capital cost results include a percentage of total unit costs to account for engineering, site preparation, electrical and control installation, and building costs. O&M results include operations labor, maintenance labor (both costed at $25/hour), maintenance materials, chemicals (alum for selected phosphorus removal processes, methanol for selected nitrogen removal processes), and energy. This is in contrast to the Maryland study, the Connecticut study, and the case studies, in which labor and maintenance material costs were excluded. This means that the CAPDETWorks O&M estimates will be higher than those for similar systems in those studies.

4.2.2 Retrofit Phosphorus Removal Technologies

Five retrofit plans for implementing phosphorus removal were evaluated. All five involved modifying or making additions around an existing biological treatment system that was not accomplishing phosphorus removal.

1. Alum addition at one point upstream of an existing conventional activated-sludge process, 0.5 parts per million (ppm) TP target, no filter
2. Alum addition at two points, both upstream and downstream of an existing conventional activated-sludge process, 0.1 ppm TP target, sand filter downstream

Table 4-5. Cost model influent wastewater parameters

Parameter	Value
Average flow (MGD)	1, 5, 10
TSS (mg/L)	220
% volatile solids	75%
BOD (mg/L)	220
Soluble BOD (mg/L)	80
COD (mg/L)	500
Soluble COD (mg/L)	300
TKN (mg/L)	40
Soluble TKN (mg/L)	25
Ammonia nitrogen (mg/L)	22
TP (mg/L)	5
pH (standard units)	7.6
Nitrite/nitrate (mg/L)	0.0
Temperature, summer	23 ºC
Temperature, winter	10 ºC

Notes:
BOD = biochemical oxygen demand
COD = chemical oxygen demand
TKN = total Kjeldahl nitrogen
TP = total phosphorus
TSS = total suspended solids

3. Refitting an existing conventional activated-sludge system to an A/O (Phoredox) or University of Cape Town (UCT) process, with a fermenter added to generate volatile fatty acids to support biological phosphorus uptake, with a target of 0.5 mg/L. The A/O CAPDET model was used to represent all such processes.
4. Refitting an existing conventional activated-sludge system to an A/O configuration, with a fermenter added to generate volatile fatty acids and a sand filter added to aid removal of phosphorus-containing solids, with a target of 0.5 mg/L

5. Refitting an existing conventional activated-sludge system to an A/O configuration, with a fermenter and sand filter added, as well as alum addition, with a target of 0.1 mg/L

In these calculations, the following assumptions were made with respect to planning cost estimates:

1. The cost to convert an existing activated-sludge process to A/O would incur 25 percent of the cost of a new A/O unit with the same capacity; this allows for the addition of walls and baffles to set off anaerobic zones.
2. As a fermenter module has yet to be established for CAPDETWorks, the capital and operating costs of a gravity thickener, adjusted upward by 50 percent, were used instead. A thickener operated anaerobically is a good approximation of a fermenter; the 50 percent allowance includes the pipes and mixers that would be required for a fermenter.
3. Post-secondary chemical treatment was assumed to require an alum dose four times the dose that would be suggested by stoichiometry; for example, if the stoichiometric dose was 0.5 mg/L, a dose of 2 mg/L of alum as Al was used.
4. A sand filter was assumed for tertiary filtration used in retrofits 4 and 5.
5. It was assumed that no additional aeration capacity would be needed for retrofitting an existing activated-sludge reactor to A/O.

Table 4-6 presents the alum doses used in the costing for retrofits 1, 2, and 5.

Figures 4-1 through 4-3 are graphs of the O&M costs (in dollars per MG treated), capital costs (in dollars per gpd capacity), and life-cycle costs (in dollars per MG treated) for these scenarios. The costs were estimated at 1 MGD, 5 MGD, and 10 MGD average influent flow. The life-cycle costs were determined by summing the O&M dollars per MG treated and the capital costs annualized to 20 years at 6 percent per MG treated.

From these figures, the following observations can be made:

1. The overall lowest cost option for O&M, as well as for life-cycle, is installation of a fermenter with a retrofit of the existing process to A/O. The lowest capital cost alternative is one-point chemical addition, but the retrofit/fermenter alternative is the second-lowest capital alternative. The estimated O&M for A/O plus fermentation is $5 to $25 per MG treated, while the capital ranges between $0.19 and $0.32 per gpd capacity.
2. The lowest operating costs are associated with not using chemical, i.e., the alternatives of A/O with a fermenter and A/O with a fermenter and filter. These

alternatives are two of the three lowest-life-cycle-cost alternatives, with the low capital cost for one-point chemical addition also resulting in a low life-cycle cost. The highest operating cost was associated with doing two chemical additions because both have elevated doses. These high operating costs result in that alternative's having the highest life-cycle costs as well.

Table 4-6. Process parameters for phosphorus removal scenarios

Process	Costed items	Alum dose (mg/L as Al)
1	Alum feed	2.4
2	Alum feed (2 points) with sand filter	2.6; 2.4
3	A/O conversion, fermenter	N/A
4	A/O conversion, fermenter, Sand filter	N/A
5	A/O conversion, fermenter, sand filter, alum feed (post-secondary treatment)	1.3

Notes:
A/O = anoxic/oxic
N/A = not available

3. If a TP level of 0.1 mg/L or lower in the effluent is required, use of post-secondary chemical treatment with a filter might be needed. In that instance, these results indicate that using biological phosphorus removal with a fermenter to get the concentration down to between 0.5 and 1 mg/L can save resources because doing so reduces the required post-secondary chemical dose.
4. The cost for using alum in the second feed point was conservative, because regular biological uptake was not excluded in the dosage estimation. This means that actual costs for post-secondary chemical phosphorus removal will be lower.
5. A tertiary clarifier might be applicable to aid the tertiary filtration process, but it was not included in these retrofit cost curves.

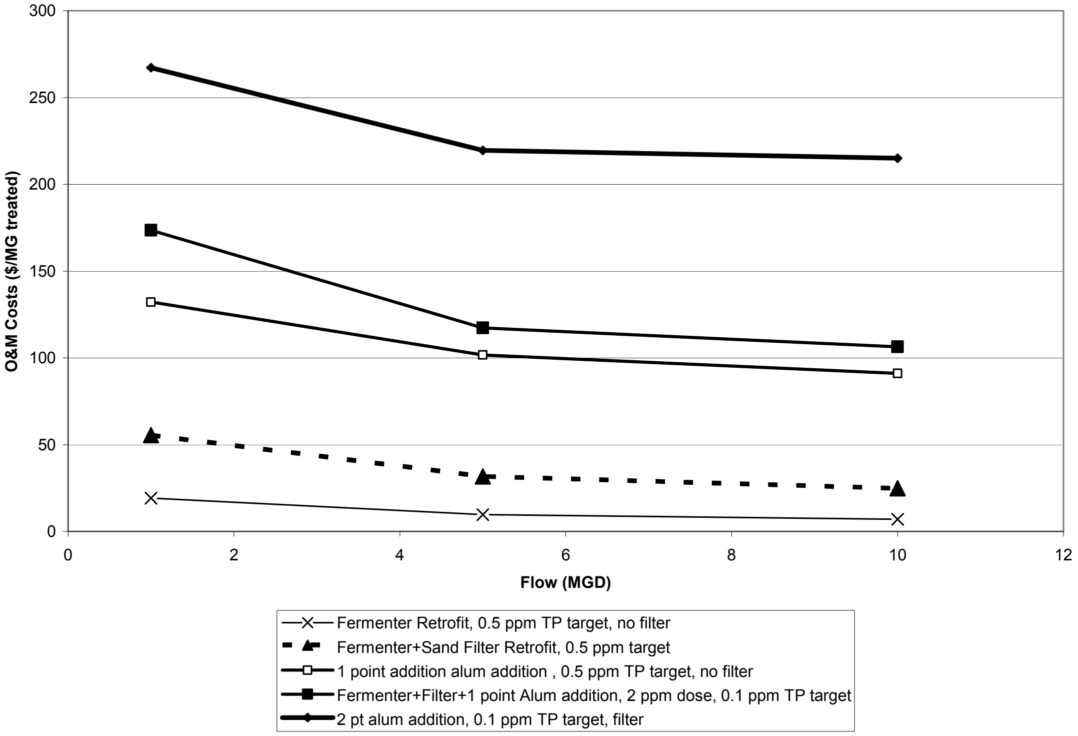

Figure 4-1. O&M costs for retrofit phosphorus removal technologies ($/MG treated).

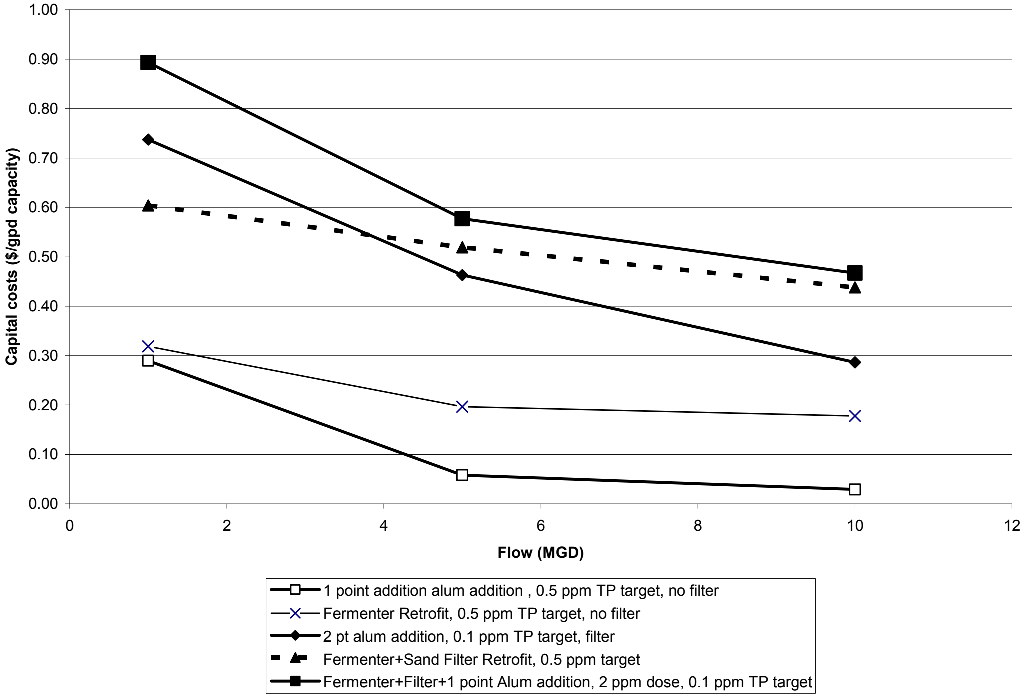

Figure 4-2. Capital costs for retrofit phosphorus removal technologies ($ per gpd capacity).

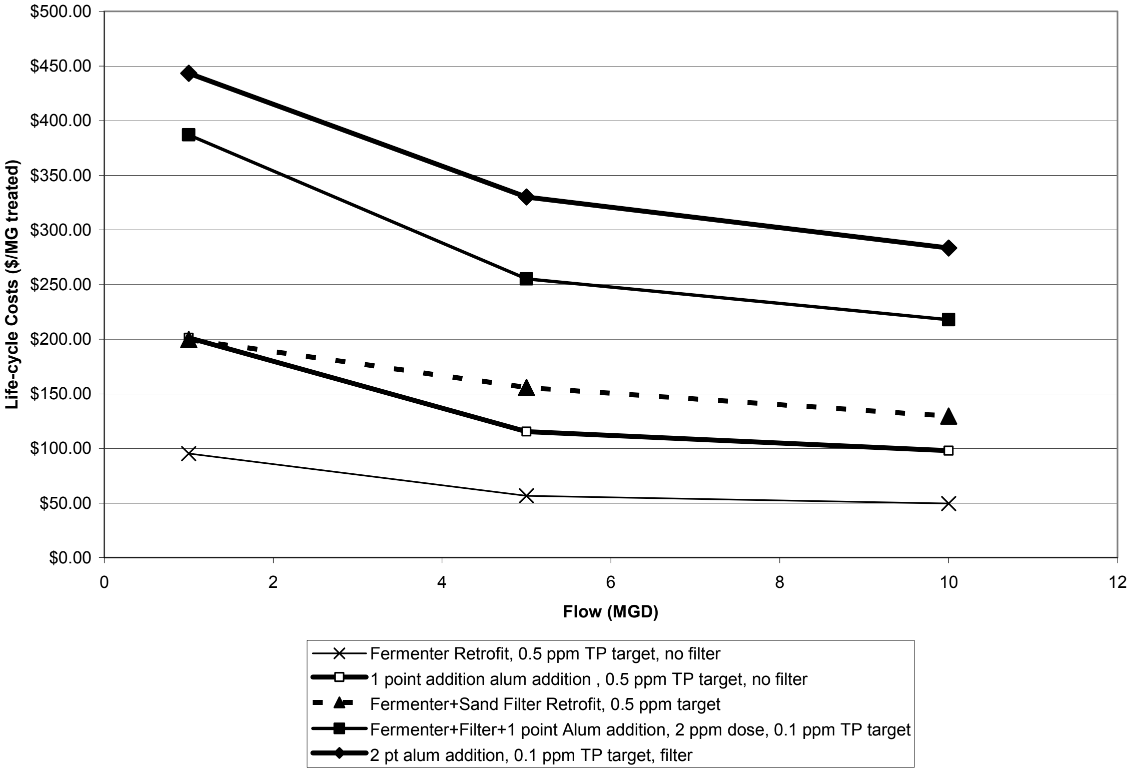

Figure 4-3. Life-cycle costs for retrofit phosphorus removal technologies ($/MG treated).

4.2.3 Retrofit Nitrogen Removal Technologies

Four retrofit plans for implementing nitrogen removal were evaluated. All four involved modifying or making additions around an existing biological treatment system that was not accomplishing phosphorus removal.

1. Installation of additional tank capacity for an oxidation ditch to allow sufficient residence time in an anoxic zone for denitrification to occur
2. Retrofitting an existing activated-sludge system as a modified Lutzak-Ettinger (MLE), with anoxic and aerobic zones and internal recirculation
3. Retrofitting an existing activated-sludge system to be a step-feed system to provide sufficient biochemical oxygen demand (BOD) to the anoxic zones to allow denitrification to occur
4. Installation of a denitrifying filter, along with a methanol system

In these calculations, the following assumptions were made with respect to planning cost estimates:

1. The cost to convert an existing activated-sludge process to MLE or step-feed would incur 33 percent of the cost of a new such unit with the same capacity; this allows for the addition of walls and baffles to set off anoxic zones, along with additional piping and recirculation pumping. The same ratio was applied to install additional secondary clarification and blower capacity.
2. Methanol was assumed to be needed only for the denitrifying filter, at a dose of 3 mg methanol per mg nitrate nitrogen.
3. The target TN concentration was assumed to be 3 mg/L.

Table 4-7 presents the items costed for these plans, as well as the methanol dosage if applicable.

Table 4-7. Process parameters for nitrogen removal scenarios

Process	Costed items	Methanol dose (lb/lb NO_3)
1	Tanks, mixers	N/A
2	MLE conversion, filter	N/A
3	Step-feed AS conversion, filter	N/A
4	Denitrifying filter	3

Notes:
AS = activated sludge
MLE = modified Ludzack-Ettinger
N/A = not available

Figures 4-4 through 4-6 are graphs of the capital costs (in dollars per gpd capacity), O&M costs (in dollars per MG treated), and life-cycle costs (in dollars per MG treated) for these nitrogen treatment plants. The costs were estimated at 1 MGD, 5 MGD, and 10 MGD average influent flow. The life-cycle costs were determined by summing the O&M dollars per MG treated and the capital costs annualized to 20 years at 6 percent per MG treated.

From these figures, the following observations can be made:

1. The lowest-cost option is the installation of additional tank capacity for an existing oxidation ditch because this had both the lowest capital and O&M costs and therefore the lowest life-cycle cost. It should be noted, however, that a great deal of land area could be required for this alternative.
2. The denitrifying filter had clearly the highest operating cost, because of the methanol requirement. The other processes use organic matter in the wastewater to support denitrification, which gives a large cost savings. The high operating cost for

denitrifying filters led to that alternative's having the highest life-cycle cost. The advantage of denitrifying filters, however, is that they have very small footprints.

3. The step-feed and MLE alternatives had similar capital, O&M, and life-cycle costs. Technically, each has advantages and disadvantages, as described elsewhere in this manual.
4. None of these alternatives employed a post-secondary filter. Such a filter could be useful in achieving low TN concentrations through removal of solids containing nitrogen. For example, an effluent with a total suspended solids (TSS) at 10 mg/L could contain 0.5 mg/L (i.e., 5 percent) TN in a filterable form. Therefore, in meeting an overall effluent limit of 3 mg/L TN, a tertiary filter could be helpful.

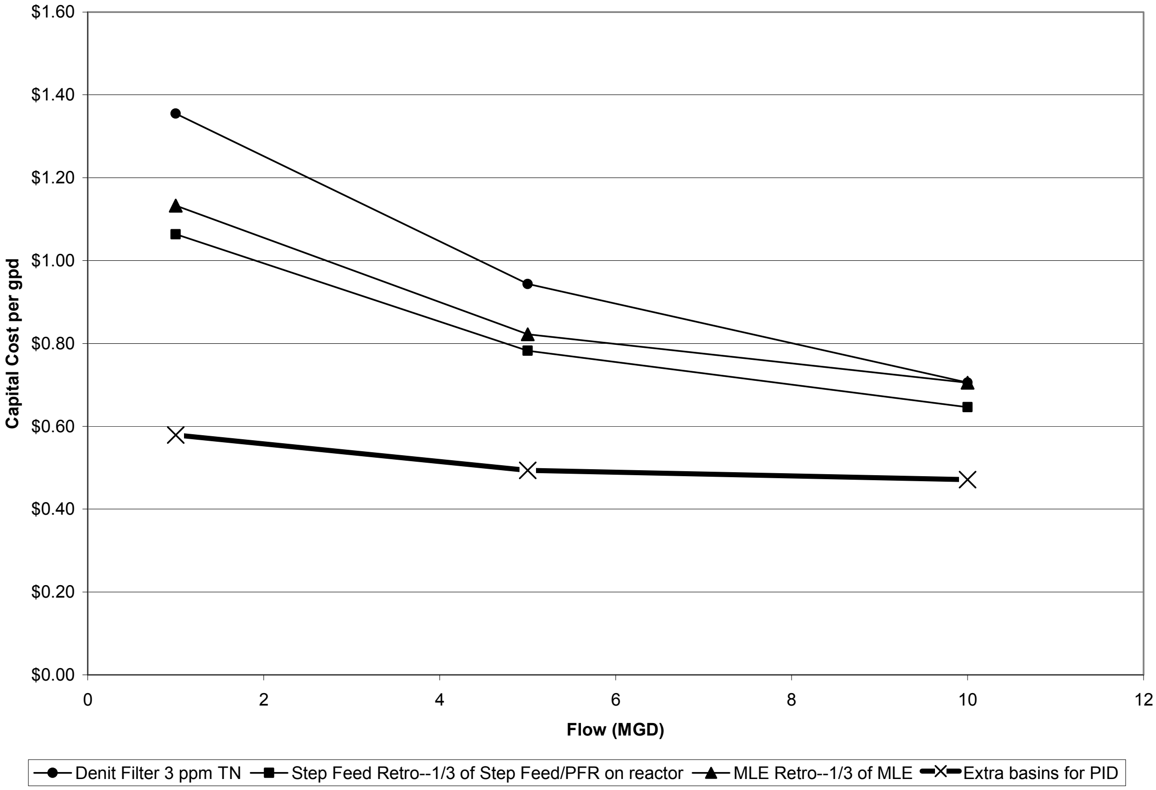

Figure 4-4. Capital costs for retrofit nitrogen removal scenarios.

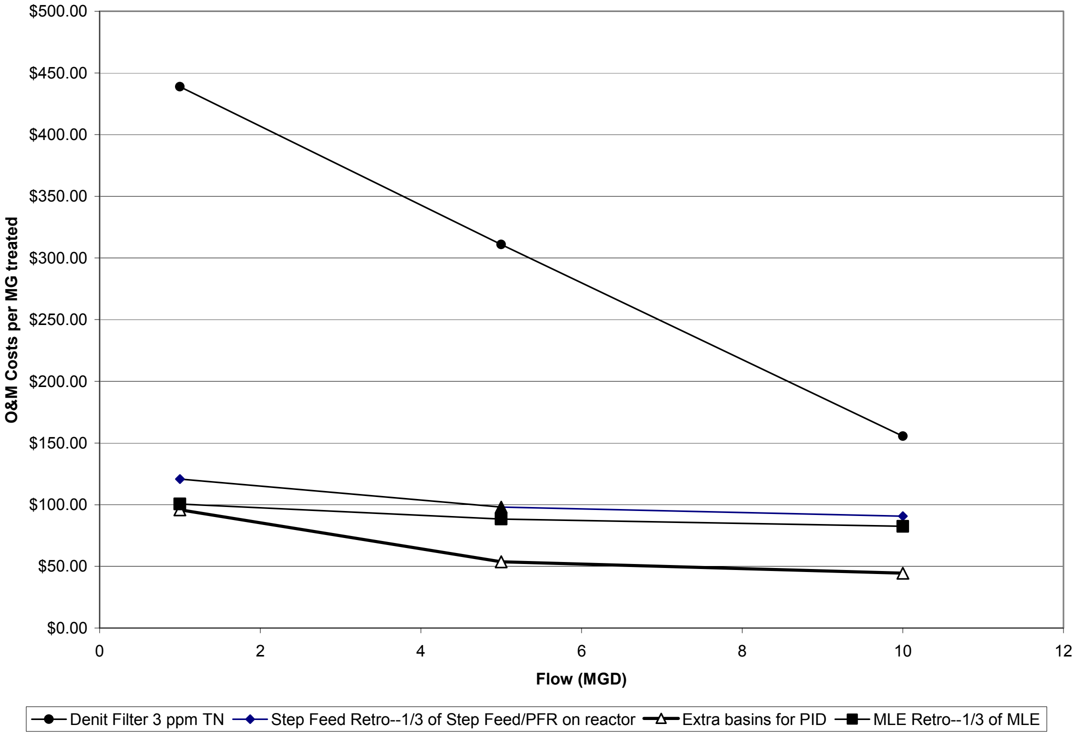

Figure 4-5. O&M costs for retrofit nitrogen removal scenarios.

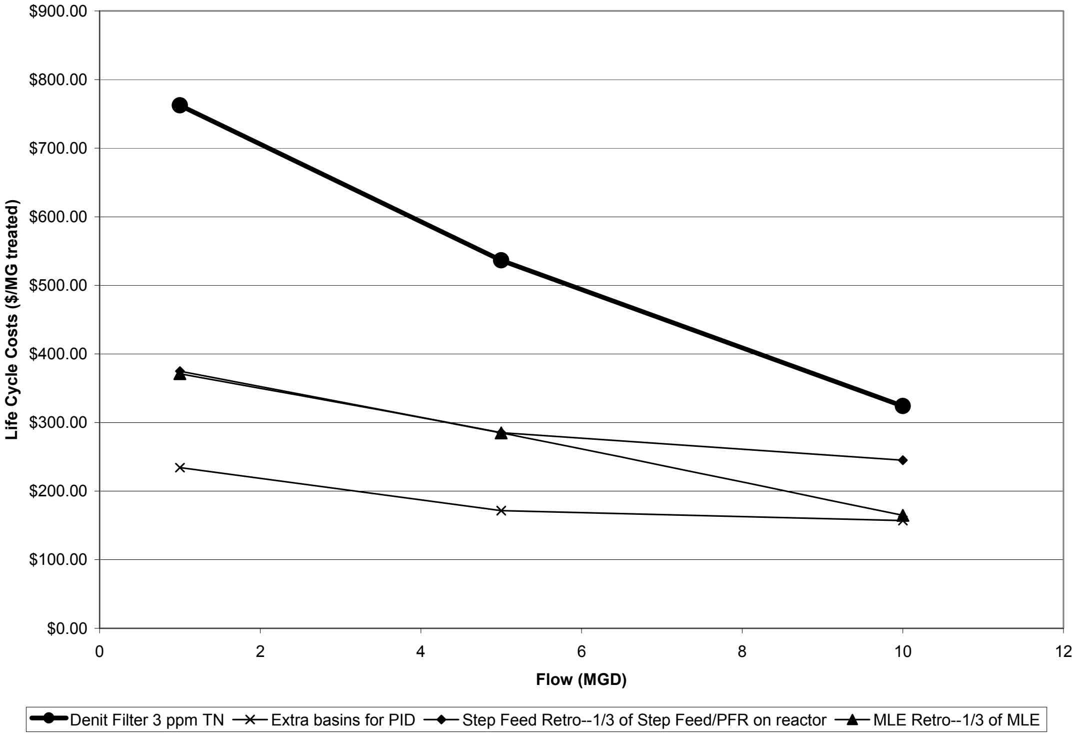

Figure 4-6. Life-cycle costs for retrofit nitrogen removal scenarios.

4.2.4 Combined Nitrogen and Phosphorus Scenarios

The following three retrofit alternatives for achieving combined nitrogen and phosphorus removal to very low levels (3 ppm TN, 0.1 ppm TP) were evaluated. All the technologies used a combination of biological nutrient removal to achieve nitrification/denitrification and some phosphorus removal, plus chemical phosphorus removal to polish the effluent to 0.1 ppm TP. All scenarios included a tertiary filter (or denitrification filter).

1. Oxidation ditch retrofitted with additional tanks (as was done above for nitrogen removal), with one-point alum addition for phosphorus removal, plus a tertiary clarifier and a tertiary sand filter.
2. A nitrifying activated-sludge reactor retrofitted with two-point alum addition for phosphorus removal plus a denitrification filter.
3. Conversion of an activated-sludge system to a 5-stage Bardenpho with chemical addition for polishing phosphorus and a tertiary filter

In these calculations, the following assumptions were made with respect to planning cost estimates:

1. The cost to convert an existing activated-sludge process to a 5-stage Bardenpho would incur 50 percent of the cost of a new such unit with the same capacity. This allows for the addition of walls and baffles to set off anoxic zones, installation of additional tank capacity, and additional piping and recirculation pumping. The same ratio was applied to install additional secondary clarification and blower capacity, as well as material and energy costs.
2. Methanol was assumed to be needed only for the denitrifying filter, at a dose of 3 mg methanol per mg nitrate-nitrogen.
3. The target TN concentration was assumed to be 3 mg/L; the target TP concentration, 0.1 mg/L.

Table 4-8 presents the items costed for these plans, as well as the methanol dosage if applicable.

Table 4-8. Process parameters for nitrogen removal scenarios

Process	Costed items	Methanol dose (lb/lb NO_3)
1	Basins, mixers, alum addition, tertiary clarifier, tertiary filter	N/A
2	Alum addition, denitrifying filter	3 mg/mg NO_3-N
3	5-stage Bardenpho conversion, alum addition, tertiary filter	N/A

Note: N/A = not available

Figures 4-7 through 4-9 are graphs of the capital costs (in dollars per gpd capacity), O&M costs (in dollars per MG treated), and life-cycle costs (in dollars per MG treated) for these alternatives. The costs were estimated at 1 MGD, 5 MGD, and 10 MGD average influent flow. The life-cycle costs were determined by summing the O&M dollars per MG treated and the capital costs annualized to 20 years at 6 percent per MG treated.

From these figures, the following observation can be made:

1. The lowest-cost alternative on a life-cycle basis is the oxidation ditch modifications. This is so because it has the best combination of reduced operating costs (because of accomplishing biological nutrient removal) and capital costs. The lowest-capital-cost option was the use of chemical phosphorus removal and a denitrifying filter, but that alternative has the highest operating costs; the lowest operating costs were attached to the ditch additions.

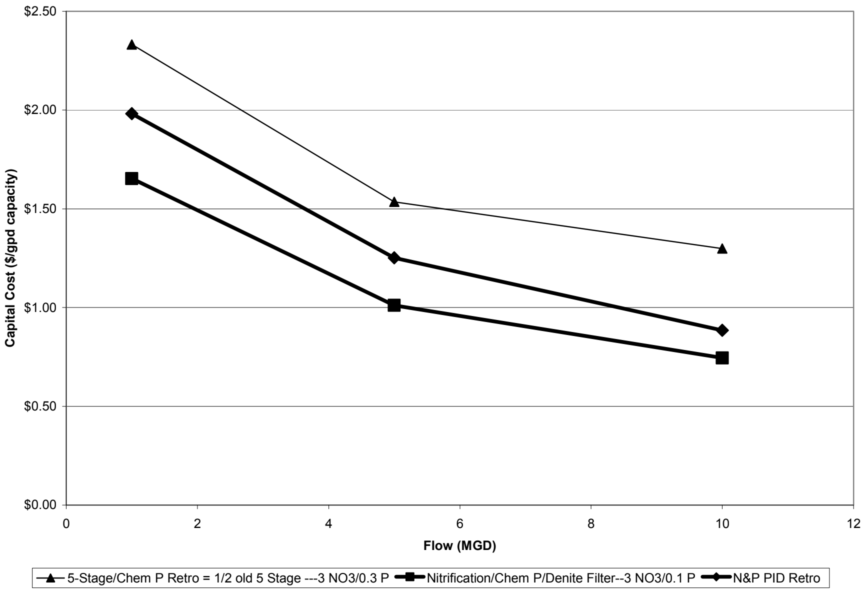

Figure 4-7. Capital costs for retrofit nitrogen plus phosphorus removal technologies.

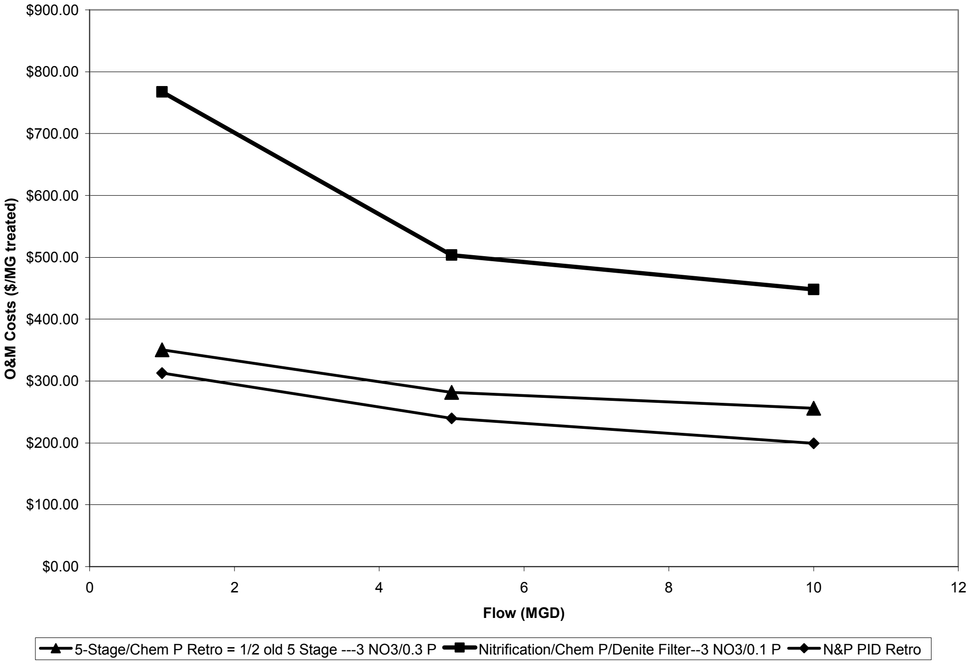

Figure 4-8. O&M costs for retrofit nitrogen plus phosphorus removal technologies.

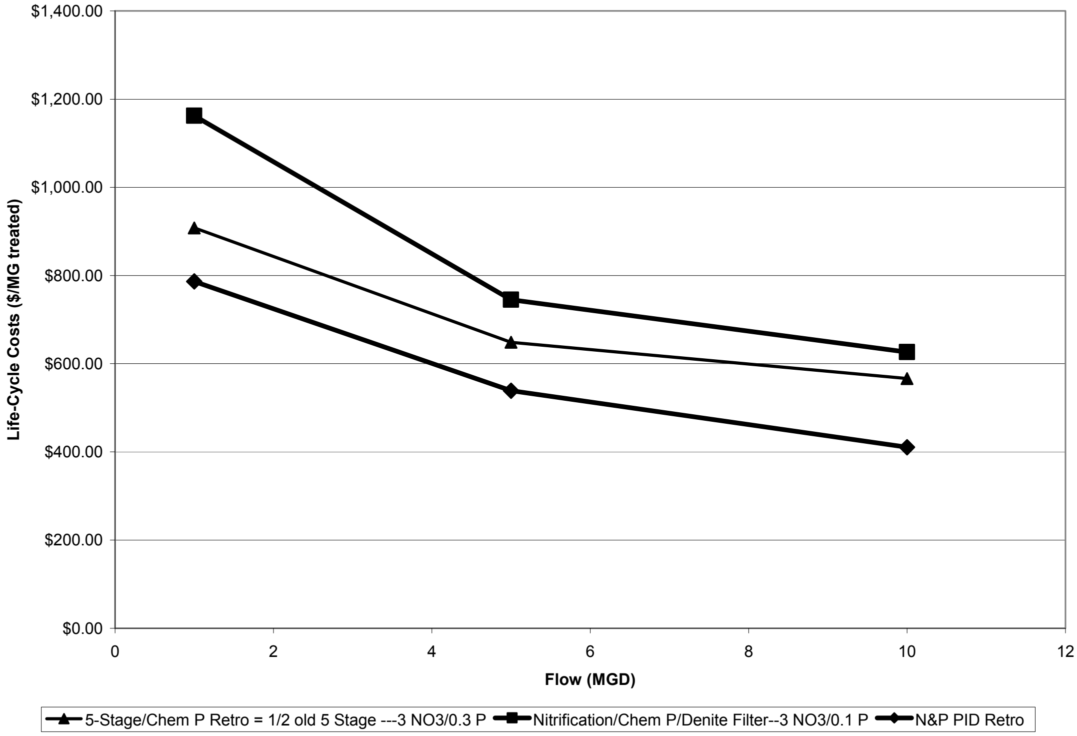

Figure 4-9. Life-cycle costs for retrofit nitrogen plus phosphorus removal technologies.

4.3 Expansion Process Cost Models

There are situations where a retrofit might not be adequate and the owners of the wastewater treatment operation must consider a plant expansion. As discussed in detail in Chapter 5, project teams seeking to select a new process technology to meet nutrient removal requirements should first determine which processes are technically suited for the specific objectives at their facility. Once candidate technologies have been identified, the project team members should assess the estimated capital and O&M costs. They should use this cost information as a factor in making the process selection, along with other factors such as reliability, sustainability, and environmental considerations.

This section provides planning-level cost estimates for 20 treatment scenarios. The scenarios were selected because they are most representative of the likely treatment process candidates to be considered, on the basis of what has been observed from the case studies and other evaluations conducted for this document. As was done for the retrofit cases above, each scenario was evaluated using CAPDETWorks at three different flow rates—1 MGD, 5 MGD, and 10 MGD. This range was deemed to cover the flow rate range of most readers of this manual. All basic process assumptions used in the retrofit section also apply for these expansion cases.

4.3.1 Phosphorus Removal Technologies

Eight scenarios for implementing phosphorus removal were evaluated:

1. Alum addition at one point upstream of an existing conventional activated-sludge process, 0.5 ppm TP target, no filter
2. Alum addition at two points, both upstream and downstream of an existing conventional activated-sludge process, 0.1 ppm TP target, sand filter downstream
3. A/O process with biological treatment to 1 ppm TP, no extra equipment
4. A/O process with biological treatment to 0.5 ppm TP, fermenter included to supply volatile fatty acids
5. A/O process with biological treatment to 0.5 ppm TP, fermenter and sand filter included
6. A/O process with biological treatment to 0.1 ppm TP, fermenter, chemical addition downstream of bioreactor, and sand filter included
7. Modified UCT (3-stage) process with biological treatment to 0.5 ppm TP, fermenter and sand filter included
8. Five-stage Bardenpho process with biological treatment to 0.5 ppm TP, sand filter included

Table 4-9 presents process parameters for these scenarios, including the hydraulic residence time (HRT) in the bioreactor, if applicable, and the chemical dosage of alum.

Figures 4-10 through 4-12 are graphs of the O&M costs (in dollars per MG treated), capital costs (in dollars per gpd capacity), and life-cycle costs (in dollars per MG treated) for these scenarios. The costs were estimated at 1 MGD, 5 MGD, and 10 MGD average influent flow. The life-cycle costs were determined by summing the O&M dollars per MG treated and the capital costs annualized to 20 years at 6 percent per MG treated. Figures 4-13 and 4-14 present the percentage breakdown of the total O&M costs for the eight technologies for 1 MGD and 10 MGD, respectively.

Table 4-9. Process parameters for phosphorus removal scenarios

Process	Costed items	HRT (hr)	Alum dose (mg/L as Al)
1	Alum feed	N/A	2.4
2	Alum feed (2 points)	N/A	2.6; 2.4
3	A/O, clarifier, blower	12.6	N/A
4	A/O, clarifier, fermenter, Blower	13.1	N/A
5	A/O, clarifier, fermenter, filter, blower	13.1	N/A
6	A/O, clarifier, fermenter, filter, blower, alum feed	13.1	2.4
7	UCT, clarifier, filter, fermenter, blower	12.0	N/A
8	5-stage Bardenpho, clarifier, filter, blower	15.1	N/A

Notes:
A/O = anoxic/oxic
N/A = not available
UCT = University of Cape Town process

From these figures, the following observations can be made:

1. The lowest-cost options are to implement one-point and two-point alum addition for an existing biological treatment system. This can be done if no additional nitrogen removal is required.

2. The A/O costs are fairly close together, with higher overall costs incurred as additional units are added, as well as when chemical addition is included. These costs include installation of a new A/O biological treatment system (reactor plus clarifier). If an existing reactor/clarifier can be modified to an A/O system by installing walls in the basin, adding an anaerobic tank, and so forth, the capital costs for these alternatives would be substantially reduced.

3. The costs for including post-secondary chemical phosphorus removal are conservative, because regular biological phosphorus uptake was not accounted for. This means the required dose would be lower, thereby reducing chemical costs.

4. The 3-stage modified UCT process and 5-stage Bardenpho process have the highest overall costs because of process complexity, as well as conservative inclusion of a number of recycle pumps (4 Q for internal recirculation and 3 Q for return-activated sludge and other back-up design elements). These processes should be considered when substantial nitrogen removal is also required.

5. As shown by processes 5 and 6, all biological removal processes, whether new or existing, can be enhanced for phosphorus removal by installing a sand filter, alum addition, or both. As discussed elsewhere in this document, including a sand filter or alum addition might have overall treatment benefits beyond nutrient removal, including enhanced wastewater reuse potential, TSS control, and reduction of trihalomethane formation.

6. The UCT process costs can be assumed to be representative of other 3-stage processes, such as the Virginia Initiative process (VIP), anaerobic/anoxic/oxic process (A^2O), and Westbank bioreactor because all involve three stages with varying amounts of mixing and aeration in the different stages.

7. Figures 4-13 and 4-14 indicate that the fraction of the total cost attached to labor expenses decreases with larger facilities; associated increases in the fraction of the cost are attached to energy and other components. In particular, the 1 MGD labor costs range from 40 to 50 percent of the total except for the two simple chemical addition technologies, while the 10 MGD labor costs range from 15 to 25percent of the total.

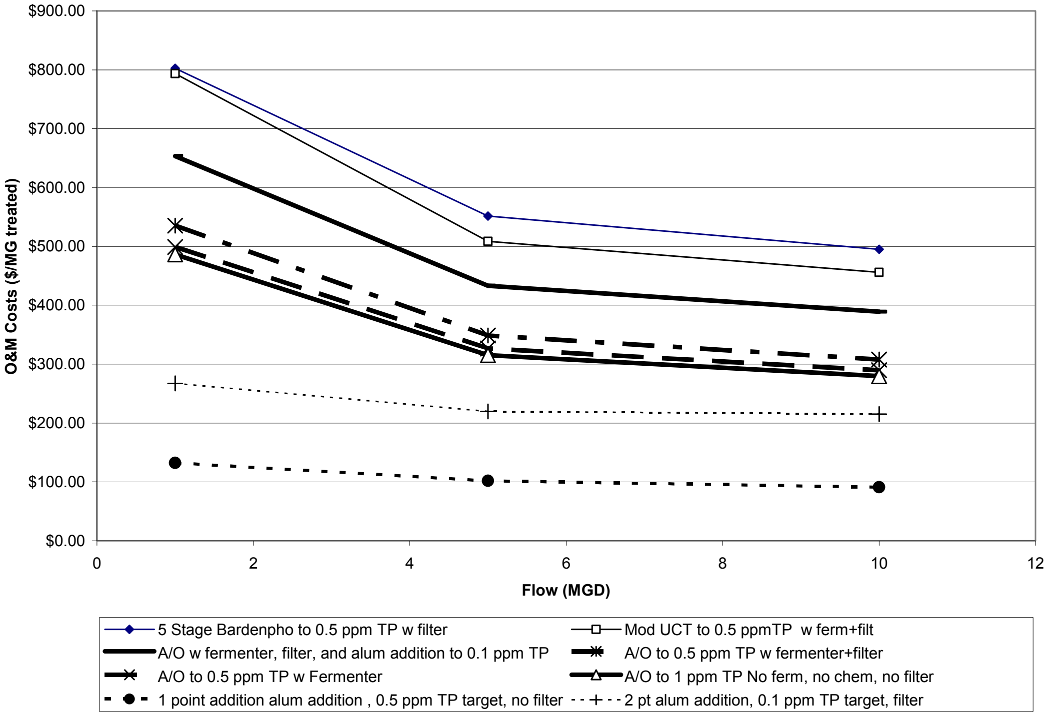

Figure 4-10. O&M costs for expansion phosphorus removal technologies ($/MG treated).

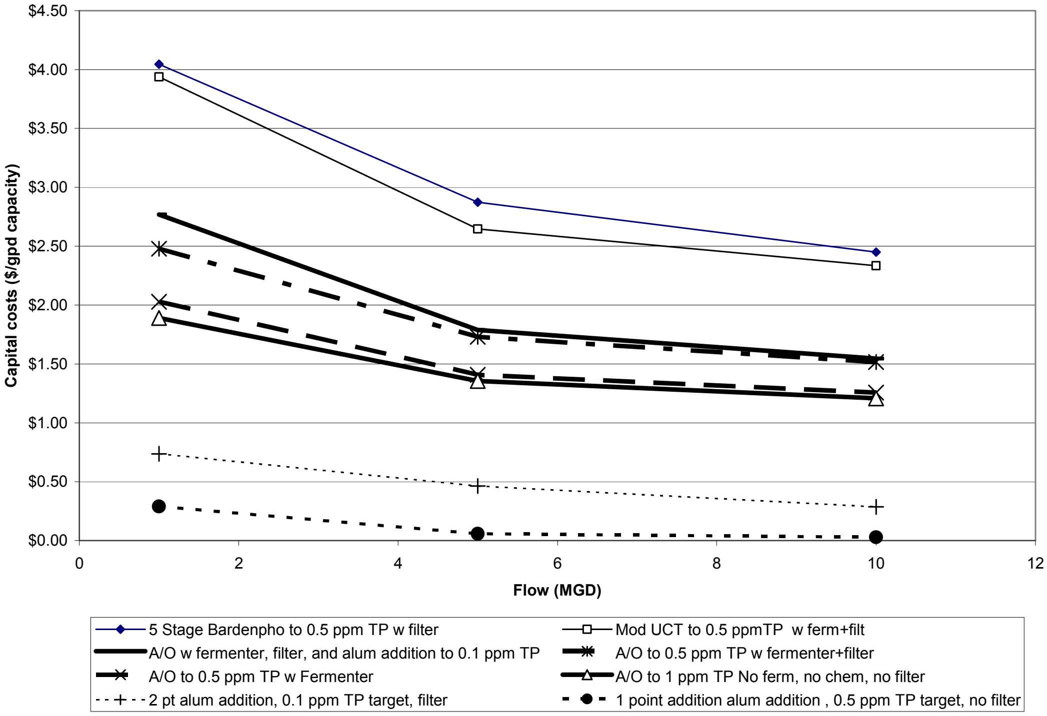

Figure 4-11. Capital costs for expansion phosphorus removal technologies.

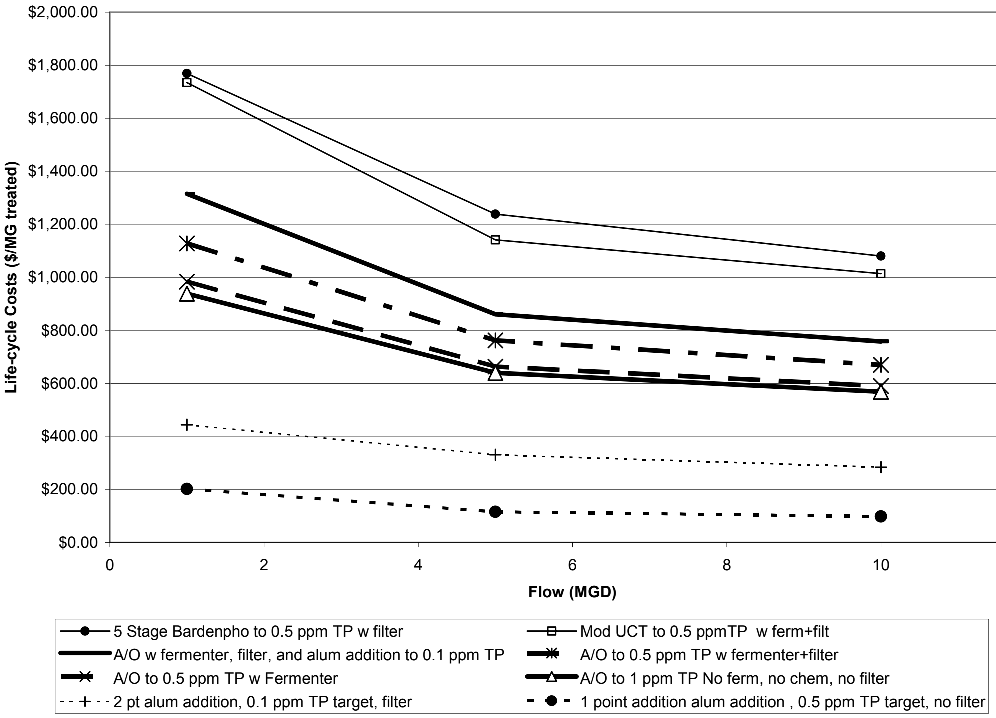

Figure 4-12. Life-cycle costs for expansion phosphorus removal technologies.

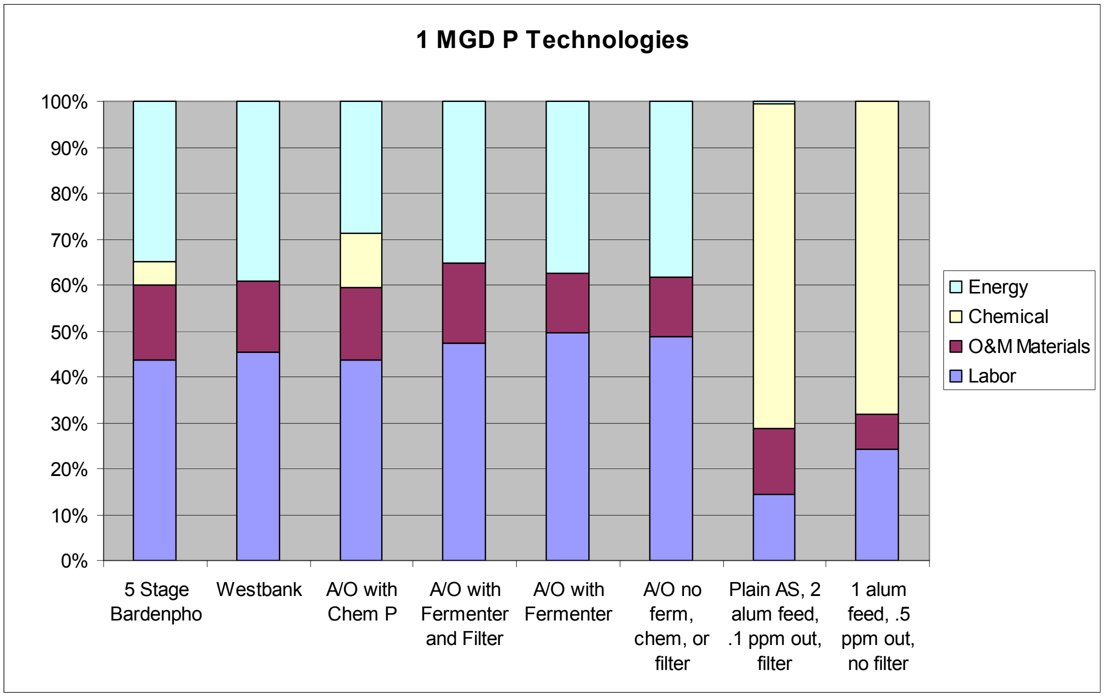

Figure 4-13. Component percentages of total O&M costs for 1 MGD expansion phosphorus removal technologies.

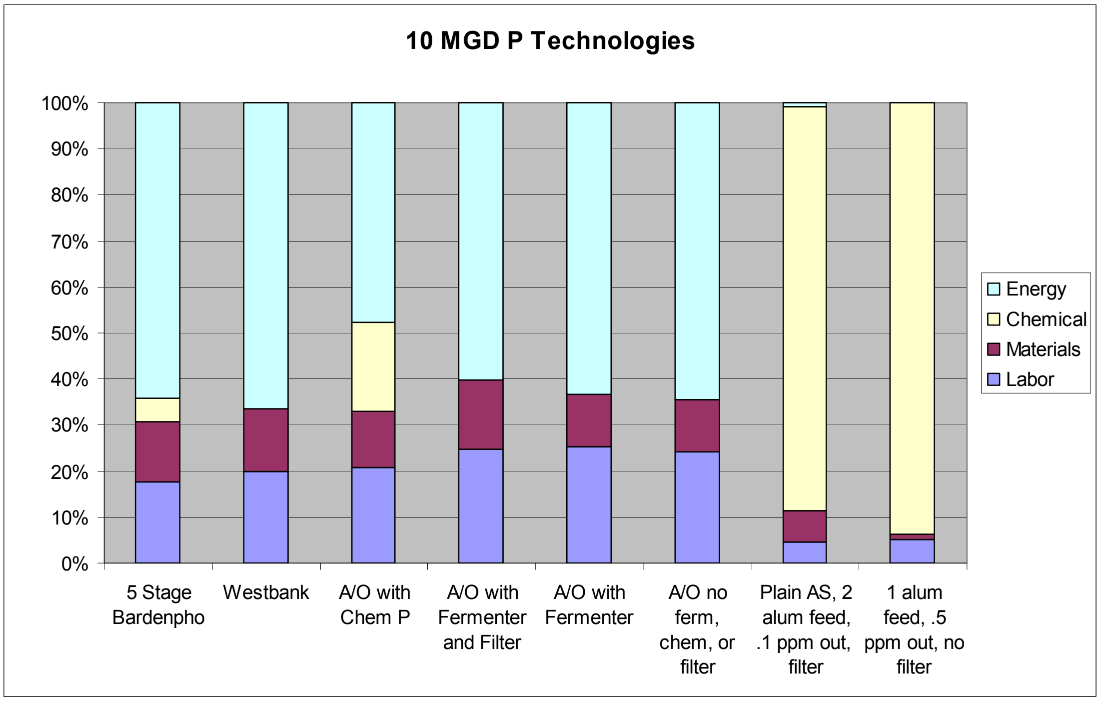

Figure 4-14. Component percentages of total O&M costs for 10 MGD expansion phosphorus removal technologies.

4.3.2 Nitrogen Removal Technologies

The following eight scenarios for implementing nitrogen removal were evaluated. All the technologies were selected specifically for TN removal, i.e., including denitrification. If a facility is required to remove only ammonia or total Kjeldahl nitrogen (TKN), all systems outlined below except the denitrifying filter will accomplish that task. In that case, the denitification system need not be operated. The project team might, however, wish to design the implementation to allow for future denitrification should the permit eventually change.

1. PID, 5 ppm TN target
2. Denitrifying filter, 3 ppm TN target
3. Step-feed, 5 ppm TN target (1 ppm TP)
4. MLE, 5 ppm TN target, without sand filter
5. Sequencing batch reactor (SBR), 5 ppm TN target
6. Three-stage UCT 5 ppm TN target (1 ppm TP)
7. Four-stage Bardenpho, 5 ppm TN target (biological phosphorus removal not supported)

8. Five-stage Bardenpho process, 5 ppm TN target with biological treatment to 0.5 ppm TP

Table 4-10 presents process assumptions for these scenarios, including the HRT in the bioreactor (if applicable) and methanol dosage (if applicable).

Figures 4-15 through 4-17 are graphs of the O&M costs (in dollars per MG treated), capital costs (in dollars per gpd capacity), and life-cycle costs (in dollars per MG treated) for these nitrogen scenarios. The costs were estimated at 1 MGD, 5 MGD, and 10 MGD average influent flow. The life-cycle costs were determined by summing the O&M dollars per MG treated and the capital costs annualized to 20 years at 6 percent per MG treated. Figures 4-18 and 4-19 present the percentage breakdown of the total O&M costs for the eight technologies for 1 MGD and 10 MGD, respectively.

Table 4-10. Process parameters for nitrogen removal scenarios

Process	Costed items	HRT (hr)	Methanol dose (lb/lb NO_3)
1	PID, clarifier, sand filter	8.8	N/A
2	Denitrification filter	N/A	3
3	Step-feed AS, clarifier, filter, blower	12.4	N/A
4	MLE reactor, clarifier, blower	16.8	N/A
5	SBR, sand filter, blower	9 (total cycle time)	N/A
6	UCT, clarifier, blower	12.1	N/A
7	4-stage Bardenpho, clarifier, blower	14.5	N/A
8	5-stage Bardenpho, clarifier, blower	15.1	N/A

Notes:
AS = activated sludge
MLE = modified Ludzack-Ettinger
N/A = not available
PID = phased isolation ditch
SBR = sequencing batch reactor
UCT = University of Cape Town process

From these figures, the following observations can be made:

1. The lowest-cost options are the oxidation ditch and the denitrification filter. The denitrification filter has the advantage of a very small footprint for inclusion within an existing system.

2. Step feed also has major advantages as a retrofit technology with an existing activated-sludge system because it can be accomplished largely by redirecting flows without needing to build substantial additional tank capacity.

3. The 3-, 4-, and 5-stage processes had the highest overall costs because of the number of tanks and process complexity. The 3-stage process costs can be assumed to be representative of all 3-stage processes—UCT, A^2O, VIP, Westbank, or others.

4. Figures 4-18 and 4-20 indicate that the fraction of the total cost attached to labor expenses decreases with larger facilities; the associated increases in the fraction of the cost are attached to energy and other components. In particular, the 1 MGD labor costs range from 40 to 50 percent of the total for all technologies except for the denitrification filter that has comparatively low other costs, so that labor is a large fraction of the total, while the 10 MGD labor costs range from 15 to 25 percent of the total.

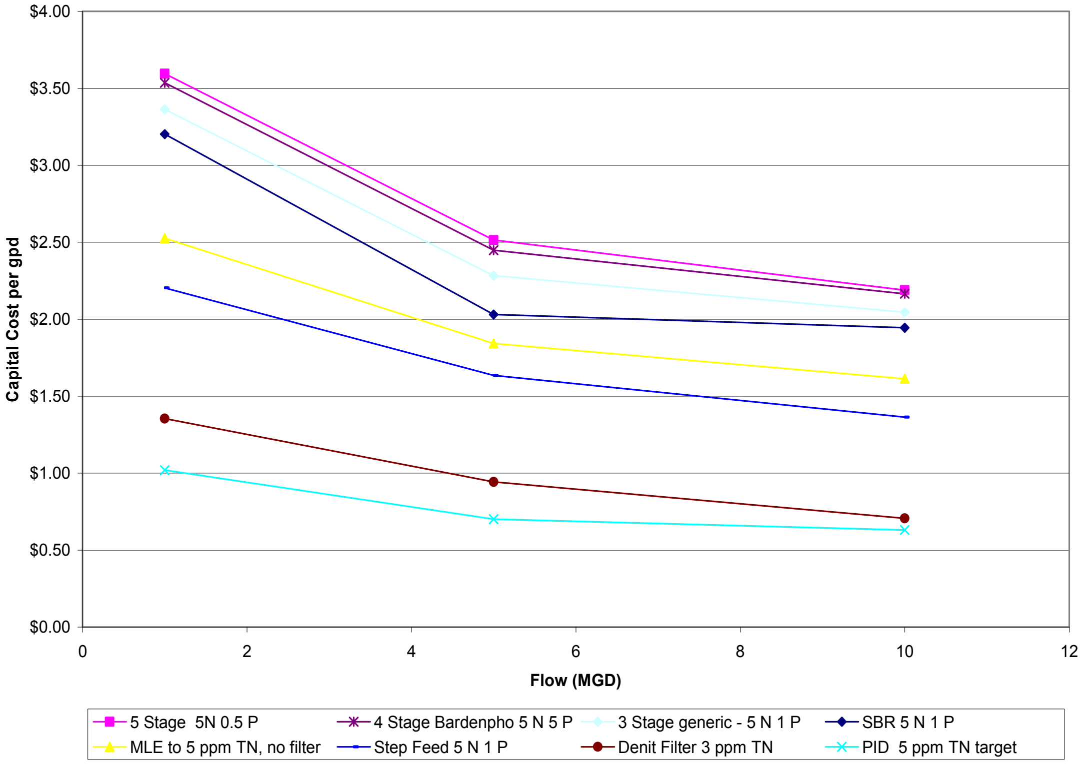

Figure 4-15. Capital costs for expansion nitrogen removal scenarios.

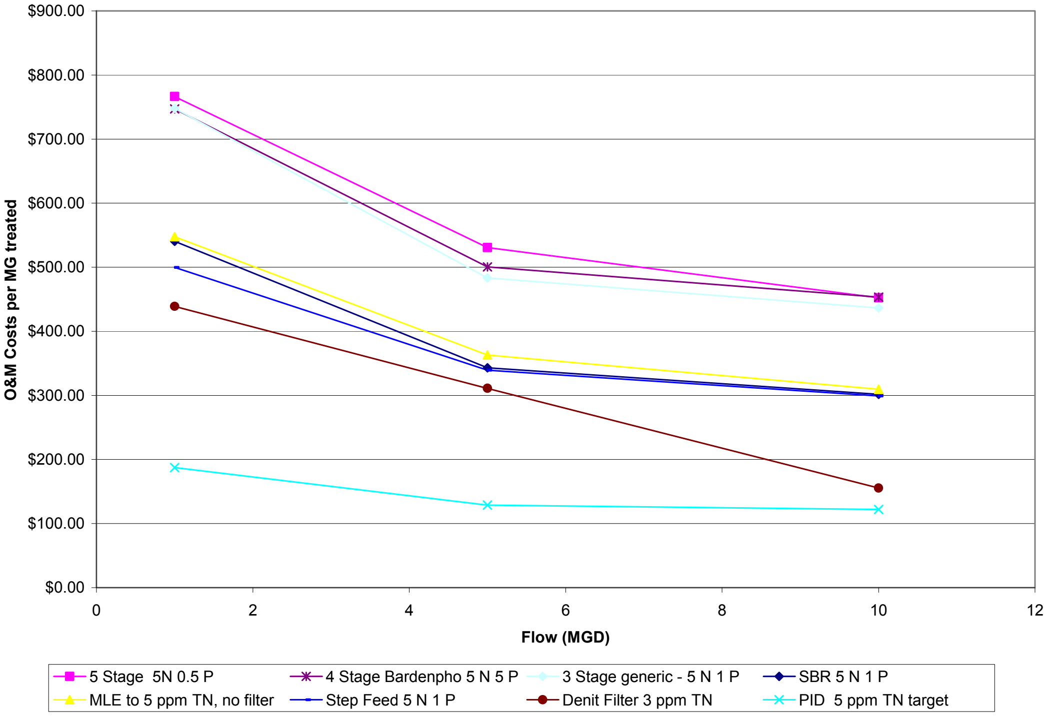

Figure 4-16. O&M costs for expansion nitrogen removal scenarios.

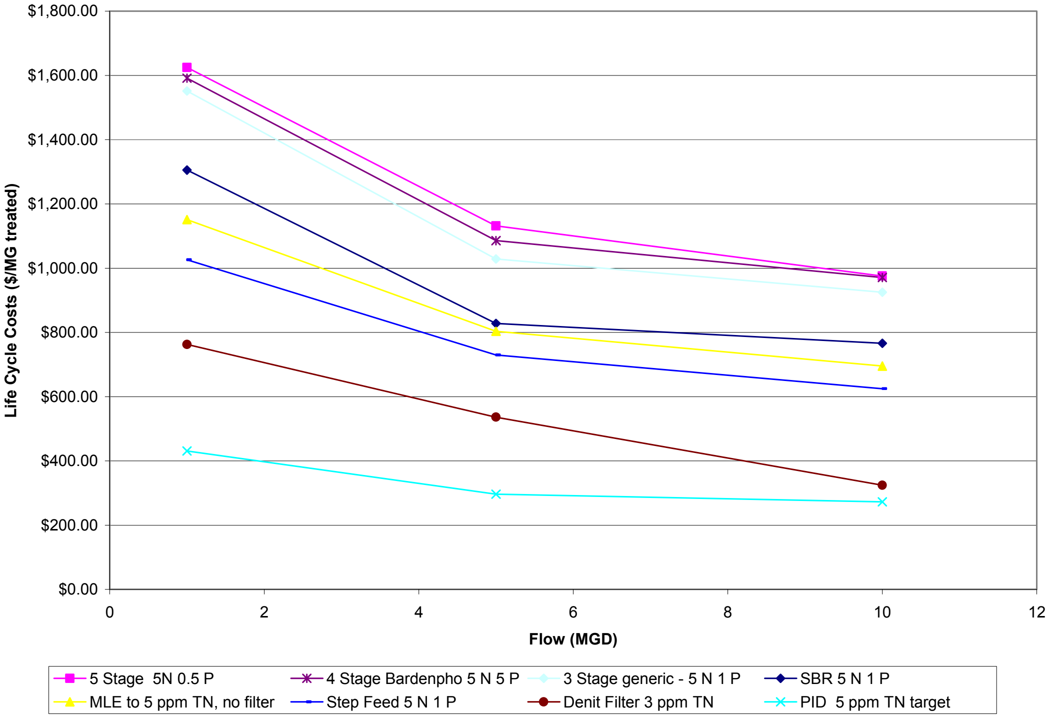

Figure 4-17. Life-cycle costs for expansion nitrogen removal scenarios.

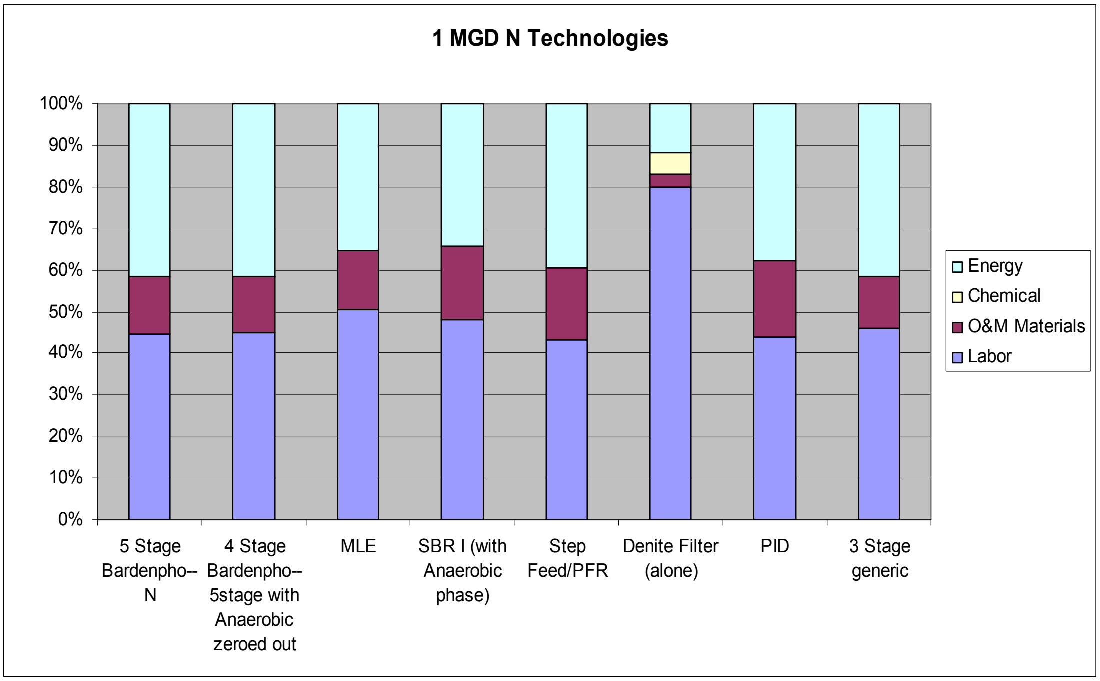

Figure 4-18. Component percentages of total O&M costs for 1 MGD expansion nitrogen removal technologies.

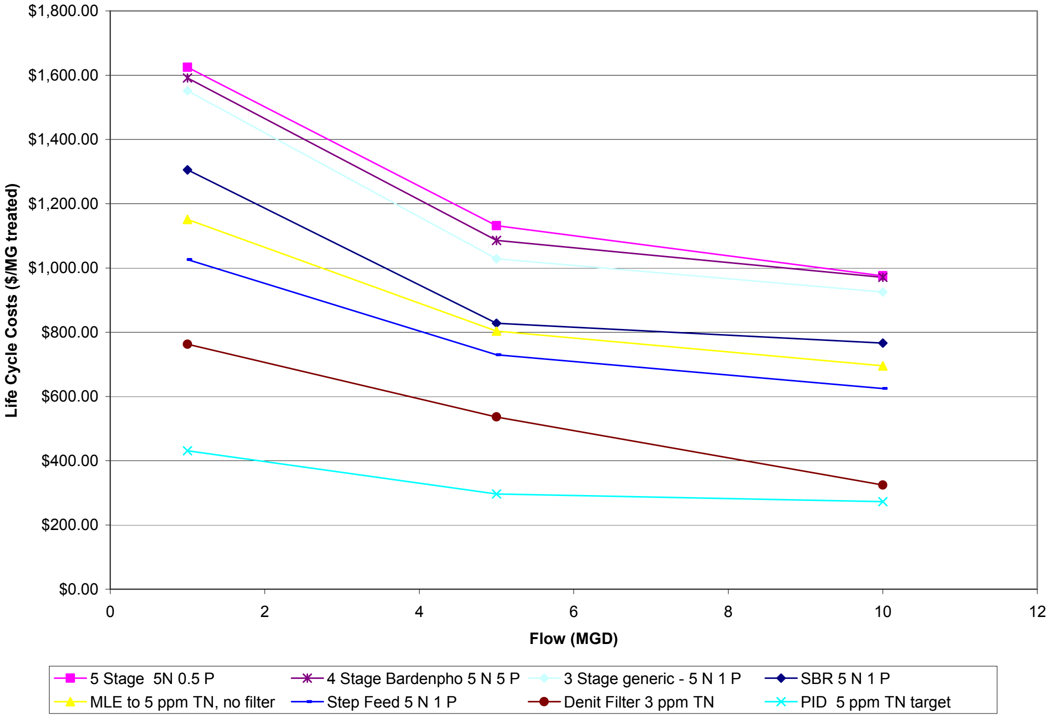

Figure 4-19. Life-cycle costs for expansion nitrogen plus phosphorus removal technologies.

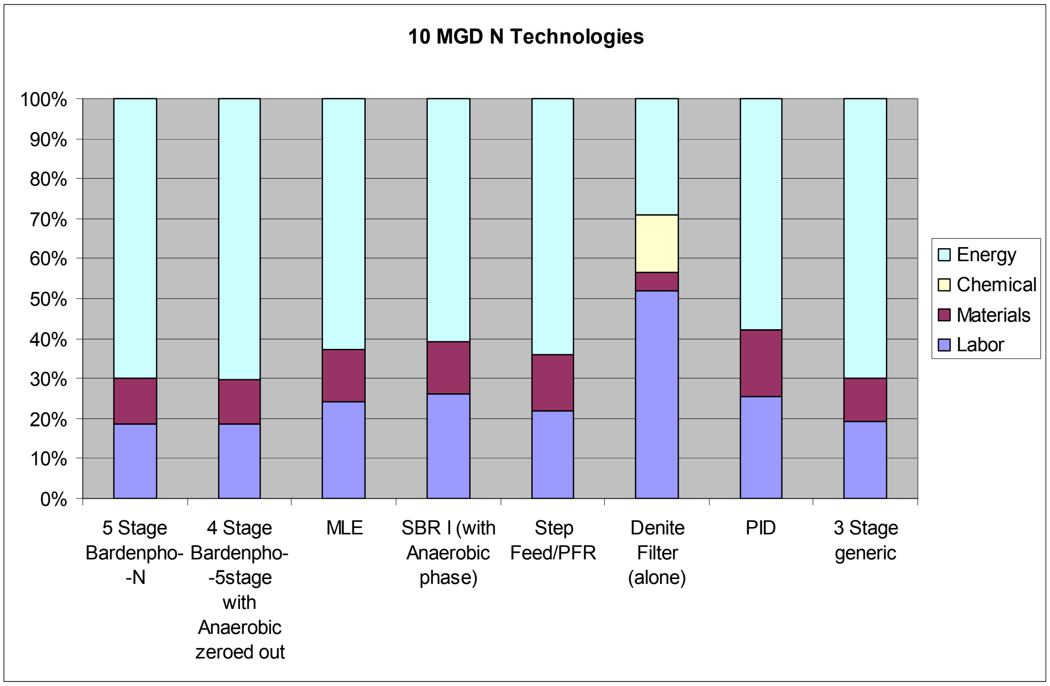

Figure 4-20. Component percentages of total O&M costs for 10 MGD expansion nitrogen removal technologies.

4.3.3 Combined Nitrogen and Phosphorus Scenarios

The following four scenarios for implementing combined nitrogen and phosphorus removal to very low levels (3 ppm TN, 0.1 ppm TP) were evaluated. All the technologies used a combination of biological nutrient removal to achieve nitrification/denitrification and some phosphorus removal, plus chemical phosphorus removal to polish the effluent to 0.1 ppm TP. All scenarios included a tertiary filter (or denitrification filter), and all except the SBR included a tertiary clarifier to help get solids (and thus TN and TP) lower.

1. PID with dual (two-point) chemical addition plus tertiary clarifier plus tertiary filter
2. SBR with chemical plus tertiary filter
3. Nitrification with chemical plus denitrification filter
4. Five-stage Bardenpho with chemical plus tertiary filter

Table 4-11 presents process assumptions for these scenarios, including the HRT in the bioreactor and alum dose for the chemical phosphorus removal.

Table 4-11. Process parameters for combined nitrogen and phosphorus removal scenarios

Process	Costed items	HRT (hr)	Alum dose (mg/L as Al)
1	PID, clarifier, dual alum additions, tertiary clarifier, sand filter, blower	13	2.0; 1.0
2	SBR, alum addition, sand filter, blower	15 (total cycle time)	2.08
3	Nitrifying AS, clarifier, dual alum addition, denitrification filter, blower	11.8	2.6, 4.8
4	5-stage Bardenpho, clarifier, alum addition, sand filter, blower	15.5	2.1

Notes:
AS = activated sludge
PID = phased isolation ditch
SBR = sequencing batch reactor

Figures 4-21 through 4-23 are graphs of the O&M costs (in dollars per MG treated), capital costs (in dollars per gpd capacity), and life-cycle costs (in dollars per MG treated) for these nitrogen scenarios. The costs were estimated at 1 MGD, 5 MGD, and 10 MGD average influent flow. The life-cycle costs were determined by summing the O&M dollar per MG treated and the capital costs annualized to 20 years at 6 percent per MG treated. Figures 4-24 and 4-25 present the percentage breakdown of the total O&M costs for the four technologies for 1 MGD and 10 MGD, respectively.

From these figures, the following observations can be made:

1. The lowest-cost option was the oxidation ditch, with the other technologies close in cost.
2. In comparing the nitrification with dual alum additions and denitrifcation filter (which is very dependent on chemicals) with the 5-stage Bardenpho, the capital costs for the Bardenpho are greater than those for the nitrification system, but the operating costs are much lower, with the result that the total costs (as reflected in the life-cycle graph) are similar.
3. Overall, if chemical phosphorus removal is to be used to attain 0.1 ppm or lower, either early chemical phosphorus removal or an efficient biological phosphorus removal system should be employed so that the dose in the final stage is reasonable.
4. As Figures 4-24 and 4-25 indicate, the fraction of the total cost attached to labor expenses decreases with larger facilities, with the associated increases in the fraction of the cost attached to energy and other components. In particular, the 1 MGD labor costs range from 40 to 50 percent of the total for all technologies, while the 10 MGD labor costs range from 15 to 25 percent of the total.

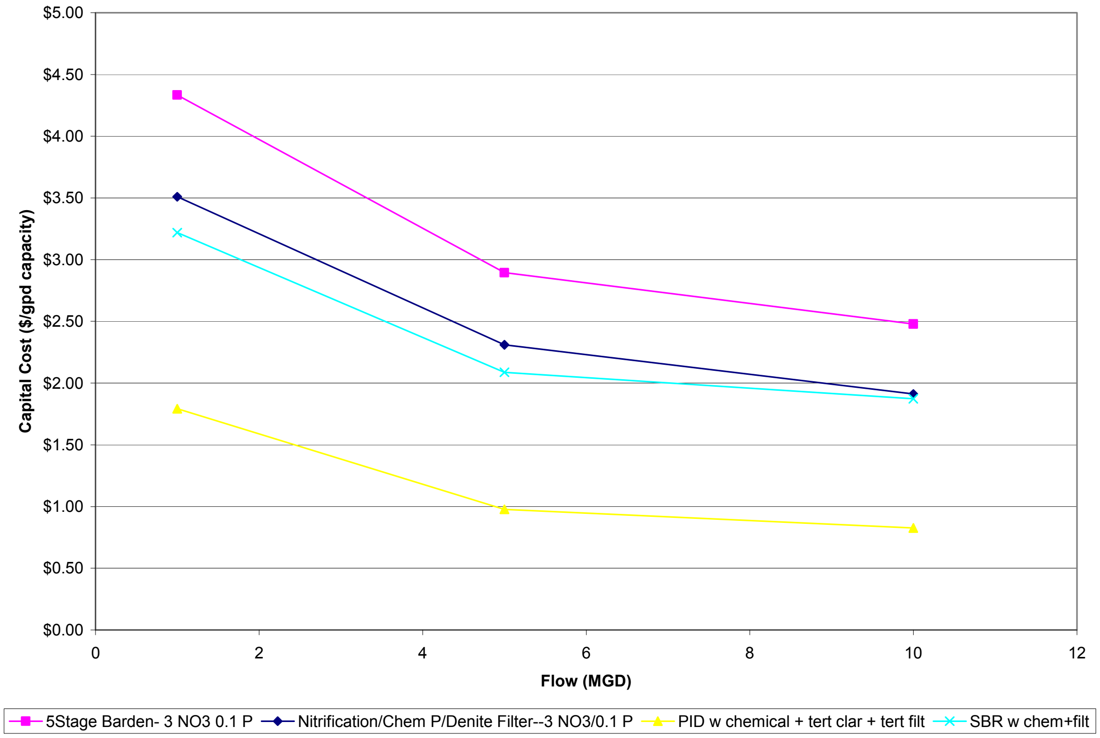

Figure 4-21. Capital costs for expansion nitrogen plus phosphorus removal technologies.

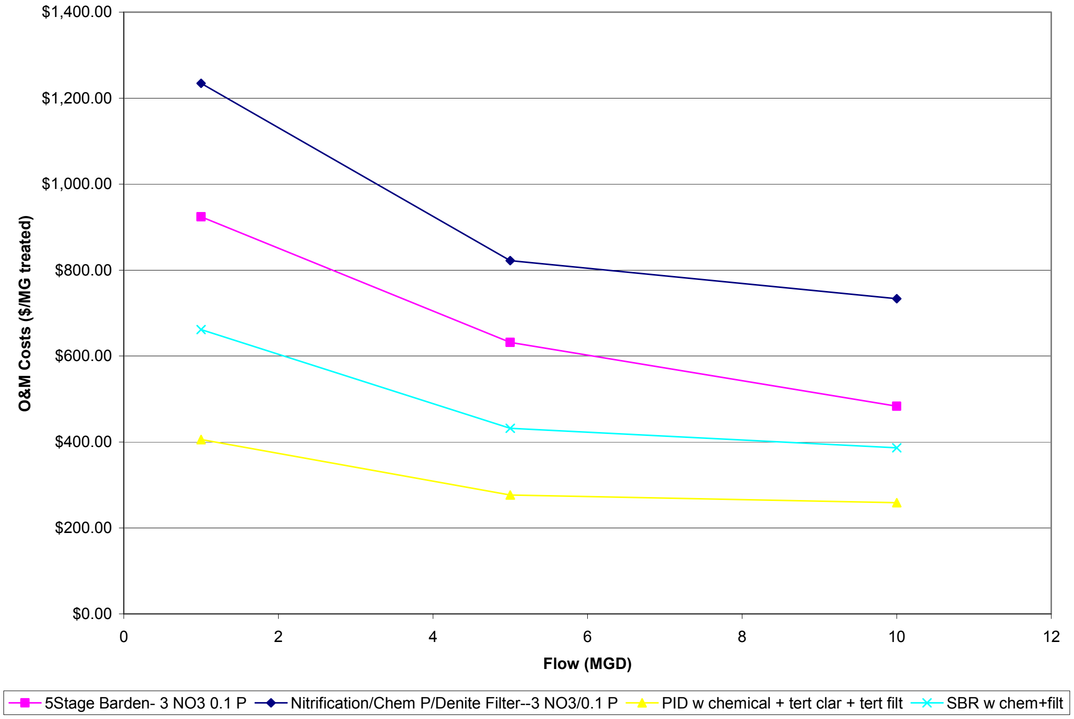

Figure 4-22. O&M costs for expansion nitrogen plus phosphorus removal technologies.

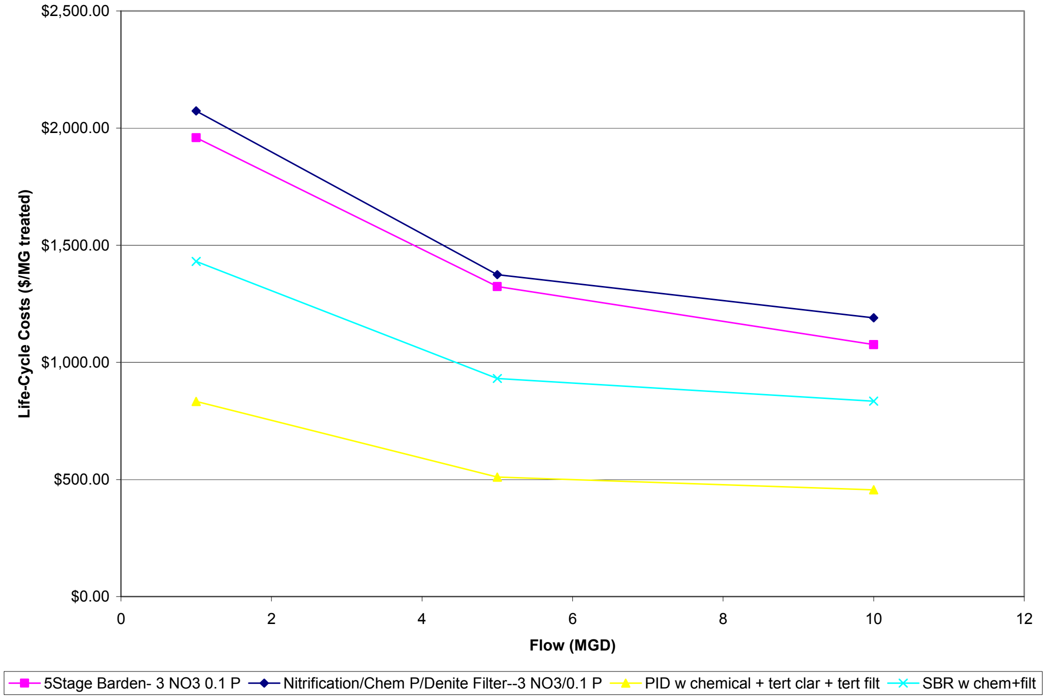

Figure 4-23. Life-cycle costs for expansion nitrogen plus phosphorus removal technologies.

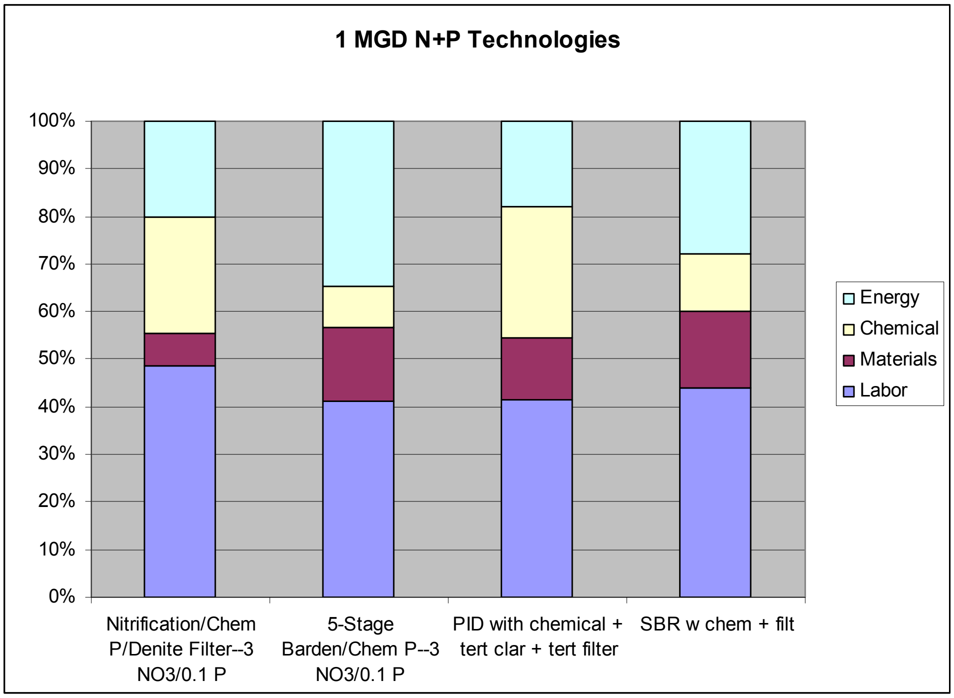

Figure 4-24. Component percentages of total O&M costs for 1 MGD expansion nitrogen plus phosphorus removal technologies.

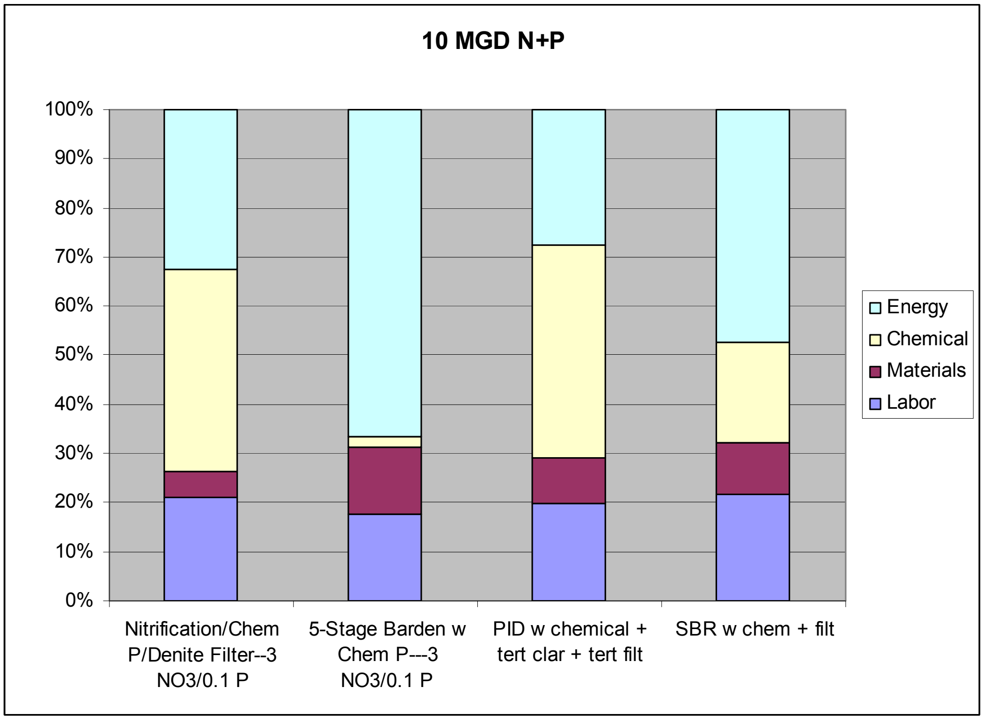

Figure 4-25. Component percentages of total O&M costs for 10 MGD expansion nitrogen plus phosphorus removal technologies.

4.4 Discussion of Cost Factors

For a project team trying to estimate costs for upgrading existing facilities, the first step is to obtain site-specific data. On the basis of the data, the project team can determine what modifications are available and their associated costs.

The cost estimates given in this chapter are for only the technologies directly involved in nutrient removal; however, an overall program to upgrade a plant to allow for nutrient removal will likely include other costs not reflected in this analysis. The ability of a plant to adequately handle nutrients will be enhanced by flow equalization, proper management of recycle flows, wise selection of sludge-handling processes, and allowance for variable conditions in the influent wastewater and the weather. To the extent that these considerations are included in a plant's process design, the overall costs could be substantially affected. However, most of these plant-wide processes will be the same for all nutrient removal technologies, so inclusion is not an issue for planning-level process selection.

In addition, site-specific sludge-handling practices affect the overall costs. Decisions about sludge handling could be made on the basis of more than the process upgrades for nutrient removal in some instances, or they could be a part of the nutrient removal project. In particular, the cost of thickening, stabilization (digestion), and dewatering might favor one alternative over another. Aerobic digestion of secondary sludge would be desired for the biological phosphorus removal alternative, whereas it would not be critical for the chemical phosphorus removal alternative. Anaerobic digestion might not be favored for nitrogen removal if another alternative exists.

O&M costs are dependent on the nutrient removal technology selected, the flexibility of the operating mode, and whether the systems for controlling power and chemical feed are automated or manual. Operating the exact number of aerobic zones at optimal dissolved oxygen control levels would be critical in power management. Chemical dosage for methanol or coagulating chemical is a key factor at such facilities. The labor cost included in this section uses the national average rate of $25 per hour. Regional labor cost variations can be expected.

As shown in Figures 4-13, 4-14, 4-18, 4-19, 4-24, and 4-25, the distribution of the O&M costs among the various components will vary depending on facility size, in addition to natural variations based on the selected technology, the characteristics of the influent wastewater, and the required level of treatment. In general, smaller facilities will have a larger fraction of the total O&M costs devoted to labor compared to that fraction for larger facilities. This is because, essentially, a 10-MGD facility will not require 10 times the staffing of a 1-MGD facility but will require 10 times the energy and chemical expenditure. Thus, the overall fraction devoted to staffing decreases, and the relative fraction devoted to other components increases. For the technologies examined in this study, the labor costs for 1-MGD facilities are generally around 40 to 50 percent of the total, while the costs for 10-MGD facilities are generally around 15 to 25 percent of the total. These are general trends, and local conditions could greatly affect the final fractions.

4.5 Summary

In upgrading existing facilities, readers are encouraged to evaluate all feasible alternatives and determine the best plan in accordance with the success criteria established early in the planning phase. Key factors in the decisionmaking include the proposed permit limits, the availability of an existing facility for modification, site requirements, costs in capital and O&M, sustainability in energy, chemical and sludge management, and potential odor or other environmental factors. For analyzing costs, the reader can examine what has happened at potentially similar facilities, as described in Section 4.1. From the case studies developed for this manual, as well as surveys carried out in Maryland and Connecticut, it was found that costs could vary greatly, depending on the nature of the existing facility, the required upgrade, and other site-specific factors. The costs for modification ranged from a low of $0.20 to a high of $5.25 per gpd capacity.

Planning-level capital, O&M, and life-cycle cost curves for 12 retrofit and 20 expansion alternatives were developed using CAPDETWorks. These curves provide an estimate of the costs; exact costs will likely be different because of local conditions. In general, unit costs are higher for smaller facilities than for larger facilities because of economies of scale. The relative fraction of the O&M costs for the expansion alternatives were also determined for 1-and 10-MGD plants. In general, labor represents 40 to 50 percent of the O&M costs for 1-MGD facilities and 15 to 25 percent of the O&M costs for 10-MGD facilities.

The cost curves were developed so that project teams can directly compare costs of selected alternatives. Project teams following the selection matrix process described in Chapter 5 can use the curves for capital, O&M, or life-cycle costs in their evaluation of alternatives.

4.6 References

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CHAPTER 5: Upgrading Existing Facilities

The topic of upgrading existing facilities has become increasingly important because many publicly owned treatment works (POTWs) in the United States are considering the addition of nutrient removal technologies. This chapter presents important issues to be considered in upgrading existing facilities, including evaluation of available technologies, general selection factors, design and operational factors, and finally selection of the appropriate technology using a decision matrix methodology.

5.1 General Approach to Upgrading

In addition to the important topics presented in this chapter for assessing appropriate nutrient removal technologies, readers are encouraged to review relevant technical references on the subject of upgrading wastewater treatment plants, such as WEF Manual of Practice No. 28, *Upgrading and Retrofitting Water and Wastewater Treatment Plants* (WEF 2004); EPA’s *Handbook: Retrofitting POTWs for Phosphorus Removal in the Chesapeake Bay Drainage Basin* (1987); EPA’s *Nitrogen Control Manual* (1993) and, specifically on nutrient removal, WEF Manual of Practice No. 30, *Biological Nutrient Removal Operation in Wastewater Treatment Plants* (WEF 2005).

5.1.1 Success Criteria

One of the key tasks of the project team evaluating a facility upgrade is to establish success criteria for the project early in the planning stage. In doing so, the team should seek input from all stakeholders in the permittee’s organization. The stakeholders might include elected officials; utility administrators; and representatives from operation, maintenance, and technical services at the facility. They might also include representatives from environmental organizations, citizens groups, and the public. The selection criteria might include sustainability, cost-effectiveness, ease of operation and maintenance (O&M), project schedule, and site requirements. *Sustainability* implies a simultaneous focus on economic, social, and environmental performance. A technology selected on the basis of sustainability has reliably lower energy use, lower production of sludge, lower use of added chemicals, and generally lower use of resources compared to other technologies. Another selection criterion to be considered is the potential to use the treated wastewater for reuse applications. For nutrient removal technologies, selection considerations include energy usage, chemical usage, and generation of biosolids. The project team can establish a rating formula by which each topic is assigned a weighting factor. A final score can thereby be obtained for each alternative evaluated.

5.1.2 Facility Planning

The project team has three main tasks to carry out:

1. Assess the existing facility and its ability to meet permit requirements.
2. Identify needed upgrades.
3. Develop and evaluate feasible alternatives.

Choices in upgrading the plant might include modifying existing facilities, adding new processes in parallel or in series, or replacing existing processes with new ones. Critical issues to be considered by the project team include agreement on design loadings for the planning period based on mass balance and energy balances (where applicable), the degree of flexibility for future uncertainties in regulation or influent wastewater characteristics (or both), operation during construction, interim permit compliance, and the phasing of the construction in the future. Related project issues include the schedule, safety, site requirements, the potential for odor, and the costs of the alternatives.

The project team typically presents a recommended implementation plan, from previously agreed-upon selection criteria, to the approval authority. The team can invite the public to participate in the evaluation.

5.2 Available Technologies

WEF Manual of Practice No. 8, *Design of Municipal Wastewater Treatment Plants* (WEF and ASCE 1988), Volume II, Chapter 15, defines an *integrated process* as a method that combines biological and chemical or physical unit operations and processes to reduce concentrations of nitrogen and phosphorus in plant effluent to below levels that would be attainable solely by synthesis in a typical secondary treatment facility. The manual, referred to as MOP 8, includes design guidelines. It also identifies inherent limitations in biological processes, which stem from variables such as influent wastewater characteristics, methods of solids handling, and the biological population dynamics of mixed cultures within a given treatment facility. For example, an insufficient supply of volatile fatty acids (VFAs) during the year can decrease denitrification and phosphorus removal.

Decreased VFAs are expected with cold wastewater temperatures and wet-weather events. Nutrients, particularly ammonia and phosphorus, can be released because of prolonged sludge storage and digestion. High nutrient concentrations can also be found in return flows from the other sludge-handling processes. Secondary release of nutrients in secondary clarifiers and tertiary filters can occur, and the nutrients are capable of overloading the biological process when returned upstream of the sensitive unit process. Upsets from slug discharges from industries can also negatively affect the treatment plant by organically or hydraulically overloading the biological process or introducing pollutants that are toxic to the microorganisms. Because of these biological limitations, the biological system often needs to be supplemented by a physical process, such as final filtration, to remove additional phosphorus and nitrogen-bearing solids; chemical addition, to precipitate additional phosphorus; or an external carbon source for denitrification. Chemical coagulants and flocculants are added to wastewater to remove phosphorus separately or concurrently with nitrogen removal.

The project team can consider technologies on the basis of target concentrations, as shown in Tables 5-1 through 5-4, which are adapted from similar tables in Chapter 2. These tables show the levels of treatment possible by using various technologies for the removal of total nitrogen (TN), total phosphorus (TP), or both. While the lists are reasonably comprehensive, a number of technologies, particularly emerging technologies, are not included in the tables. Such technologies might be available and appropriate for a given application and should therefore be considered, where applicable.

Table 5-1. Process list: TP

Concentration	Technology
Up to 1 mg/L	All
1.0 to 0.5 mg/L	Chemical precipitation A/O; filter preferred 5-stage Bardenpho PhoStrip; filter preferred SBR
0.1 to 0.5 mg/L	Chemical precipitation with filter 3-stage Westbank with filter Chemical precipitation with tertiary clarifier and filter Modified UCT with filter PID with filter 5-stage Bardenpho with chemical and filter Step-feed AS with filter
0.1 mg/L and below	Membrane filter High-performance filter: Trident Dynasand D2 Advanced Filtration System Blue PRO CoMag Land application/infiltration bed

Notes:
A/O = anoxic/oxic
AS = activated sludge
PID = phased isolation ditch
SBR = sequencing batch reactor
UCT = University of Cape Town process
“filter” = conventional filter, such as a sand filter, deep bed anthracite filter, dual-media filter, or one of the other traditional filters described in Chapter 2

Table 5-2. Process list: TN

Concentration	Technology
8 mg/L or higher	All; no filter required
3 to 8 mg/L	A^2O; filter preferred MLE SBR Cyclic activated sludge BAF IFAS MBBR 3-stage Westbank 4-stage Bardenpho Post-aeration anoxic with methanol (Blue Plains process)
3 mg/L and below	4-stage Bardenpho; filter required PID with filter Denitrification AS with filter Step-feed AS with filter Denitrification filter: Tetra filter Leopold filter Biostyr filter

Notes:
A^2O = anaerobic/anaerobic/oxic
AS = activated sludge
BAF = biological aerated filter
IFAS = integrated fixed-film activated sludge
MBBR = moving-bed biofilm reactor
MLE =modified Ludzack-Ettinger
PID = phased isolation ditch
"filter" = conventional filter, such as sand filter, deep bed anthracite filter, dual-media filter, or one of the other traditional filters described in Chapter 2

Table 5-3. Process list: ammonia nitrogen

Concentration	Technology
All	Add aeration and clarifier IFAS MBBR A/O MLE Oxidation ditch SBR 3-stage Westbank

Notes:
IFAS/MBBR = integrated fixed-film activated sludge/moving-bed biofilm reactor
A /O = anoxic/oxic
MLE = Modified Ludzack-Ettinger
SBR = sequencing batch reactor

Table 5-4. Process list: TN plus TP

TN concentration	TP concentration	Technology
3 to 8 mg/L	> 1 mg/L	SBR; filter preferred Modified UCT A/O PhoStrip II 3-stage Westbank Step-feed AS PID 5-stage Bardenpho Table 5-2 alternatives for TN 3 to 8 mg/L plus chemical phosphorus removal
3 to 8 mg/L	0.5 to 1 mg/L	SBR; filter required Modified UCT A/O PhoStrip II 3-stage Westbank Step-feed AS PID 5-stage Bardenpho Table 5-2 alternatives for TN 3 to 8 mg/L plus chemical phosphorus removal
3 mg/L or less	0.1 to 0.5 mg/L	Step-feed AS; filter required 5-stage Bardenpho; filter required PID; filter required Denitrification filter Table 5-2 alternatives for TN 3 mg/L or below plus chemical phosphorus removal and filtration
3 mg/L or less	0.1 mg/L or less	Step-feed AS + tertiary clarifier with chemical and filter Denitrification filter with chemical and additional filter Ammonia removal process Phospshorus process Upstream process MBR with chemical Land application from above three

Notes:
A/O = anoxic/oxic
AS = activated sludge
PID = phased isolation ditch
MBR = membrane bioreactor
SBR= sequencing batch reactor
UCT = University of Cape Town process
“filter” = conventional filter, such as a sand filter, deep bed anthracite filter, dual-media filter, or one of the other traditional filters described in Chapter 2

After the project team identifies the processes that are capable of meeting the desired target levels for nitrogen, phosphorus, or both, they can review the descriptions of the processes and schematics in Chapter 2. Capital and O&M costs can be found in Chapter 4. Section 5.3, Technology Selection Criteria, and Section 5.4, Design and Operational Factors in Nutrient Removal, can then be reviewed. Using all this information and the individual conditions at the plant, the project team can narrow down the list of processes to be evaluated for implementation and then conduct an evaluation as described in Section 5.5, Finalizing Process Selection, to select the process that will be used to upgrade the plant.

5.3 Technology Selection Criteria

In developing technology alternatives, the project team should consider appropriate factors for design and operation. These include site constraints; reliability of the technology; capital and O&M costs; and sustainability, including energy usage, chemical usage, and additional sludge protection. A detailed discussion on reliability and contributing factors is presented in Chapter 2 of this document, and cost considerations are provided in Chapter 4. This chapter presents process selection factors for the project team to consider.

Tables 5-5 and 5-6 contain factors that affect the selection of feasible technologies for nitrogen and phosphorus removal, respectively. Table 5-7 summarizes the selection factors for technologies that remove both TN and TP. These tables present key factors, arranged in a matrix that should be considered in selecting the best technology for a given location. The factors in the tables represent general trends. However, some of the factors might not be applicable in specific situations. For example, in warm-weather locations, a building might not be required, despite a *Yes* for that technology.

For the same discharge limit, the constraints could favor one technology over another. Some of the processes can be used in series. For example, to achieve low TN levels, a denitrification filter might be used following the 4-stage Bardenpho process. Both the 4-stage Bardenpho and filtration lines would need to be reviewed in Table 5-5 to determine the site, wastewater, and operational factors that apply to these processes. Similarly, to achieve low TP effluent concentrations, a 5-stage Bardenpho process could be used in combination with chemical precipitation and filtration. All three lines for these processes would need to be reviewed in Table 5-6 to determine the applicable selection factor values. Descriptions of the columns under each factor category are provided below.

The following three sections (5.3.1, 5.3.2, and 5.3.3) discuss selection factors to be considered for achieving nitrogen and phosphorus removal, as well as nitrogen plus phosphorus removal.

- ***Footprint.*** The *Footprint* column refers to the relative amount of space that a process requires, which depends on the number of tanks and the required size of each process. General size indications of small, medium, and large are provided in the tables.

Table 5-5. Technology selection matrix: nitrogen removal

Process	Site factors						Wastewater factors	Operation factors		
	Footprint	Building needed	Construction in existing aeration basin	Piping and pumping	Extra head needed	Secondary process recycle streams	Additional carbon source needed	Extra electricity	Chemicals needed	Add'l sludge
Denitrification filters	Small	Yes	No	Yes	Yes	No	Yes	Yes	Yes	Yes
MBBR	Medium	No	Maybe[1]	No	No	No	Maybe	No	Maybe	Maybe
SBR/cyclic activated sludge	Medium	No	Yes	No	Maybe	No	Maybe	Yes	Maybe	Maybe
MLE/3-stage Westbank	Medium	No	Maybe	Yes	No	Yes	Maybe	Yes	Maybe	Maybe
Phased isolation ditch	Large	No	Maybe	No	No	No	Maybe	Yes	Maybe	Maybe
4-stage Bardenpho	Large	No	Maybe	Yes	No	Yes	Maybe	Yes	Maybe	Maybe
A^2O	Medium	No	Maybe	Yes	No	Yes	Maybe	Yes	Maybe	Maybe
Anoxic zone following aeration(Blue Plains)	Medium	No	Maybe	No	Maybe	No	Yes	Yes	Yes	Yes
Biological aerated filtration	Medium	Maybe	No	Yes	Yes	Yes	Maybe	Maybe	Maybe	Maybe
Step-feed AS	Large	No	Yes	Yes	No	Yes	No	No	No	No
IFAS	Medium	No	Maybe[1]	No	No	Yes	Maybe	Maybe	Maybe	Maybe

Notes:

[1] Installation of media retention screens, as needed.

Additional carbon source and chemical needs: To obtain sufficient carbon for anoxic reaction, use either external source (methanol) or step-feed activated sludge.

Construction needs: External filters require extra space and could require a building depending on conditions.

Adjustments in basins include walls to section off anoxic basins and, potentially, piping to accommodate step-feed activated sludge.

Additional sludge generation is partially dependent on the need for addition of a carbon source.

Selection factor designations are general guidelines and may not apply for all site-specific conditions.

Table 5-6. Technology selection matrix: phosphorus removal

Process	Site factors						Wastewater factors	Operation factors			
	Footprint	Building needed	Construction in existing aeration basin	Piping and pumping	Extra head needed	Secondary process recycle streams	Additional carbon source needed	Extra electricity	Chemicals needed		Add’l Sludge
									C add’n	Chem P rem	
A/O	Medium	No	Yes	No	No	No	Maybe	Yes	Maybe	Maybe	No
SBR/CAS	Medium	No	Yes	No	Maybe	No	No	Yes	No	Maybe	No
VIP/modified UCT	Medium	No	Maybe	Yes	No	Yes	Maybe	Yes	Maybe	Maybe	No
Chemical precipitation	Small	Yes	No	Yes	No	No	No	Minimal	No	Yes	Yes
Tertiary clarifier/tertiary filter	Medium	Maybe	No	Maybe	Yes	Yes	No	Yes	No	Yes	Yes
PhoStrip	Small	No	No	Yes	No	Yes	No	Yes	No	Yes	Yes
5-stage Bardenpho	Large	No	Maybe	Yes	No	Yes	Maybe	Yes	Maybe	Maybe	Maybe
3-stage Westbank	Medium	No	Maybe	Yes	No	Yes	Maybe	Yes	Maybe	Maybe	Maybe
EBPR with VFA addition and filters	Medium	Maybe (Filters)	Maybe	Yes	Yes	Yes	Yes	Yes	Yes	Maybe	Maybe
MBR	Small	Yes	Yes	Yes	Yes	No	Maybe	Yes	Maybe	Yes	Yes
Phased isolation ditch	Large	No	Maybe	No	No	No	No	Yes	No	Maybe	Maybe
Membrane filter	Small	Maybe	No	Yes	Yes	Yes	No	Yes	No	Yes	Yes
CoMag	Medium	Maybe	Maybe	Yes	Yes	Yes	No	Yes	No	Yes	Yes
Infiltration basin	Large	No	No	No	Yes	No	No	Minimal	No	Yes	Yes

Notes: CAS = cyclic activated sludge; VIP = Virginia Initiative process; EBPR = enhanced biological phosphorus removal; VFA = volatile fatty acids
See “Tertiary clarifier/tertiary filter” row for information that applies to specialty filters.
Selection factor designations are general guidelines and may not apply for all site-specific conditions.

Table 5-7. Technology selection matrix: nitrogen and phosphorus removal

Process	Site factors						Wastewater factors	Operation factors			
	Footprint	Building needed	Construction in existing aeration basin	Piping and pumping	Extra head needed	Secondary process recycle streams	Additional carbon source needed	Extra electricity	Chemicals needed		Add'l sludge
									C Add'n	Chem P Rem	
Step-feed with selector	Medium	No	Yes	Yes	No	No	No	No	No	Maybe	Maybe
IFAS with selector	Medium	No	Maybe[1]	No	No	Maybe	Maybe	Maybe	Maybe	Maybe	Maybe
A^2O	Medium	No	Yes	Yes	No	Yes	Maybe	Yes	Maybe	Maybe	Maybe
SBR/CAS	Medium	No	Yes	No	Maybe	No	Maybe	Maybe	Maybe	Maybe	Maybe
VIP/modified UCT	Medium	No	Maybe	Yes	No	Yes	Maybe	Yes	Maybe	Maybe	Maybe
Phostrip II	Medium	No	No	Yes	No	Yes	No	Yes	No	Yes	Yes
3-stage Westbank	Medium	No	Maybe	Yes	No	Yes	Maybe	Yes	Maybe	Maybe	Maybe
5-stage Bardenpho	Large	No	Maybe	Yes	No	Yes	Maybe	Yes	Maybe	Maybe	Maybe
EBPR with VFA addition	Medium	No	Maybe	Yes	Maybe	No	Yes	Yes	Yes	Maybe	Maybe
MBR	Small	Yes	Yes	Yes	Yes	Yes	Maybe	Yes	Maybe	Yes	Yes
Chemical precipitation	Small	Yes	No	Yes	No	No	No	Minimal	No	Yes	Yes
Filtration	Medium	Maybe	No	Maybe	Yes	Yes	No	Yes	No	Yes	Yes
Denitrification filter	Small	Yes	No	Yes	Yes	No	Yes	Yes	Yes	Maybe	Yes
Phased isolation ditch	Large	No	Maybe	No	No	No	No	Yes	No	Maybe	Maybe

Notes:
[1] Installation of media retention screens, as needed.
Additional carbon source: To obtain sufficient carbon for anoxic reaction, use external source (methanol) or step-feed activated sludge.
Construction needs: Adjustments in basins include walls to section off anoxic basins and, potentially, piping to accommodate step-feed activated sludge.
Selection factor designations are general guidelines and may not apply for all site-specific conditions

- ***Building Needed.*** This column indicates whether the process should be placed in a building to prevent operational problems at low temperatures. An entry of *Maybe* in the tables indicates that for plants in the north (with low winter temperatures) the equipment probably needs to be placed in a building, while for plants in more moderate climates it might not.

- ***Construction in Existing Aeration Basin.*** The existing aeration basin might require modifications (construction) to implement the process. Processes that are external to the secondary treatment process, such as filters, would not involve construction in the existing aeration basin. Several of the processes require a particular ratio of anaerobic, anoxic, and aerobic zones to function properly. Construction within the existing aeration basin would involve building walls, baffles, or both to create such zones. Replacement of the aeration devices with mixers would also be needed in the anaerobic and anoxic zones. A table entry of *Maybe* indicates that modifications might be needed, depending on the size of the existing aeration basins. For example, if there is space to create an anoxic zone within the existing aeration basin and sufficient volume remains to perform nitrification in the aerobic zone, the answer is *Yes*. Conversely, if insufficient volume remains for the aerobic zone, the anoxic zone would be constructed ahead of the existing aeration basin and minimal changes would be made, with the exception of allowing flow to enter the aeration basin from the newly constructed anoxic zone.

- ***Piping and Pumping.*** The term *piping and pumping* refers to whether the process involves return lines, requires more than one feed line (in the case of step-feed applications), or requires additional pumping equipment, or both. In the case of processes that use filters (for nitrogen or phosphorus removal), the column entry is *Yes* because the filter backwash is returned upstream of the secondary process. Piping and pumping are needed for all processes that involve new internal recycle lines. An entry of *Maybe* indicates that internal recycles might be needed, depending on the existing process. For example, an existing modified Ludzack-Ettinger (MLE) process that is being upgraded to a 4-stage Bardenpho might not need a new internal recycle, depending on the existing capacity, whereas an existing extended-aeration plant would require the installation of a new internal recycle line.

- ***Extra Head Needed.*** The hydraulics for the proposed process should always be reviewed to verify that the wastewater flows through the plant as designed. Certain processes require additional head, which likely needs to be provided by pumps, for the wastewater to enter and pass through the process, and therefore additional land is required. Processes that require additional biological tanks likely require little additional head, whereas add-on processes might require pumping.

- ***Secondary Process Recycle Streams.*** This column identifies the processes that have internal recycle lines that require additional land. To achieve low effluent nitrogen limits, internal recycle lines are usually required to denitrify the nitrates created during nitrification. Phosphorus removal technologies, unless paired with nitrogen removal processes, usually have only a return activated sludge (RAS) line.

- ***Additional Carbon Source Needed.*** Some processes require an additional carbon source to function. For example, denitrification filters require methanol or an alternative carbon source, such as MicroC, for the process to function correctly because most of the biochemical oxygen demand (BOD) has been removed from the wastewater after secondary treatment. Adequate carbon is needed for biological nutrient removal (BNR) (USEPA 1993). A table entry of *Maybe* indicates that whether additional carbon is needed depends on the available carbon source (in the form of BOD or VFAs) in the influent to the process at the plant where the upgrades are being evaluated. Some plants might require additional carbon sources, whereas others might not.
- ***Additional Electricity.*** An increase in electricity usage is expected in cases where the upgrade requires the addition of pumps because of increased hydraulic head, additional reactor volume (to aerate or provide mixing), or new or increased return rates. A table entry of *Maybe* indicates that electricity usage depends on the existing process at the plant and the type of upgrade being evaluated.
- ***Chemicals Needed.*** This column is included under *Operation Factors* to emphasize the cost associated with chemical addition. If VFAs are generated by using an on-site fermenter, no additional chemical cost would be expected at the plant, although electricity usage might increase somewhat because of the power required to operate the mixer. Other chemicals that might need to be added include caustic soda or lime for alkalinity control and metal salts, such as alum or ferric chloride, for phosphorus removal.
- ***Additional Sludge.*** Additional sludge generation is typically associated with chemical addition. If the process or the site-specific conditions at the plant being retrofitted require chemical addition, additional sludge will be generated compared to the amount generated by the existing process. Plants that do not need chemical addition are unlikely to generate significant additional sludge, unless an expansion is being performed in conjunction with the plant upgrades. All additional sludges will typically incur additional disposal costs.

5.3.1 Nitrogen Removal

Site Factors

Footprint

The footprint of the selected process is important in any retrofit because space is typically limited. For retrofitting an existing activated-sludge system, an additional footprint could be avoided with the MLE process. MLE is a single-stage, two-basin system in which the anoxic zone precedes the aerobic zone, with recycling of aerobic mixed liquor to allow the nitrate arising in the aerobic zone to be denitrified. This configuration has the advantage of not requiring an additional carbon source for the anoxic process because the anoxic microorganisms are mixed with the full-strength waste from the primary clarifier. The disadvantages of the process are twofold: the overall nitrogen removal is limited by the mixed-liquor recycling rate, and the oxygen included in the recycle stream reduces denitrification (WEF 2006). Thus, MLE is suitable where the site is limited and the level of treatment required is in the mid-range between 7.5 and 8 mg/L for TN.

The 3-stage Westbank process is similar to the 5-stage Bardenpho without the last two zones. In addition, the 3-stage Westbank process has the capability to feed a portion of the primary effluent to the head of the process, the anaerobic zone, and/or the anoxic zone. This step-feed approach results in a smaller anoxic zone compared to prior practice because of a steady supply of readily biodegradable carbon for denitrification.

Denitrification filters provide the smallest footprint for a newly installed process, and they are a proven option for achieving low nitrogen concentrations. The technology involves passing secondary effluent through a deep-bed filter that contains denitrifying organisms. The process has the additional advantage of acting as a filter for removing suspended solids. Implementation requires capital expenditures for equipment, building, and costs associated with a pumping station where available hydraulic head is limited. In addition, denitrification filters require an external carbon source, typically methanol, which results in a significant chemical cost. The effluent TN concentrations can be below 3 mg/L; they are less than 2 mg/L at some facilities.

For ammonia removal, using an integrated fixed-film activated sludge (IFAS) technology or a moving-bed biofilm reactor (MBBR) technology offers a small footprint as a retrofit into aeration basins or as an add-on process at existing facilities (Copithorn 2007).

Building Needed

Most of the retrofit systems for nitrogen removal do not need additional buildings because the systems use existing aeration basins or construction of external anoxic zones. Filters, such as denitrifying filters and upflow biological aerated filters, could be housed in buildings in cold-weather locations. Certain manufacturers use biological upflow filters in integrated systems designed to achieve nitrification and denitrification in a single unit. In addition, upflow filters can be operated aerobically to achieve nitrification (WEF and ASCE 2006).

Construction in Existing Aeration Basin

Most of the retrofit technologies make use of existing aeration basins because of site limitations and the need to increase capacity in existing tanks. Where needed, partitions are installed to create zones, and screens are installed to hold microbial carrier media, if required.

Such media are used in both the MBBR and IFAS systems (Johnson et al. 2005; McQuarrie et al. 2004). Example installations include denitrification by IFAS using sponge media in Fairfield, Connecticut; ringlace in Annapolis, Maryland; and BioWeb in Windsor Locks, Connecticut. Nitrification examples include Kaldnes media in Broomfield, Colorado, and Cheyenne, Wyoming (Loader 2007). Readers are encouraged to review the process descriptions for MBBR and IFAS in Chapter 2.

The phased isolation ditch (PID) is appropriate only if an oxidation ditch is already present but would require construction in the existing aeration basin. Filtration units, such as denitrification filters and biological aerated filters, are usually not constructed in existing aeration basins. The MLE, 3-stage Westbank, and 4-stage Bardenpho processes would likely require construction in the aeration basin to install baffle walls and internal recycle lines. If sufficient volume is available, anoxic and anaerobic zones (in the case of the 3-stage Westbank and 4-stage Bardenpho process) could be constructed in the existing aerobic tank. The step-feed activated-sludge system would require constructing anoxic basins and feed points within the aeration basin. If sufficient volume is not available, some of the alternating anoxic and aerobic basins could be constructed outside the existing aeration basin.

Piping and Pumping/Extra Head Needed/Secondary Process Recycle Streams

Most of the nitrogen removal technologies involve secondary recycles. Exceptions include the MBBR, denitrifying filters, sequencing batch reactor (SBR), cyclic activated sludge (CAS), anoxic zone following aeration, and PID. The piping and pump size for the secondary recycle depend on the design flow, but they must be sufficient to maintain the needed nutrient concentrations in each zone so that nitrate is removed to the target level (WEF and ASCE 2006). The step-feed activated-sludge and 3-stage Westbank processes would also require piping for the multiple feed points to the anoxic zones.

Pumping would be required in plants with denitrifying filters because sufficient head following the aeration basins is not usually available. The filters could be constructed as a new add-on process such as the Tetra Filter or Leopold filter or as a retrofit of existing granular media filters into biological anoxic filters (BAFs).

Wastewater Factors

Additional Carbon Source Needed

With the exception of the step-feed activated-sludge process, any of the biological nitrogen removal processes could need additional carbon, depending on the primary effluent characteristics at the specific plant. The carbon source could be methanol or a VFA source inside or outside the plant. VFAs can be added directly in the form of acetic acid, brewery waste, molasses, or other sugar forms. A fermenter could also be used to generate VFAs on-site from primary sludge, RAS, or the supernatant from the anaerobic digesters. Additional discussion is presented in Section 5.4 of this chapter, Design and Operational Factors in

Biological Nutrient Removal. Methanol or MicroC would need to be added for the denitrification filter to function properly.

Operational Factors

Additional Electricity

If internal recycle lines are not included in the existing process, electricity costs would likely increase when implementing biological nitrogen removal. Electricity costs would also increase with the use of a denitrification filter or biological aerated filter because of the pumping that would be required.

Chemicals Needed

All TN removal applications require adequate influent alkalinity concentrations for nitrification to occur. The denitrification process restores some of the alkalinity consumed during nitrification, but there is a net loss of alkalinity through the TN removal process. In soft-water regions, nitrification consumes alkalinity, lowers pH, and impairs biological processes. Adequate alkalinity is maintained by adding lime or caustic when influent alkalinity levels are low. Maintaining a target pH between 6.2 and 6.5 standard units (s.u.) will minimize the cost of adding lime or caustic soda in nitrifying facilities. Chapter 2 presents more details on alkalinity.

As mentioned previously, a carbon source in the form of methanol or VFAs might also be required for biological nitrogen removal. The filter options would also require the addition of methanol or an alternative carbon source for nitrogen removal.

Additional Sludge

Upgrades that require the addition of methanol would produce additional sludge compared to the existing process. If chemical addition is not needed, the sludge yield would not be expected to change significantly.

5.3.2 Phosphorus Removal Plant Factors

Site Factors

Footprint

Chemical addition of metal salts, such as ferric chloride or alum, requires a minimal footprint for installing chemical feed pumps and a storage tank. The capital cost is low, but O&M costs are high because of ongoing chemical costs. Chemical feed doses increase exponentially as the phosphorus effluent goal approaches 0 mg/L; the feed dose curve is particularly steep in the range of 0 to 0.3 mg/L. The minimum phosphorus concentration that can be achieved economically through chemical addition is dependent on the wastewater characteristics and the effectiveness of the clarifiers. Polymer addition is practiced in many plants to aid settling, with minimal site requirements. Chemical addition can increase the effectiveness of other phosphorus removal technologies, such as tertiary clarifiers and filters. Therefore, chemical addition is rarely used as the sole phosphorus removal technology to achieve low-level phosphorus limits (less than 0.2 mg/L).

Membrane bioreactors (MBRs) have a relatively small footprint because of the higher mixed-liquor suspended solids (MLSS) concentration in the aeration basin when compared to conventional plants that rely on clarifiers to remove solids. The pores in the membranes form a physical barrier to particles/floc that is larger than the pore size (and any biological growth that forms on the membrane). Clarifiers require floc of sufficient size and density to settle so the floc can be removed from the flow stream. The higher MLSS concentration in the MBR system allows more biomass to be treated in a smaller tank.

The SBR and CAS systems have smaller footprints than the other activated-sludge alternatives because clarifiers are not needed to settle the mixed liquor. The settling occurs in the same tank as the reaction phase.

The anaerobic/oxic (A/O) process consists of three anaerobic cells upstream of an aerated zone. This process might require little additional footprint in a retrofit application if sufficient volume capacity is available in the existing aeration basin. Otherwise, additional tankage would need to be constructed for the anaerobic cells. A fermenter might be required if additional VFAs are needed to meet biological phosphorus removal requirements.

Alternatives that include the 5-stage Bardenpho process would require a large footprint because two anoxic zones would be needed in addition to the anaerobic zone. The 5-stage Bardenpho process is usually geared more toward removal of both phosphorus and nitrogen, rather than phosphorus removal alone. However, the process denitrifies the nitrates produced during nitrification, reducing the amount of nitrates in the RAS, which increases the effectiveness of the anaerobic zone. The 5-stage Bardenpho process with chemical addition and filtration would require a very large footprint. The PID also has a relatively large footprint because two oxidation ditch tanks would need to be constructed. The infiltration bed would require the largest footprint among all the technologies, but it also provides the highest level of removal with low O&M costs.

Buildings

Buildings are usually required for chemical feed equipment and filtration processes in areas of the country that experience low winter temperatures. The chemical needs to be protected to avoid freezing, which can decrease the efficiency or make feeding the chemicals difficult. Blowers that provide air to the aeration tanks would need to be installed in a building for ease of maintenance as well as for noise control. The fermenter, anaerobic, anoxic, and aerobic tanks typically would not need to be covered or constructed inside a building.

Construction within Aeration Basins

Anaerobic zones could be constructed in existing aeration basins if sufficient capacity would remain for the removal of BOD and ammonia nitrogen (if needed). SBRs are usually constructed at new plants rather than as a retrofit alternative because the tank is usually deeper than a typical aeration basin and extensive piping changes would be required.

Chemical feed equipment, fermenters, and filters would all be constructed outside the aeration basin. Piping to connect any of the chemical feed pumps, fermenter tank, or filter backwash system to the aeration basins would be needed, but construction within the aeration basins would be minimal. The PhoStrip process treats a portion of the settled mixed liquor from the secondary clarifier. Therefore, construction within the aeration basin would not be necessary, but modifications to the RAS system would be needed.

Conversion to an MBR system could involve significant construction within the aeration basins. In addition to installing the membranes, a portion of the aeration basin might be converted to anaerobic and anoxic zones to promote biological phosphorus removal and alkalinity recovery. If sufficient aerobic capacity is not available at the existing plant, the anaerobic and anoxic zones, as well as the membrane tank, could be constructed outside the aeration basins.

Piping and Pumping/Extra Head/Secondary Recycle Streams

The 5-stage Bardenpho process, MBRs, and filters all have return streams. Internal return streams are part of the 5-stage Bardenpho and MBR processes, and backwash is returned during cleaning of the filters. Filter backwash can be returned to the head of the plant or to the secondary process. Phosphorus concentrations in the filter backwash could be high. Returning the backwash to an equalization basin to allow the flow to be slowly blended with the raw influent might improve the phosphorus removal efficiency by not introducing a slug of phosphorus all at once. Alternatively, sidestream treatment of the filter backwash, which could involve settling the solids before returning the flow, could also improve phosphorus removal at the plant.

Under most circumstances MBRs require additional head to draw wastewater through the membranes at the end of the process. Certain manufacturers might be able to provide a gravity-fed system in special situations, depending on the hydraulics of the rest of the plant. Pumping is generally required at plants that select filtration to achieve phosphorus removal.

Wastewater Factors

Additional Carbon Source Needed

In biological phosphorus removal, a sufficient concentration of VFAs is required. VFAs are present in sufficient quantity in the wastewater in many locations but need to be supplemented in other locations. Where needed, VFAs can be added directly from an external source in the form of acetic acid, brewery waste, molasses, or other sugar forms. A fermenter could also be installed to generate VFAs on-site from primary sludge, RAS, or the supernatant from the anaerobic digesters. Carbon sources would not be necessary for process options that rely on chemical phosphorus removal or filtration, unless they are used in conjunction with biological phosphorus removal. Additional discussion is presented in Section 5.4 of this chapter, Design and Operational Factors in Biological Nutrient Removal.

Operational Factors

Additional Electricity

Anaerobic zones and fermenters require mixers that use a relatively small amount of electricity. Electricity usage increases would be expected with processes that have an internal recycle, such as the Virginia Initiative process (VIP) and 5-stage Bardenpho, whereas a significant decrease would be expected from a reduced aeration volume in the tank. The MBR process requires additional electricity for aeration and pumping. Chemical phosphorus removal would require the addition of pumps; however, the pumps would be relatively small, and electricity usage would be expected to be minimal.

Chemicals Needed

Phosphorus removal technologies could require adding metal salts like alum or ferric chloride, particularly in plants required to meet low phosphorus limits. Because of the possibility of an upset in the biological phosphorus process, many plants incorporate chemical phosphorus removal as a back-up system to ensure that permit limits can be met. At facilities that must meet an extremely low phosphorus limit, two or more feed points for coagulants and polymer addition would be expected.

The PhoStrip process treats only a portion of the RAS. Following the anaerobic tank, chemical precipitation, using lime, removes the phosphorus from the stripper overflow before returning the rest of the flow to the primary clarifier. Because only a portion of the wastewater is treated through the PhoStrip process, the amount of chemical needed is significantly less than what would be required in a secondary treatment process.

Additional Sludge

Upgrades that incorporate chemical phosphorus removal would be expected to generate additional sludge, especially at facilities that are required to feed a high dose of chemicals to reach a low level of phosphorus, such as below 0.5 mg/L and especially below 0.2 mg/L. Biological phosphorus removal, however, would not generate additional sludge. In some circumstances, less sludge might be generated if the coagulant was fed to the primary clarifiers, thereby reducing the BOD and total suspended solids (TSS) loadings to the secondary process.

5.3.3 Nitrogen and Phosphorus Removal Plant Factors

Most of the processes listed in Table 5-7 were included in Table 5-5 or 5-6 but without certain minor modifications (e.g., step-feed and IFAS include selectors in Table 5-7, but not those in Table 5-5). The selector would consist of an anaerobic tank to promote biological phosphorus removal. As with the A/O process, the anaerobic tank could be constructed in the aeration tank if sufficient capacity was available to accommodate the anoxic and aerobic zones needed for treatment. No building would be required for the anaerobic tank. No internal recycles are associated with the selector beyond the RAS line.

The PhoStrip II process is similar to the PhoStrip process, except a pre-stripper tank is upstream of the stripper tank to remove nitrates before they enter the anaerobic zone.

As noted in Table 5-4, chemical precipitation, tertiary filtration, or both could be used to provide phosphorus removal at the end of any of the processes designed solely for nitrogen removal. Therefore, readers should consult Table 5-5 in conjunction with Table 5-7 to identify other feasible upgrade alternatives for plants attempting to achieve both nitrogen and phosphorus removal. Adding chemical and physical phosphorus removal technologies, as opposed to biological removal, might be particularly attractive at a plant that already is removing nitrogen.

5.4 Design and Operational Factors in Nutrient Removal

The following factors are critical in assessing existing facilities and developing feasible alternatives:

- Influent wastewater characteristics
- Sources of biodegradable carbon
- Impact of wet-weather flows
- Managing sludge-handling processes
- Recycle flows
- Supervisory control and data acquisition (SCADA) requirements and sensors
- Staffing requirements
- Training needs
- Pilot testing

The project team members are encouraged to consider all these factors and incorporate them in developing feasible technology alternatives.

Influent wastewater characteristics are important to consider because the technology being considered might not operate properly if the plant cannot provide the conditions required. For example, if the BOD-to-TP ratio is too low in the primary effluent, biological phosphorus removal will not occur consistently at the plant without a supplemental carbon source. For plants that need to increase biodegradable carbon, considering possible sources is important. Such sources could include chemical addition or in-plant generation by constructing fermenters.

Permit limits for nitrogen, phosphorus, or both must be met during dry- and wet-weather conditions. Wet-weather flows can be significantly higher than average dry-weather conditions for plants with combined collection systems or separated sewers that have high rates of inflow and infiltration. Biological systems can lose solids during elevated flow periods at the plant. Biological processes rely on sufficient biomass to remove nitrogen, phosphorus, or both. Chemical and physical processes are also less efficient under elevated hydraulic conditions. Considering alternative operation strategies will minimize the possibility of exceeding permit limits because of wet-weather events. For example, the North Cary Water Reclamation Facility in North Cary, North Carolina, was able to comply with all permit limits when 8 inches of rain fell during Tropical Storm Alberto in 2006 by diverting water to equalization and operating the BioDenipho process in the storm mode. For more information on this plant, see the North Cary Case Study in Volume II, Appendix A.

The return flows from sludge handling can contain significant amounts of nitrogen and phosphorus, which can organically overload the removal process and cause poor performance and the possibility that the plant’s permit limits will be exceeded. Reviewing methods to minimize or treat the return of nitrogen and phosphorus and incorporating them into the plant design will improve the operation of the selected process. In addition, recycle flows from other processes, such as internal recycles, RAS, and filter backwash, can also affect the nitrogen and phosphorus removal process, particularly if biological removal has been selected. For example, high concentrations of dissolved oxygen in the RAS can negatively affect anaerobic and anoxic zones. It takes time for the microorganisms to use the dissolved oxygen, turning part of an anaerobic tank into an anoxic zone or a portion of the anoxic tank into an aeration zone. The smaller effective volume could decrease the efficiency of the process.

Operation of the selected process could be optimized by incorporating SCADA and operational sensors that can be programmed to make changes using real-time analyses. Optimization could include diurnal variations in the incoming wastewater or occasional changes in the influent wastewater characteristics. If the operational strategies are programmed, the plant can react to changes much more quickly, instead of waiting for results from the laboratory, which could be hours or days old. For example, online monitoring can automatically adjust the feed rate of coagulant to be fed for chemical phosphorus removal or the methanol dose in denitrification filters. The alternative would be to wait for the laboratory results to show that the effluent concentrations were elevated for several days and then have to feed more chemicals to ensure that the monthly permit limit was met.

A plant upgraded to provide nutrient removal cannot function properly without enough staff to operate the processes and perform maintenance activities. In addition, it is important that the staff be trained to operate the upgraded processes properly.

Pilot testing of a limited number of upgrade process alternatives is highly recommended to verify that the process will operate as anticipated on the wastewater at a given location. Many of the processes depend on the proper influent wastewater characteristics and water chemistry to provide the predicted nitrogen or phosphorus concentrations. By incorporating pilot testing into the selection or design phase, accommodations can be made to amend the influent wastewater, if required. For example, if it is found during pilot testing that VFAs are deficient upstream of a biological nitrogen removal process, a fermenter can be incorporated into the design and constructed as part of the upgrade instead of incorporating it as an expensive retrofit after most of construction has been completed.

A key issue is to understand the interrelationship among the liquid treatment and solids-handling processes and to quantify and size all unit processes properly. The mass balance and energy balance should include all recycle loads for the selected technology. Preparing a mass balance during the planning stage can contribute greatly to a successful upgrade project. The mass balance needs to include all the nutrient inputs and outputs for each unit process. The return flows from sludge-handling processes and filter backwashes that are quantified as part of the mass balance can then be incorporated into the basis of design for the upgraded plant. This approach will ensure that the processes are designed to handle the nutrient loadings from these sources.

Another key question is how to manage the uncertainty in the future—changing regulations and the need for more stringent nutrient removal, variable wastewater characteristics, and phasing of future population growth in the service area. Design flexibility might be needed in anticipation of future uncertainties. It can be incorporated by constructing additional process tanks or swing zones in the secondary process. The swing zone could be between the anaerobic and anoxic zones, the anaerobic and aerobic zones, or the anoxic and aerobic zones, depending on the secondary process selected. For example, a swing zone between the anoxic and aerobic zones would contain both mixers and diffusers. If it was determined that additional residence time was needed in the anoxic zone for denitrification, the mixers in the swing zone could be operated. If additional aeration was needed for nitrification, the diffusers could be operated. Depending on the season and influent wastewater characteristics, the swing zone could be operated in either mode. Switching between modes on the basis of the wastewater characteristics would be relatively easy to do through the use of automated controls. The following sections provide additional information on the critical factors in design, operation, or in many cases both.

5.4.1 Influent Wastewater Characteristics

The evaluation begins with a review of the wastewater data described below, if available. If few or no data are available, data should be collected, especially at the influent, primary effluent, other in-plant sources, and secondary effluent if tertiary processes are being considered. It is recommended that at least a full year of data be collected to quantify seasonal variations.

Biological Phosphorus Removal

Data needed include the BOD-to-TP, VFA-to-TP, and readily biodegradable chemical oxygen demand (rbCOD)-to-P ratios in the plant influent and primary effluent. A BOD-to-TP ratio of 25 or higher at the bioreactor influent is desirable as a general guide when there are no data for the influent to the bioreactor. At the wastewater treatment plant in Durham, Oregon, the performance of the enhanced biological phosphorus removal (EBPR) was unstable at a BOD-to-TP ratio of 20 or below at the aeration influent (Neethling et al. 2005). A VFA-to-P ratio of 4.3 was observed. At the McDowell Creek, North Carolina, facility, the BOD-to-TP ratio remained above 30 and had favorable EBPR performance. The COD-to-TP ratio should exceed 8 in the plant influent (Tremblay et al. 2005), while the rbCOD-to-P ratio should exceed 11 (Barnard et al. 2005) or 18 with a fermenter (Barnard 2006). The important point to monitor is at the influent to the bioreactor, not at the plant influent. An assessment should be made to determine whether the supply of VFAs is sufficient year-round, especially in cold months when VFA production is lowest in the sewer system and incidental sources at the wastewater treatment plant. The decision to install a fermenter should be based on this assessment. As Chapter 3 of this document notes, the benefits of having a fermenter on-site were documented in Kalispell, Montana, and Kelowna, British Columbia. The phosphorus removal at those plants was achieved strictly by a biological process using the modified University of Cape Town (UCT) process and 3-stage Westbank process, respectively. The reliability graphs indicate low coefficient of variation (COV) values at both plants.

Chemical Phosphorus Removal

Jar testing is recommended to determine the design parameters for chemical phosphorus removal. Jar tests can be conducted to compare coagulants, such as lime, ferric chloride, and alum, to determine which one provides the most efficient and cost-effective removal. The tests should be conducted on the wastewater from the location where the chemical will be added. Mixing, flocculation, and settling times that approximate the conditions that will be found at the chemical feed point and settling location should be selected. Several doses of each coagulant should be tested, and the quantity of sludge produced from each should be compared before selecting which one will be used at the plant. Analyzing the clear liquid for TP will aid in determining which coagulant provides the best removal. Polymer can be combined with the coagulant testing to determine whether performance can be improved. The jar tests can also be used to determine the recommended chemical feed dose by focusing on the value that provided the best result. For example, if alum was selected as the coagulant and a dose of 10 mg/L as aluminum produced the clearest effluent during the comparison of coagulants, performing additional jar tests at 6, 8, 10, 12, and 14 mg/L as aluminum would help to identify the best feed dose for use in the design.

Titrations can be conducted to determine the alkalinity of the solids produced during the jar testing. The pH, TSS, and volatile suspended solids in the settled precipitate can also be analyzed to estimate the amount of chemical sludge that will be produced in the upgraded plant. This information can be used to ensure that sufficient sludge-handling capacity is included in the design for the upgraded plant.

Chemical addition can also be used to remove phosphorus from recycle flows, such as RAS, filter backwash, or return from sludge-handling processes. Lime, alum, or ferric chloride could be used in this application. Jar tests for these chemical could be performed. Sludge quantities produced from chemical addition could be estimated by analyzing the settled solids for alkalinity, pH, ortho-phosphorus, ammonia nitrogen, and TSS.

Nitrogen Removal

Useful data include the BOD-to-total Kjeldahl nitrogen (TKN) and COD-to-TKN ratios in the plant influent and primary effluent. Both ratios are important factors to achieve low TN effluents. Low ratios might limit the nitrogen removal efficiency. Grady et al. (1999) reported a BOD-to-TKN ratio in the range of 6.8 to 9.5 for near-complete nitrogen removal. Most wastewater has a COD-to-TKN ratio of 8 to 10 after primary sedimentation. At low ratios, more reliance on endogenous respiration is needed to denitrify. When the COD-to-TKN ratio falls below 9 and high nitrogen removal is required, providing some form of methanol, acetate, or other carbon source is necessary. Alternatively, fermentation would produce supplemental carbon to increase the BOD-to-TKN ratio to promote nitrogen removal. When the ratio is expected to be low consistently throughout the year or periodically (from cold weather, for example), decisions need to be made regarding the source, feed equipment, and storage tanks that will be used to supply carbon.

Temperature Impacts

The impact of temperature on biological removal processes varies depending on the location of the plant in the country. Usually, areas that experience cold winters need to incorporate operational flexibility to successfully treat wastewater at low temperatures. On the other hand, areas of the country that experience wastewater temperatures greater than 30 ^{o}C could experience inhibition of the biological phosphorus removal process and might need to consider using chemical phosphorus removal during such periods. For nitrifying organisms, the upper limit is 33 ^{o}C, beyond which the nitrification rate falls off quickly (USEPA 1993).

The most critical low-temperature impact occurs during spring wet-weather events when the snow and ice melts increase the TSS and BOD in the influent wastewater and at the same time lower the water temperature. Lower wastewater temperatures result in a reduction of the

VFAs from the sewer system and in-plant process sources. With fermenters on-site, both Kalispell, Montana, and Kelowna, British Columbia, reported full EBPR operation at temperatures as low as 8 ^{o}C, as shown in Chapter 3 of this document. Other plants experienced poor biological performance and used chemical addition as a standard procedure when low wastewater temperatures were recorded.

Generally, biological processes operate with a longer sludge age at low temperatures, especially when nitrification and denitrification are required as part of the treatment process. If the process cannot be operated at a long enough sludge age to support nitrification and denitrification, alternative processes should be considered, such as converting the existing process to IFAS or MBBR. Generally, low temperatures do not adversely affect EBPR.

5.4.2 Sources of Biodegradable Carbon

This section discusses how to use wastewater characteristics in selecting a carbon source.

Maximize In-plant Sources

Using primary settling tanks, an anaerobic digester, a thickener, or sludge storage tanks can produce a significant amount of VFAs. A primary settling tank has been used successfully as a VFA generator in many locations. At the Clark County, Nevada, facility, the primary sludge is thickened up to 5 percent to produce VFAs in the range of 35 to 45 mg/L. The upper solids limit was established at 6 percent. However, the VFA production from primary tanks varies from plant to plant. Thickeners and sludge storage tanks are rich sources of VFAs. The Genesee County, Michigan, facility relies on these two sources for a year-round supply of VFAs.

Find Local Commercial or Industrial Sources

Local and industrial sources can provide a good supply of readily biodegradable waste materials. At the McDowell Creek plant owned by Charlotte-Mecklenburg Utility in North Carolina, high-level BOD water from a soft drink manufacturer is delivered directly to an anaerobic zone. The waste has a BOD of 40,000 to 130,000 mg/L (Neethling 2005).

Install Fermenters

Installing fermenters by converting existing thickeners or by adding a new fermenter can ensure an adequate supply of VFAs for cold months and wet periods. The reliability of EBPR depends on a continuous supply of VFAs. The Kelowna, British Columbia, facility converted its existing thickeners. The Kalispell, Montana, facility designed a two-stage fermenter system. A new generation of fermenters provides improved mixing and effective controls. The minimal solids retention time was 2.5 days to generate sufficient VFAs at the McDowell Creek Plant (Tremblay 2005). The fermenters were also an additional carbon source for nitrogen removal. The feed points can be prioritized between phosphorus removal and nitrogen removal on the basis of the performance of both. For example, the VFAs from the fermenter can be sent to either the anaerobic zone or the anoxic zone, depending on the need at the time. In Kelowna, British Columbia, VFAs are sent to both zones, thereby reducing the size of the anoxic zone significantly. While the fermenter requires land to build on, the reduction in the anoxic zone can result in an overall decreased land requirement. The critical factor to note is the improved reliability of the BNR process when operated in conjunction with a fermenter.

Temperature Impacts

The impact of temperature on biodegradable carbon sources varies depending on the location of the plant in the country. Temperature has a greater effect on areas that experience cold winter temperatures. Lower wastewater temperatures result in a reduction of the VFAs from the sewer system and in-plant process sources. Fermenters in the northern portions of the country need to be sized such that sufficient VFAs are produced to support BNR at the lowest wastewater temperatures anticipated.

5.4.3 Impact of Wet-Weather Flows

This section describes how the management of wet-weather flows can affect process selection. Although this issue is not usually severe in municipalities with separate storm sewers, it is an important issue in communities with combined sewer systems or collection systems that have high inflow and infiltration rates. The peaking factor and permit conditions are usually established for the critical discharge season.

Two factors are noteworthy in selecting technologies. First, certain technologies handle wet-weather flows better than those based on activated sludge and clarification. They include denitrification filters, fixed-film processes such as MBBR, and step-feed activated sludge. Second, off-line storage of peak flows can be a critical requirement.

During wet-weather events, equalization basins can be used to store as much water as possible, if they are available. If equalization basins are not part of the existing plant, it is recommended that they be included as part of the upgrade project. The equalization basin can be operated as an on-line process in that all wastewater flows through the basin as part of normal operation. Alternatively, the equalization basin could be an off-line process, such that flow would be diverted to the equalization basins only when needed during wet-weather events. The equalization basin could be located in the sewer system, at the headworks, or following the primary clarifiers or secondary clarifiers. Placement after the secondary clarifiers would be recommended only if the nutrient removal processes to be protected from elevated hydraulic flow rates are tertiary clarifiers or filters. The size of the equalization basin should be based on the flow patterns that the plant experiences (including diurnal flow variations and wet-weather volumes received). The permit limits also should be considered when sizing the equalization basins: more equalization volume would be required at plants that need to meet a daily nitrogen or phosphorus limit, while plants with monthly or yearly limits might be able to use smaller equalization basins.

How the plant is operated during wet-weather events depends on the specific unit processes available. For plants that would like to continue the normal mode of operation during wet-weather events, converting the existing facilities to a step-feed activated-sludge or IFAS system is a possible alternative. Both processes protect the MLSS, so they are designed to maintain the biomass inventory in the process train during elevated hydraulic flow periods.

Another alternative would be to construct new facilities to address wet-weather flows. For example, additional secondary clarifiers could be constructed to decrease the surface overflow rates during wet-weather events. Step-feed activated-sludge, CAS, PID, or IFAS systems could be constructed as new facilities to handle wet-weather flows. The PID would need to be operated in storm mode, which can be controlled by a SCADA system, similar to the operating mode used at the North Cary Water Reclamation Facility in North Cary, North Carolina. Storm mode operation includes allowing flow to enter and leave only the outer ring of the ditch. Wastewater within the middle and inner channels continues to circulate, but it is not discharged to the clarifier until after the wet-weather event has ended, thereby retaining the MLSS, which are then available to repopulate the outer channel. New storage tanks could also be used to store recycle flows during wet-weather events.

The existing or upgraded facilities could also be operated using a wet-weather mode specific to the plant. For example, the return of recycle flows could be temporarily suspended during wet-weather events, or the aeration zone(s) could be shut down for up to 24 hours. These are common practices used to protect secondary clarifiers from solids overloading. In addition, temporarily shutting down the sludge-handling processes is an option, if feasible.

5.4.4 Managing Sludge-Handling Processes

Managing sludge-handling processes is one of the key factors to consider in selecting a nutrient removal process and making the upgrade successful. As described below, both quantity and quality are of concern in sludge management and recycle flows in a nutrient removal process.

- ***Quantity.*** Upgrading from the normal secondary process to the advanced treatment level might increase the sludge quantity (and alter the quality) significantly. Adequate capacity should be incorporated into the design. Chemical phosphorus removal increases the quantity of sludge. The quantity could double if the target phosphorus concentration falls below 0.5 mg/L and could triple if the target falls below 0.2 mg/L because of the increased capture of fine particles through tertiary filtration (USEPA 1987). No increases in sludge, however, were reported from conversion to biological phosphorus removal when operated at a long sludge age. Similarly, no additional increase in sludge production was reported for conversion to nitrogen removal, except in the case where methanol or another carbon source was added. In both examples, the sludge age was a key parameter. The biological sludge quantity is inversely proportional to the design sludge age (WEF and ASCE 1998). The sludge-handling processes might need to be sized up to handle the increased quantity of sludge from the upgrades.

- ***Quality.*** In terms of quality of sludge and recycle, significant changes can be expected. They include changes in thickening and dewatering due to changes in the composition of the sludge from the new technology. For example, the chemical sludge produced from phosphorus removal from iron salts is easier to settle, thicken, and dewater (USEPA 1987) than from traditional activated sludge processes. Tertiary filters, however, capture fine particles and make sludge thickening, dewatering, and digestion more challenging. BNR sludge with a long sludge age tends to digest well but results in significant recycle loads to the main treatment train (see the next section). In some BNR facilities, primary settling and anaerobic digestion are eliminated, and thus the sludge quality is changed drastically; aerobic thickening and aerobic digestion reduce the recycle loadings back to the main train, and the digested sludge might require different chemicals for dewatering.

5.4.5 Recycle Flows

Recycle flows can introduce significant variability in the treatment plant flows and loads. It has been reported that a total of 15 percent to 50 percent of the phosphorus removed originates in the recycle flows (WERF 2005). The recycle flows to be evaluated are flows from sludge-handling facilities and backwash water from tertiary filters and tertiary clarifiers because those flows can carry nutrients that are not removed with the sludge back upstream in the treatment process.

Anaerobic digestion releases high concentrations of ammonia. High levels of phosphorus can also be released at an EBPR plant. Return flows can upset the carbon-to-nitrogen ratio or the carbon-to-phosphorus ratio required for effective BNR processes. This is particularly true at regional facilities that handle sludge from multiple plants or at facilities that receive large volumes of septage.

In many cases, the magnitude of loadings from recycle flows can be minimized. Following are recommendations for properly handling recycle flows.

The first recommendation to minimize the recycle impact is to keep the sludge aerobic for EBPR upgrades. This means keeping the sludge aerobic at each step and all the way through the sludge-handling processes—digestion, thickening, and storage. Two examples are noted in Chapter 3. All waste-activated secondary sludge (WAS) is kept aerobic at the Kelowna, British Columbia, facility, while the primary sludge is separately fermented and stored before dewatering. No digestion is provided at the Kelowna plant because the sludge is shipped off-site for composting. The recycle loads contained 13 percent TP and 0.1 percent TKN. When lime was added before dewatering, the phosphorus load to the EBPR process was further reduced to 6 percent. When sludge is digested, the recycle loads increase. At the Kalispell, Montana, facility, the secondary sludge is thickened using dissolved air flotation, while the primary sludge is anaerobically digested. The primary and secondary sludges are kept separate until the time of dewatering. This operation is noteworthy in avoiding a potential increase in recycled phosphorus loads if these two sludges were to be combined and stored for any time.

The second recommendation is to plan for recycle loading from anaerobic digestion and dewatering, especially from BNR facilities. The recycle phosphorus loads were reported to be high in the facilities with anaerobic processes—25 percent of the total influent phosphorus loading at the Durham, Oregon, facility and the McDowell Creek, North Carolina, facility and 50 percent at the Lower Reedy Creek, Florida, facility (WEF 2005). Jones and Takacs (2004) reported a modeling technique that can be used to estimate recycle loads in such cases. They reported recycle loadings of 35 percent to 50 percent in phosphorus and 20 percent to 30 percent in ammonia nitrogen from the anaerobic digester operating at a 20-day sludge age in their modeling when the volatile suspended solids destruction was assumed to be 35 percent. These authors used the General Activated Sludge–Digestion Model (ASDM), which was developed and implemented in BioWin 2.1. They also identified the variables that can reduce recycle loads on the basis of the digestion detention time, equilibrium chemistry, and formation of struvite in the system. Tang et al. (2004) reported recycle loads of 50 percent for ammonia nitrogen in comparison to the plant influent loads at the Valencia Water Reclamation Plant in the Los Angeles County Sanitation District, where both primary and secondary sludge were anaerobically digested.

At non-EBPR plants, the recycle loads are less than those from EBPR plants; they are typically less than 15 percent. Anaerobic digestion is the main source of ammonia and phosphorus recycle back to the main plant. Under a normal operating schedule, this load can be managed successfully by carefully controlling the operating schedule and implementing certain strategies. These include flow equalization to avoid shock loadings, filter backwash to an equalization basin with sufficient storage capacity, and proper controls. At nitrogen removal plants, it was not suggested that the nitrogen in recycle be removed. Ammonia nitrogen is high in anaerobic digester supernatant and in the liquid from dewatering such sludge.

For EBPR plants, it is suggested that WAS be kept separate from primary sludge and be kept aerobic, as described earlier. The following guidelines are suggested for biosolids thickening, stabilization, and storage facilities:

- For sludge thickened by dissolved air flotation or other mechanical thickeners (rotary belt thickeners), aerobic digestion is preferred.

- Storage tanks that have aeration capability and are large enough to hold biosolids are recommended.
- An equalization basin for recycle flows might be needed.
- The dewatering schedule might need to be modified to alter when the recycle loads are returned from the day shift to a later shift.
- Rerouting the feed point to the bioreactor to maximize VFAs and other beneficial ingredients, that is, step feed to the anaerobic and anoxic zones, or to one of them, depending on the priority at the time, should be considered.
- Adding lime to the centrifuge to minimize the amount of recycle load should be considered.

The centrifuge at the Noman Cole plant in Fairfax County, Virginia, was designed to add lime to reduce recycle loads at the dosage of up to 13 percent of the solids loading. To remove most of the soluble phosphorus, the Kelowna, British Columbia, facility fed lime to centrifuges to a pH of 9. This practice ended in 2006. The net increase of phosphorus in recycle loads was dramatic, while the TSS loads were reduced as illustrated below:

- *Soluble COD*: 417 to 700 mg/L, a 67 percent gain
- *Soluble P*: 34 to 137 mg/L, a 405 percent gain
- *Total P*: 109 to 200 mg/L, an 82 percent gain
- *Ammonia nitrogen*: 15 to 21 mg/L, a 39 percent gain
- *TKN*: 52 to 68 mg/L, a 30 percent gain
- *TSS*: 1,031 to 765 mg/L, a 24 percent decrease

The Kelowna plant determined that the increase in recycle loads in both nitrogen and phosphorus was manageable because of the availability of VFAs from the fermenter, wastewater characteristics, and other process control parameters in place.

The third recommendation is that facilities should consider converting anaerobic digestion to aerobic digestion as part of long-term planning, where it is found to be cost-effective. Consideration should also be given to the potential loss of *green energy* when deciding how to act on this recommendation.

For drying, composting, or other Class A products, additional guidelines might apply.

In addition, the following guidelines apply for certain facilities with tertiary clarifiers and tertiary filters:

- Consider adding an equalization basin for filter backwash water so that the flow can be distributed evenly throughout the 24-hour period, thereby controlling the power usage and chemical feed rate evenly. Most facilities send this water to the headworks or equalization basin, thereby reducing shock loads to the rest of the plant.
- Add thickener for tertiary clarifier sludge. Because the waste sludge is dilute with a low solids concentration, thickening of the tertiary sludge is recommended.

All the above should be incorporated into the design basis (flow and loadings), and each unit process should be evaluated for efficient and reliable performance.

5.4.6 SCADA Requirements and Sensors

Most BNR facilities have adopted automated control systems, which include dissolved oxygen controllers operating with a programmable logic controller (PLC) and associated SCADA. In other facilities more specific controls, based on other sensors for nitrate, oxidation-reduction potential (ORP), and flow have been adopted. Each facility has developed specific programs on the basis of its permit limits, the skill levels of personnel, and the technology employed at the facility. In other facilities, more specific controls using a combination of sensors have been installed. Available sensors included ammonia nitrogen, nitrate and nitrite nitrogen, ortho-phosphorus, and ORP (Weerapperuma and De Silva 2004; Demko et al. 2007). New tools included the sludge blanket monitor, TSS and turbidity meters, and total organic carbon analyzers. Some of these tools have been installed at BNR facilities for accurate monitoring of performance and also as an early warning system for toxic shocks that might be entering the plant. No facility has employed these data in actual control of treatment processes, however. At the North Cary, North Carolina, plant, the oxidation ditch operates in anoxic and aerobic cycles in alternating phases. The exact decision on phasing is made on the basis of preset control logic, which is site-specific and fully automated. Most facilities install PLCs with a human-machine interface and multiple screens connected to the main computer system, flow meters, level switches, online sensors, and system alarms.

5.4.7 Staffing Requirements

Most facilities maintained the same level of staffing after the BNR processes were installed. No new hires were reported for the BNR upgrades. The existing staff was trained on the operation of the BNR process and related new equipment and instrumentation.

5.4.8 Training Needs

In the upgrade projects and case studies included in this document, training in new process operation and monitoring was provided by the project consultant or the manufacturer of the process, when applicable, during the design and construction period. During startup, more detailed training was provided as a part of the construction contract. Follow-up training included seminars and workshops offered by state agencies and professional organizations like the Water Environment Federation (WEF), its local affiliates, and other organizations. Training in process controls and information technology was provided mostly in-house. Laboratory personnel received training from equipment suppliers, where applicable. Additional training could be obtained by attending specialty courses on the subject.

5.4.9 Pilot Testing

In many of the upgrade projects, before selecting a technology, the project team went through a preliminary engineering evaluation and conducted pilot testing of the favored alternative to verify the concept with the actual wastewater to be treated and to develop sizing data for the process. The duration of the pilot testing depended on the seasonal changes in wastewater characteristics, the technologies selected, whether the facility was new versus well established, the number of parameters to operate under, and the specific objectives established. The typical duration ranged from 6 months to 1 year for a biological process. At the Blue Plains facility in Washington, D.C., pilot testing of the process lasted 6 months. For physical and chemical processes, however, pilot testing requires far less time.

This pilot testing can be carried out at the planning stage or delayed until the design phase of the project to accommodate the overall schedule.

5.5 Finalizing Process Selection

The project team's final step is to evaluate feasible alternatives and determine the recommended plan. The project team can compare the potential alternatives and select the best process in accordance with the success criteria established at the beginning of the project.

The reader can obtain information on general cost estimation from Chapter 4 of this document for appropriate technologies being considered. Alternatively, a cost estimate based on upgrading the individual plant to the processes being considered could be performed as part of the evaluation.

The accuracy of the cost estimate will vary depending on the level of detail provided in the evaluation. If the cost estimate is based on cost curves or costs from similar facilities or technologies with very little consideration of local conditions, the cost estimate might be accurate to only within approximately 50 percent. If more detailed studies such as soil borings, preliminary engineering design drawings, and outline specifications are prepared, the estimate will be more accurate. Non-cost criteria should include those items of most concern to the public. At a minimum, odor, traffic, noise, air emissions, dust, water quality, wetland infringement, and other environmental impacts need to be assessed.

Two examples are shown below. Both examples include a weighting factor to indicate the relative importance of each criterion being considered. In Table 5-8, the alternatives are assigned values from 1 to 4 (there are four alternatives), with 1 the most favored alternative for the given criteria. Therefore, the most favored alternative is the one with the lowest number at the bottom of the table. Once this basic rating is complete, a sensitivity analysis can be carried out by changing the weighting factors for one or more criteria items, e.g., increasing the cost of energy by a factor of 50 percent or 100 percent. The process can be repeated for another parameter, such as the future regulatory requirements or biosolids handling. The new ranking can be compared among these alternatives.

Table 5-8. Decision matrix example 1

Criteria for comparison	Weighting factor	Alternatives			
		A	B	C	D
Costs (capital and O&M)	1	2	1	4	3
Reliability	1	3	1	2	4
Efficiency	1	1	2	4	3
Expandability	1	2	3	1	4
Ease of O&M	2	3	1	4	2
Environmental impacts—chemical use	3	4	1	3	2
Efficient land use	1	2	1	4	3
Energy use	2	2	1	4	3
Future regulatory requirements	1	4	3	2	1
Employee and public health and safety	2	2	3	4	1
Market stability for biosolids reuse/disposal	1	1	4	2	3
TOTAL	--	41	28	52	39

Note*:* The rating scale is 1 to 4. Smaller numbers represent a more favorable alternative. No two alternatives may have the same score for the same category. Totals are determined by summing the score for each criterion times the weighting factor, which should be adjusted for the local situation. The alternative with the lowest total is the best overall alternative by this measure.

In the second example, an alternate scale is used to evaluate the alternatives. It is based on assigning a number from 1 to 10 for each criterion, with 1 being the least favored and 10 the most favored. The weighting factor is multiplied by the score to determine the total points. Table 5-9 shows an example from another case (WEF 2004). The bottom line figures are then compared to select the recommended alternative. The highest scorer is the best alternative in this method. The same sensitivity analysis described under the first example can be carried out for the cost of energy, or other parameters of choice, and compared again.

Table 5-9. Decision matrix example 2

Criteria	Weight	Option 1		Option 2		Option 3	
		Score	Points	Score	Points	Score	Points
Site requirement	8	5	40	5	40	4	32
Capital cost	6	3	18	5	30	4	24
O&M cost	6	3	18	4	24	4	24
Reliability	8	4	32	5	40	5	40
Flexibility	10	3	30	4	40	3	30
Use of existing facilities	6	3	18	4	24	4	24
Sludge production	8	2	16	3	24	3	24
Public perception	7	3	21	3	21	3	21
Total points			193		243		219
Rank			3		1		2

The project team can choose which method to employ and the criteria to be included in making the final technology selection.

Once a technology is selected, the next step is to prepare a conceptual design. The design should include a demolition plan, a site plan with major pipes and facilities, a hydraulic profile, process flow and mass balances, general drawings with building and major equipment footprints, one-line electrical drawings, a basic instrumentation and control philosophy and block diagram, and architectural renderings. A construction schedule should also be developed to ensure that the existing plant can continue to operate and meet permit limits until the upgrades are placed on-line. The recommended plan is then presented for public approval and implementation.

5.6 Summary

As more municipalities are required to meet stringent nutrient load limitations to protect receiving waterbodies, upgrading existing facilities with sustainable technologies is an important challenge, as well as an opportunity. This chapter presented the general approaches to upgrading existing facilities, explained how to set success criteria for the upgrade, and provided tables that can be used to screen feasible alternatives, along with selection factors and design and operational factors that can assist in identifying the right technology for the municipality.

Planning for process upgrades includes projecting future loads, assessing existing capabilities, preparing a mass balance that includes all return and recycle flows and loads, developing the needed expansion and upgrade that should incorporate flexibility into the operation of the plant to account for future uncertainties, evaluating feasible alternatives, and selecting the recommended plan. The success criteria might include sustainability, cost-effectiveness, ease of O&M, project schedule, and site requirements. The sustainability factors could include energy usage, chemical usage, and the recycling of biosolids.

A list of technologies capable of meeting the selected target effluent range for nitrogen and/or phosphorus can be developed. Technology selection factors, including the following, should then be reviewed for each alternative:

- Site factors
  - Footprint
  - Need for a building
  - Possibility of construction in the existing aeration basin
  - Piping and pumping requirements
  - Need for additional head
  - Presence of secondary process recycle streams

- Wastewater factors
  - Need for additional carbon source

- Operation factors
  - Extra electricity usage
  - Need for chemical addition
  - Generation of additional sludge

The next step is to identify and evaluate feasible technologies on the basis of design and operational factors and cost factors like the following:

- Wastewater characteristics in the influent, primary effluent, and recycle streams

- Carbon source management, internal and external

- How to manage wet-weather flows

- How to manage recycle flows

- How to design and operate sludge-handling processes

- How much automation and control is needed

- Staffing and training needs

Important considerations include how to incorporate flexibility in anticipation of uncertainties and changing conditions in wastewater characteristics and regulations. Factors considered might include flow equalization, the number of swing zones, alternative modes of operation, and safety factors.

The project team should select the recommended process in accordance with the established success criteria. The final process might include a combination of the following, in parallel or in series:

- Replace the existing process with a new process.
- Convert the existing process to a new process.
- Add a new process to the existing process.

The recommended process option should accompany an implementation plan that includes an overall schedule, funding, a construction and operational plan, costs, and startup procedures. The success of the upgrades will ensure full compliance with the new permit with good reliability in the most sustainable way (in energy usage, chemical usage, and recycle of biosolids).

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